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Signal Timing Analysis for Vehicle-Actuated Control

by

Rahmi Akçelik

Chief Research Scientist
Australian Road Research Board Ltd

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NOTES

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Rahmi Akçelik

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Related Publications

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ABSTRACT

This report presents methods for estimating average green, red and cycle times for various types of vehicle-actuated (VA) signal control. Methods for the basic VA control that uses a fixed gap setting, volume-density control by time-waiting and number-waiting gap reduction, and the Australian control method that uses headway and waste settings are described. The methods apply to both presence and passage detectors. Pedestrian crossings, pedestrians at signalised intersections, semi-actuated signals and linked VA operations are discussed. The practical cycle time method for fixed-time settings is also given. The relationships between the displayed (controller) and effective (movement) green and red times are clarified. Allowance is made for the treatment of late start and early cut-off intervals. The effects of rest intervals which are significant under low to medium flow conditions are included in the analysis. The methods are presented in terms of both the traditional uniform arrivals model (same arrival flow rate throughout the signal cycle) and a simple model of platooned arrivals (different arrival flow rates during the green and red periods). Comparisons of the estimates of queue clearance times, and average green and cycle times against simulated values are given. Appendix A discusses the headway, occupancy time, space time and gap time parameters measured using presence and passage detection, as well as their relationships to the fundamental flow, speed, density and spacing parameters. A discussion of the SCATS DS parameter is also presented. Appendix B describes methods to determine various settings for VA control. Appendix C summarises the results of a survey of Australian VA control practice. Appendix D is a reprint of an earlier paper on arrival headway distributions.

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NOTATIONS AND BASIC RELATIONSHIPS

b	Bunching factor in the formula for estimating the proportion of unbunched vehicles in the traffic stream, φ Recommended values: b = 0.6 for single-lane case b = 0.5 for two-lane case b = 0.8 for more than two lanes
C	Average *displayed* cycle time (seconds) $C = \Sigma (I + G)$, summation for all phases
c	Average *effective* cycle time for a movement (differs from the displayed cycle time due to the effect of rest intervals) (seconds) $c = r + g$
c_p	Practical cycle time: the cycle time calculated to achieve specified target degrees of saturation (x_p) and satisfy the minimum and maximum green time constraints (seconds)
DS	SCATS DS parameter (analogous to degree of saturation) $DS = (t_{og} + t_{sm}) / h_{dg}$
e_g	Gap setting as a *gap time* value as used with *passage* detection (seconds)
e_h	Gap setting as a *headway* value (seconds) $e_h = e_s + t_{ou}$ for presence detection, or $e_h = e_g + t_{vu}$ for passage detection
e_i	Vehicle increment setting for the variable initial green interval (seconds): this represents the additional minimum green per vehicle for each vehicle that actuates the detector at the start of the red period (applies with non-zero detector setback distance)
e_{min}	Minimum gap setting as a headway value for passage detection, or as a space time value for presence detection (seconds)
e_{max}	Maximum gap setting (typically $e_{max} = e_p$ as a headway value for passage detection, or $e_{max} = e_p - t_o$ as a space time value for presence detection) (seconds)
e_p	Passage time as a *headway* value (a safe time which enables the last vehicle to travel the distance between the detector and the stop line before the start of yellow signal) (seconds)
e_{rl}	Time before reduction (seconds): a setting in volume-density type VA control (the gap setting remains at its maximum value, e_{max}, during this initial period)

e_{r2} Time to reduce (seconds): a setting in volume-density type VA control (the gap setting is reduced from e_{max} to e_{min} during this period)

e_s Gap setting as a *space time* value as used with *presence* detection (seconds)

$e_s = e_h - t_{ou}$

e_t Terminating time as a *headway* value (seconds): time after the last detected vehicle and before the end of the green period ($e_t = e_h$ or e_p)

f_c A pcu factor for the actual traffic mix (used to find the saturation flow using $s = s_m / f_c$)

$f_c = p_{HV} (f_{HV} - 1) + 1$

f_{HV} A heavy vehicle factor for saturation flow purposes (pcu/veh), typically 1.5 to 2.0

f_q A calibration factor to allow for variations in queue clearance time

f_s A factor used in determining an appropriate *headway setting*: used to allow for small variations in saturation flow that would cause undesired waste increments during saturated part of green

$t_h = f_s h_s - t_{ou}$

G Average displayed (controller) green time (seconds)

$G = G_{ls} + G_{min} + G_r + G_e + G_{ec}$

g Average effective (movement) green time (seconds)

$g = G - l_s + l_e$

G_e Extension green time (seconds)

G_{ec} Early cut-off green time (seconds)

G_{emax} Maximum extension green setting (seconds)

G_{iv} Variable initial green time (does not apply in the case of stop-line detection) (seconds)

G_{ls} Late start green time (seconds)

G_m Sum of effective minimum and maximum green times for critical movements whose green times are set equal to minimum or maximum green time (seconds)

g_{max} Effective maximum green time (seconds)

$g_{max} = G_{max} - l_s + l_e$

G_{max} Maximum green setting as a controller (displayed) value (seconds)

$G_{max} = G_{min} + G_{emax}$

G_{mf} Fixed minimum green setting (seconds)

g_{min} Effective minimum green time (seconds)

$g_{min} = G_{min} - l_s + l_e$

G_{min} Minimum green setting as a controller (displayed) value (seconds)

$G_{min} = G_{mf} + G_{iv}$ (in the case of stop-line detection: $G_{min} = G_{mf}$)

g_p Effective green time for a pedestrian movement (walk plus clearance time gain less start loss)

$g_p = t_{pw} + t_{pg} - l_{ps}$

G_{pmin} Pedestrian minimum time (minimum walk time plus clearance time excluding overlap with terminating intergreen time)

$G_{pmin} = t_{pwm} + t_{pcl}$

G_r Average rest time for a given phase considering all cycles with and without rest interval (seconds)

$G_r = G'_r\ p_0$ where $p_0 = \varphi_c \exp(-\lambda_c (T - \Delta_c))$ is based on $T = I_s + G_{min}$ for the subject movement, and parameters φ_c, λ_c, Δ_c are calculated considering all conflicting movements

G'_r Average rest time for a given phase considering only the cycles with rest interval (seconds)

$G'_r = 1/\lambda_c$ where λ_c is calculated considering all conflicting movements

g_s Saturated part of the green period (average queue clearance time) (seconds) (during this period, the vehicles depart from the queue at the satur ation flow rate, s, and with the saturation headway, $h_s = 1/s$)
$g = f_q\ y\ r / (1 - y)$ (this formula assumes a single green period

g_u Unsaturated portion of the green period (seconds) (during this uninterrupted flow period after the queue clearance, the vehicles depart as they arrive at the arrival flow rate, $q_u = q_a$, or with the headway, $h_u = 1/q_u$)

$g_u = g - g_s$

G_{vmax} Maximum variable initial green setting (seconds)
(does not apply in the case of stop-line detection)

h Headway (seconds/veh): the time between passage of the *front* ends of two successive vehicles

$h = 1 / q = L_h / v = t_0 + t_s = t_g + t_v$

h_a Average arrival headway (seconds/veh) measured at a point upstream of the back of queue

$h_a = 1 / q_a$

h_{dg} Average headway of all vehicles departing during the green period including both saturated and unsaturated intervals (seconds/veh)

$h_{dg} = 1 / q_{dg} = g/n_{vg}$

h_g Average headway before gapping out after queue clearance (seconds)

h_m Average saturation headway (seconds/pcu) corresponding to the saturation flow of passenger car units (SCATS *maximum flow* MF)
$h_m = 1/s_m = t_{om} + t_{sm} = h_s / f_c$

h_n Average headway (seconds/pcu) corresponding to the maximum saturation flow rate
$h_n = 1/q_n = t_{on} + t_{sn}$

h_o *Headway setting* as a *headway* value (used as relevant to the unsaturated part of green period) (seconds)
$h_o = t_h + t_{ou}$

h_s 1. Average departure headway during the saturated part of green period, i.e. average queue discharge headway corresponding to saturation flow for the actual traffic mix (seconds/veh)
$h_s = 1/q_s = t_{os} + t_{ss}$

2. Average saturation headway for the *critical lane* (seconds/veh)
$h_s = 1/s_c$

h_t Average saturation headway considering *all lanes* (seconds/veh)
$h_t = 1/s_t$

h_u Average departure headway during unsaturated portion of the green period (seconds/veh): this occurs during g_u, corresponds to the departure flow rate which is equal to the arrival flow rate during the unsaturated portion of the green period, $q_u = q_a$, and is for the actual traffic mix)
$h_u = 1/q_u = t_{ou} + t_{su}$

h_w Average of headways (seconds/veh) between the headway setting and gap setting ($h_o < h < e_h$): used for the analysis of waste change after queue clearance (probability of such headways is p_w)

I Intergreen time (clearance time consisting of yellow and all-red time (seconds)
$I = t_y + t_{ar}$

I_s Starting intergreen time (intergreen time that precedes the current green period) (seconds)

I_t Terminating intergreen time (intergreen time that follows the current green period) (seconds)

k Density (concentration): number of vehicles per unit distance
$k = q / v = 1/ L_h$ in veh/m, or $k = 1000 / L_h$ in veh/km

k_j Jam density: number of vehicles per unit distance at zero speed, i.e. for a stationary queue
$k_j = 1/ L_{hj}$ in veh/m, or $k_j = 1000 / L_{hj}$ in veh/km

k_n Density (veh/km) corresponding to the maximum saturation flow
$k_n = q_n / v_n$ (q_n in veh/h and v_n in km/h)

l Lost time for a movement (seconds)
$l = I_s + l_s - l_e$

l_e End gain for a movement, which is associated with the use of yellow interval, also including any *early cut-off time* as an additional end gain time (seconds)

l_{pe} Negative end gain for a pedestrian movement, i.e. the unused part of clearance 1 interval (seconds)

$l_{pe} = -(t_{pc1} - t_{pg})$

l_{ps} Start loss for a pedestrian movement, i.e. the unused initial part of pedestrian walk interval (seconds)

l_s Start loss for a movement, which is associated with driver reaction time and acceleration delays until the steady saturation flow rate is reached, also including any *late start time* as an additional start loss time (seconds)

L Intersection lost time: sum of lost times for critical movements (seconds)

L_d Detector setback: distance between the downstream edge of the detection zone and the stop line (m)

L_h Spacing (m/veh): the headway *distance* as measured between the *front* ends of two successive vehicles (sum of space length and vehicle length)

$L_h = v\,h = v / q = 1 / k = L_v + L_s$

L_{hj} Average spacing (m/veh) in a stationary queue (sum of vehicle length and average space length for vehicles in a stationary queue)

$L_{hj} = L_v + L_{sj}$

L_{hs} Average spacing (m/veh) during the saturated part of the green period

$L_{hs} = L_v + L_{ss}$

L_{hu} Average spacing (m/veh) during the unsaturated part of the green period

$L_{hu} = L_v + L_{su}$

L_p Effective detection zone length (m) (likely to be within $\mp$ 0.5 m of the physical loop length)

L_s Space (gap) length (m/veh): the following distance between two successive vehicles as measured between the *back* end of one vehicle and the *front* end of the next vehicle

$L_s = L_h - L_v = v\,t_g$

L_{sj} Average space length for vehicles in a stationary queue (m)

L_{sn} Average space (gap) length (m/veh) corresponding to the maximum saturation flow rate

$L_{sn} = L_{hn} - L_v = v_n\,t_{gn}$

L_{ss} Average space (gap) length (m/veh) during the saturated part of the green period

$L_{ss} = v_s\,t_{gs} = L_{hs} - L_v$

L_{su} Average space (gap) length (m/veh) during the unsaturated part of the green period

$L_{su} = v_u t_{gu} = h_u v_u - L_v$

L_v Average vehicle length (m/veh) for the actual traffic mix

$L_v = (1 - p_{HV}) L_{vm} + p_{HV} L_{vHV}$

L_{vm} Average vehicle length for light vehicles, or passenger car units (m/LV or m/pcu)

L_{vHV} Average vehicle length for heavy vehicles (m/HV)

L_w Pedestrian walking (clearance) distance (m)

n_d The number of vehicles that can be stored in one lane between the detector and the stop line (does not apply in the case of stop-line detection)

n_g Average number of arrivals before gapping out after queue clearance

n_l Number of lanes (in the movement, lane group, approach, or all approaches relevant to the current phase)

n_r The value of the number of vehicles waiting at the time of the minimum gap setting e_{min}

n_{vc} Number of vehicles (conflicting demand) waiting on the red phase (i.e. since the start of the current green period)

n_{vg} Average number of vehicles that depart during the green period including vehicles that depart during both saturated and unsaturated parts of the green period

$n_{vg} = q_s g_s + q_u g_u = q_{dg} g$
($n_{vg} = q_{ac} c$ if $g_u > 0$, $n_{vg} = s_c g$ if $g_u = 0$)

n_{ws} The number of vehicles with positive waste increments ($w_{is} > 0$) and causing a waste change before the end of the saturated part of green period

n_{wu} The number of vehicles with positive waste increments ($w_{iu} > 0$) and causing a waste change after the end of the saturated part of green period and before a gap change

P_A Platoon arrival ratio defined as the ratio of the average arrival flow rate during the green period to the average arrival rate during the signal cycle

$P_A = q_{ag} / q_{ac}$

p_c Proportion of total flow in the critical lane: the critical lane for the purpose of this paper is the lane with the highest degree of saturation (highest flow rate if all lanes have the same saturation flow and effective green time) considering all movements that run in a phase

$p_c = q_c / q_{at}$

p_{HV} Proportion of heavy vehicles (HVs) in the stream, e.g. $p_{HV} = 0.05$ means 5 % HVs and 95 % LVs (light vehicles, or pcus)

p_o Probability of no arrival during a given period T

$p_o = \varphi\, e^{-\lambda (T - \Delta)}$

p_w Proportion of headways that give positive waste increment amongst all headways before gap change; calculated as the ratio of the probability of a headway between the headway setting and gap setting ($h_o < h < e_h$, or $t_h < t_s < e_s$) to the probability of all headways less than the gap setting ($h < e_h$, or $t_s < e_s$)

q Flow rate (veh/s or veh/h): number of vehicles per unit time passing (*arriving* or *departing*) a given reference point

$q = v / L_h = v\, k$

q_a Arrival (demand) flow rate (veh/s or veh/h), i.e. the average number of vehicles per unit time as measured at a point upstream of the back of queue

q_{at} Total arrival flow rate in all lanes (veh/s or veh/h)

q_{ac} Average arrival flow rate (veh/s or veh/h) during the signal cycle (for non-platooned arrivals: $q_{ac} = q_a$)

$q_{ac} = (q_{ar}\, r + q_{ag}\, g) / c$

q_{ag} Arrival flow rate (veh/s or veh/h) during the green period (for non-platooned arrivals: $q_{ag} = q_a$)

q_{ar} Arrival flow rate (veh/s or veh/h) during the red period (for non-platooned arrivals: $q_{ar} = q_a$)

q_c Arrival flow rate (veh/s or veh/h) in the critical lane

$q_c = p_c\, q_{at}$

q_d Departure flow rate (veh/s or veh/h), i.e. average number of vehicles per unit time as measured at the signal stop line:
$q_d = q_s$ during the saturated part of the green period
$= q_u$ during the unsaturated part of the green period

q_{dg} Average departure flow rate for the whole of green period (veh/s or veh/h)

$q_{dg} = (q_s\, g_s + q_u\, g_u) / g = n_{vg} / g$

q_n Maximum saturation flow rate (veh/h or veh/s) of the actual traffic mix (maximum departure flow rate observed at the signal stop line without any downstream queue interference)

$q_n = v_n / L_{hn} = v_n\, k_n$

q_s Average departure flow rate (veh/s or veh/h) during the saturated part of the green period, which is equal to the saturation flow rate:

- $q_s = s$ for the actual traffic mix without any downstream queue interference
- $q_s = s_m$ for a traffic stream consisting of passenger car units only without any downstream queue interference
- $q_s = s_r$ for the actual traffic mix with downstream queue interference

q_u Departure flow rate (veh/s or veh/h) during the unsaturated part of the green period, which is equal to the average arrival flow rate during the green period: $q_u = q_{ag}$ (for non-platooned arrivals: $q_{ag} = q_a$)

Q Capacity (veh/s or veh/h): maximum arrival flow rate that can be serviced under prevailing flow conditions

$Q = sg/c$

R Average displayed (controller) red time (seconds)

$R = C - G$

r Average effective (movement) red time including reduction for late arrival (seconds)

$r = R + l_s - l_e - r_d = c - g$

r_d Red time reduction for lack of arrivals during the early part of displayed red period (seconds)

$r_d = G_{mint} + 1/\lambda) \varphi \exp(-\lambda (I_t + G_{mint} - \Delta))$ where G_{mint} is the minimum green time for the conflicting phase that corresponds to the first part of the subject red period, I_t is the terminating intergreen time of the subject movement, and parameters φ, λ Δ are for the subject movement

s Average saturation flow rate of the actual traffic mix without any downstream queue interference (veh/h or veh/s)

$s = 1 / h_s = s_m / f_c$

s_c Saturation flow of the critical lane (veh/h or veh/s)

s_m Saturation flow rate of the critical lane for a stream consisting of *passenger car units* only, and without any downstream queue interference (pcu/h or pcu/s), corresponding to the SCATS maximum flow (MF) parameter

$s_m = 1/h_m$

s_r Average saturation flow rate reduced as a result of downstream queue interference (veh/h or veh/s)

s_t Total saturation flow for all lanes of a movement allowing for any lane underutilisation (veh/h or veh/s)

$s_t = \Sigma \rho_i s_i$

$s\,g$ Cycle capacity (veh) (s in veh/s, g in seconds)

t Time (seconds)

t_{ar} All-red time (seconds)

t_{eg} Average extension time by gap change after queue clearance (seconds)

$t_{eg} = n_g h_g + e_t$

t_{ew} Average extension time by waste change before gap change *after queue clearance* (seconds)

$t_{eg/w}$ Average extension time by gap change or waste change *after queue clearance* (seconds)

$t_{eg/w} = min\ (t_{eg}, t_{ew})$

t_g Gap time (seconds/veh): the time between the passage of the *back* end of one vehicle and the *front* end of the next vehicle as measured at a given point along the road, e.g. at a passage detector, or at the leading edge of a presence detector loop (this is the time taken to travel the space (gap) length, L_s)

$t_g = L_s / v = h - t_v$

t_{gn} Average gap time (seconds/veh) corresponding to the maximum saturation flow rate

$t_{gn} = L_{sn} / v_n = h_n - t_{vn} = h_n - L_v / v_n$

t_{gs} Average gap time (seconds/veh) during the saturated part of the green period

$t_{gs} = L_{ss} / v_s = h_s - t_{vs} = h_s - L_v / v_s$

t_{gu} Average gap time (seconds/veh) during the unsaturated part of the green period

$t_{gu} = L_{su} / v_u = h_u - t_{vu} = h_u - L_v / v_u$

t_h *Headway setting* as a *space time* value (applicable with presence detection) (seconds)

This can be determined as $t_h = f_s\ h_s - t_{ou}$

t_i Controller time increment (seconds): all controller timers operate on the basis of this time *step* size (t_i = 0.1 s)

t_o Detector occupancy time (seconds/veh): duration of the period when the detection zone is *occupied* by a vehicle (sum of the vehicle passage time and the time to travel the detection zone length)

$t_o = h - t_s = t_v + L_p / v = (L_v + L_p) / v$

This should not be confused with *occupancy* ratios defined as the proportion of road space covered by vehicles (Ω_s), or as the proportion of time a road section is occupied by vehicles (Ω_t)

t_{og} Average occupancy time (seconds/veh) for all vehicles departing during the green period including both saturated and unsaturated green intervals

$t_{og} = h_{dg} - t_{sg}$

t_{om} Average occupancy time (seconds/pcu) at *maximum flow* (s_m), i.e. for passenger car units departing with saturation headways (this is equivalent to the KP parameter in SCATS)

$t_{om} = h_m - t_{sm} = (L_p + L_{vm}) / v_m$

t_{on} Average occupancy time (seconds/pcu) corresponding to the maximum saturation flow rate

$t_{on} = h_n - t_{sn} = (L_p + L_{vm}) / v_n$

t_{os} Average occupancy time (seconds/veh) for vehicles departing during the saturated part of the green period

$t_{os} = h_s - t_{ss} = (L_p + L_v) / v_s$

t_{ou} Average occupancy time (seconds/veh) for vehicles departing during the unsaturated part of the green period

$t_{ou} = h_u - t_{su} = (L_p + L_v) / v_u$

t_{pc} Pedestrian clearance time (seconds): the total duration of clearance 1 and 2 intervals

$t_{pc} = t_{pc1} + t_{pc2}$

t_{pcm} A minimum value for pedestrian clearance time (seconds)

t_{pc1} Duration of pedestrian clearance 1 interval (seconds): first part of the pedestrian clearance time which does not include any overlap with the terminating intergreen time

t_{pc2} Duration of pedestrian clearance 2 interval (seconds): second part of the pedestrian clearance time which overlaps with the terminating intergreen time (also with early cut-off interval where applicable)

t_{pg} Pedestrian clearance time gain (seconds), i.e. initial part of the clearance interval which is used by pedestrians (added to effective green)

t_{pw} Pedestrian walk time (seconds): duration of walk 1 and 2 intervals

t_{pwm} Pedestrian minimum walk time (seconds): duration of walk 1 interval

t_s Space time (seconds/veh): the duration of the time between the detection of two consecutive vehicles when the presence detector loop is not occupied (this is equivalent to the gap time less the time taken to travel the effective detection zone length, L_p)

$t_s = h - t_o = t_g - L_p / v = (L_s - L_p) / v$

t_{sg} Average space time (seconds/veh) for all vehicles departing during the green period including both saturated and unsaturated green intervals

$t_{sg} = h_{dg} - t_{og}$

t_{sm} Average space time (seconds/pcu) at *maximum flow* (s_m), i.e. for passenger car units departing with average saturation headway

$t_{sm} = h_m - t_{om}$

t_{sn} Average space time (seconds/pcu) corresponding to the maximum saturation flow rate

$t_{sn} = h_n - t_{on}$

t_{ss} Average space time (seconds/veh) during the saturated part of green period

$t_{ss} = h_s - t_{os}$

t_{su} Average space time (seconds/veh) for vehicles departing during the unsaturated green interval

$t_{su} = h_u - t_{ou}$

t_{sw} Average space time (seconds/veh) corresponding to arrival headways between the headway setting and gap setting ($h_o < h < e_h$): used for the analysis of waste change after queue clearance

$t_{sw} = h_w - t_{ou}$

t_v Vehicle passage time (seconds/veh): time it takes for the vehicle length (from front end to back end) to pass a given point (this can be considered to be the occupancy time, $t_o \approx t_v$, when the detection zone length is negligible, $L_p \approx 0$).

$t_v = L_v / v$

t_{vn} Average vehicle passage time (seconds/veh) corresponding to the maximum saturation flow rate

$t_{vn} = L_v / v_n$

t_{vs} Average vehicle passage time (seconds/veh) for vehicles departing during the saturated part of the green period

$t_{vs} = L_v / v_s$

t_{vu} Average vehicle passage time (seconds/veh) for vehicles departing during the unsaturated part of the green period

$t_{vu} = L_v / v_u$

t_y Yellow time (duration of yellow interval) (seconds)

u Green time ratio

$u = g / c$

u_p Green time ratio that satisfies the practical degree of saturation requirement

$u_p = y / x_p$

U_p Sum of the required green time ratios ($U_p = \Sigma u_p$) for critical movements (seconds)

v Speed (m/s or km/h)

$v = q L_h = q / k$

v_f Average free-flow speed (km/h or m/s) for the actual traffic mix at zero-flow conditions

v_a Average approach speed (km/h or m/s) for the actual traffic mix as measured at a point upstream of the back of queue under uninterrupted conditions: this is the average cruise speed of the arrival stream at arrival flow rate q_a

v_m Average departure speed (km/h or m/s) across the stop line during the saturated part of the green period for *passenger car units*

v_n Average speed (km/h or m/s) corresponding to the maximum saturation flow rate

v_p Pedestrian speed (m/s): average walking speed for the purpose of determining the required clearance time (typically 1.2 m/s)

v_r Average departure speed (km/h or m/s) across the stop line during the saturated part of the green period for the actual traffic mix, and reduced as a result of downstream queue interference

v_s Average departure speed (km/h or m/s) across the stop line during the saturated part of the green period for the actual traffic mix

v_u Average departure speed (km/h or m/s) across the stop line during the unsaturated part of the green period for the actual traffic mix ($v_u = v_a$)

w_i Waste increment (applies during extension green interval): the difference between the measured *space time* and the *headway setting as space time value*, or between the measured *headway* and the *headway setting as a headway value* (seconds)
$w_i = t_s - t_h = h - h_o$

w_{is} Waste increment during the saturated part of green period, i.e. before queue clearance (seconds)
$w_{is} = \max(0, t_{ss} - t_h)$ where $t_{ss} = h_s - t_{os}$

w_{iu} Waste increment during the unsaturated part of green period , i.e. after queue clearance (seconds)
$w_{iu} = t_{sw} - t_h$ where $t_{sw} = h_w - t_{ou}$

w_s Waste setting (waste change occurs when the accumulated waste increments equal the waste setting) (seconds)

w_t Total waste time (sum of waste increments) (seconds)
$w_t = \Sigma\, w_i$

w_{ts} Total waste time during the saturated part of the green period (seconds)

w_{tu} Total waste time during the unsaturated part of the green period (seconds)

x Degree of saturation, i.e. the ratio of arrival (demand) flow rate to capacity, equivalent to the ratio of arrivals per cycle (q_{ac} c vehs) to the maximum number of vehicles that can depart per cycle (s g vehicles)
$x = q_{ac} / Q = (q_{ac}\, c) / (s\, g)$

x_c Degree of saturation of the critical lane

x_p Practical (target) degree of saturation (e.g. 0.90 for fixed-time signals, 0.95 for vehicle-actuated signals)

y Flow ratio, i.e. the ratio of arrival flow rate to the saturation flow rate
$y = q_{ac} / s$ where q_{ac} is the average arrival (demand) flow rate during the signal cycle (for non-platooned arrivals: $q_{ac} = q_a$)

y_c Flow ratio of the critical lane

ρ_i Lane utilisation ratio of the *ith* lane (veh/h or veh/s): ratio of the degree of saturation of the *ith* lane to the critical lane degree of saturation (if all lanes have the same effective green time, this is based on relative flow ratios)
$\rho_i = x_i / x_c$ ($\rho_i = y_i / y_c$ if all lanes have the same effective green time)

ρ_k Density ratio (ratio of the density at a given speed to the jam density)
$\rho_k = k / k_j = L_{hj} / L_h$

ρ_{kn} Density ratio at the maximum saturation flow
$\rho_{kn} = k_n / k_j = L_{hj} / L_{hn}$

ρ_v Speed ratio (ratio of a given speed to the free-flow speed)
$\rho_v = v / v_f$

ρ_{vn} Speed ratio at the maximum saturation flow
$\rho_{vn} = v_n / v_f$

Ω_s Occupancy ratio based on space (proportion of road space covered by vehicles)
$\Omega_s = L_v / L_h$

Ω_t Occupancy ratio based on time (proportion of time a road section is occupied by vehicles)
$\Omega_t = t_v / h = L_v / L_h = \Omega_s$ for passage detection
$\Omega_t = t_o / h = (L_v + L_p) / L_h$ for presence detection

Δ Minimum arrival (intra-bunch) headway in a single-lane or multi-lane traffic stream (seconds)
Recommended values:
$\Delta = 1.5$ s for single-lane case
$\Delta = 0.5$ s for all multi-lane cases
(simple negative exponential distribution of headways can be obtained by putting $\Delta = 0$)

φ Proportion of unbunched (free) vehicles in a traffic stream
(proportion bunched = $1 - \varphi$)
$\varphi = e^{-b \Delta q}$ is the recommended exponential model:
$b \Delta = 0.9$ for single-lane case
$b \Delta = 0.25$ for two-lane case
$b \Delta = 0.4$ for more than two lanes
$\varphi = 1 - \Delta q$ (valid for $q < 1/\Delta$) for the traditional Tanner model
$\varphi = 1$ for the simple negative exponential model

λ A parameter in the exponential headway distribution function

$\lambda = \varphi q / (1 - \Delta q)$ *subject to* $q \leq 0.98/\Delta$

($\lambda = q$ in the Tanner model and the simple negative exponential model)

In the formulae for φ and λ:

For the purpose of estimating the average extension time by gap or waste change after queue clearance ($t_{eg/w}$), use the average arrival rate during the green period, $q = q_u = q_{ag}$ (for uniform arrivals, use $q = q_a$).

For the purpose of estimating the average rest interval and the corresponding red time reduction (G_r, r_d), use the average arrival rate during the red period, $q = q_{ar}$ (for uniform arrivals, use $q = q_a$).

❑

1. INTRODUCTION

This report presents a method to estimate average green, red and cycle times for vehicle-actuated (VA) signal control. It extends an earlier method described by the author for the analysis of basic VA control that uses a fixed gap setting (Akçelik 1994). Some changes have been introduced to the notations used in the previous paper. A comprehensive list of notations and the basic relationships among parameters are given at the start of this report.

The analysis method for VA signal operations presented in this paper can be seen as an extension of the current Australian (Akçelik 1981, 1990), US Highway Capacity Manual (Transportation Research Board 1994), UK (Webster and Cobbe 1966) and similar methods for the analysis of fixed-time (pretimed) signals. The *practical* cycle time method for computing fixed-time signal timings (Akçelik 1981, 1990) is outlined for the purpose of comparison.

The literature on VA signal operations is limited compared with the amount of published material on fixed-time signals. However, there are still many useful papers on this subject, mostly based on the use of simulation methods, and only a limited number of them describe analytical techniques. Although a detailed review of literature is outside the scope of this paper, references are provided to various articles on the subject. The papers by Staunton (1976) and Lin (1982a,b) have been particularly useful in the initial development of the methods reported in this paper (Akçelik 1994).

The relationships between the displayed (controller) and effective (movement) green and red times are clarified. Allowance is made for the treatment of late start and early cut-off intervals. The effects of rest intervals which are significant under low to medium flow conditions are taken into account in the estimation of green and red times. To deal with rest interval effects, the concept of displayed and effective cycle times is introduced. The methods are presented in terms of both the traditional uniform arrivals model (same arrival flow rate throughout the signal cycle) and a simple model of platooned arrivals that assumes different arrival flow rates during the green and red periods.

Analysis methods are given for the Australian VA control that uses headway and waste settings, and the time-waiting gap-reduction method used in the USA. Comparisons of the estimates of queue clearance times, and average green and cycle times against simulated values are given.

The methods presented in this report relate to the traditional full and semi-actuated VA signal operations with passage and presence detectors using fixed gap time, gap reduction and waste-headway methods, as well as pedestrian-actuated signals and linked VA operations. Specific control strategies such as MOVA (Vincent and Pierce 1988), LHOVRA (Kronborg and Davidsson 1993) and SCATS Master Isolated control are not covered by this paper although the methods presented have much relevance to them.

The methods given in this report are applicable to multi-phase cases although some aspects of them are presented in terms of a simple two-phase signal case.

The methods apply to both passage and presence detectors. The headway, occupancy time, space time and gap time parameters measured using presence and passage detection, and their relationships to the fundamental flow, speed, density and spacing parameters are discussed in *Appendix A*. A discussion of the SCATS DS parameter is also presented. Methods to determine various settings for VA control, based on the current Australian methods are described in *Appendix B*.

Appendix C presents a summary of the findings from a survey of VA control practice in Australia.

Appendix D is a reprint of an earlier paper on arrival headway distributions (Akçelik and Chung 1994a).

2. ACTUATED CONTROLLER OPERATION

This section presents a description of how the VA signal controllers work before discussions of the displayed and effective green, red and cycle times and methods to estimate green, red and cycle times at VA signals are given in the following sections. Although the discussions are relevant to vehicle-actuated controllers generally, it is emphasised that specific aspects of controller operation are valid for the Australian controllers. Therefore, the analysis methods given in this report should be applied to VA controller operations in other countries with care.

Two useful Australian documents that discuss the subject of traffic signal detection in some detail, and give methods for selecting the gap, headway and waste settings are the RTA (1991) manual for traffic signal operation and the AUSTROADS (1993) guide to the design of traffic signal installations. Some aspects of these guides have been discussed recently by the author (Akçelik 1995). The descriptions of the operation of VA controllers given in this section are heavily based on the material given in RTA (1991).

At vehicle-actuated signals, the green times, and hence the cycle time, are determined according to the vehicle demands registered by detectors. This may be on the basis of phase (stage) control or group (movement) control. Phase sequence may be fixed or variable. A phase can be skipped when there is no demand for it. A phase consists of various intervals. *Figures 1a and 1b* show the phase intervals for vehicle traffic and for pedestrian movements.

The ***vehicle phase intervals*** used in the Australian controller (RTA 1991) are defined below (see *Figure 1a*).

Without the early cut-off interval, the *running* part of the phase corresponds to the period when the green signal is displayed, and the *clearance* part of the phase corresponds to the period when the yellow and all-red signals are displayed. The *intergreen period* consists of the yellow and all-red intervals ($I = t_y + t_{ar}$). Where applicable, the *early cut-off green* interval is considered to be part of the phase clearance period, although it is part of the displayed green time for the phase.

Different signal indications can be displayed to different movements using the phase during the *late start* and *early cut-off* intervals. The *late start* interval is used to delay the introduction of green signal to some movements in the phase. The durations of these intervals are determined by the *late start* and *early cut-off green* settings (G_{ls}, G_{ec}). Early cut-off yellow interval is used to provide yellow interval for any movement which is terminated at the start of the early cut-off interval.

The *minimum green period* (duration G_{min}) is used for safety reasons, and consists of the *minimum green* interval and *variable initial green* interval. The duration of the minimum green interval is determined by the *minimum green* setting (G_{mf}). The *variable initial green* interval is used with advanced detectors to provide additional minimum time to discharge a queue of vehicles stored between the signal stop line and the detectors during the red period. The duration of this interval (G_{iv}) varies in response to the number of actuations of

the advance detectors and is determined according to the values of the *vehicle increment* (e_i) and *maximum initial green* (G_{vmax}) settings. This is discussed in detail in *Section B.3* of *Appendix B*.

Thus, the sum of the durations of the minimum green and variable initial green intervals is called the *minimum green time* ($G_{min} = G_{mf} + G_{iv}$). With stop-line presence detectors, the minimum green time is of fixed duration, $G_{min} = G_{mf}$.

The controller cannot enter the extension green interval until there is a demand for another phase. The *rest* interval is an untimed interval after the minimum green period, during which the controller rests until there is a conflicting phase demand. This interval is skipped if a demand is registered for other phases before the end of the minimum green time.

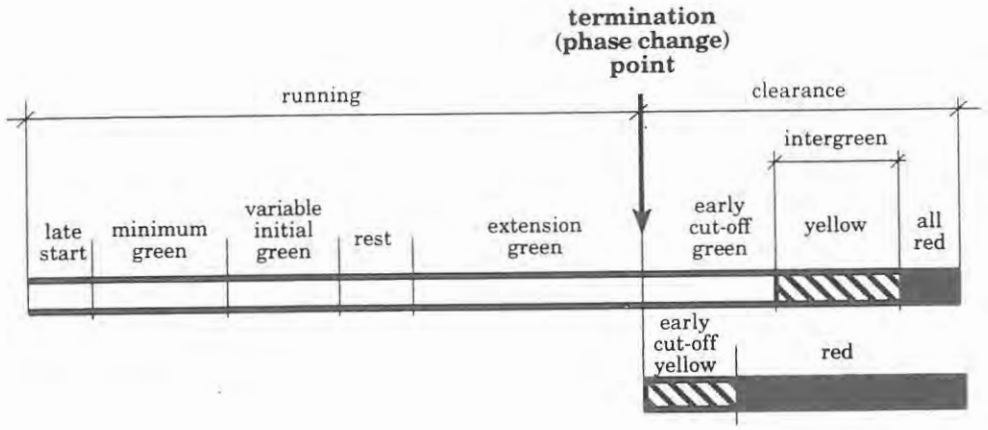


Fig. 1a – Definition of phase intervals for vehicle traffic (Source: RTA 1991)

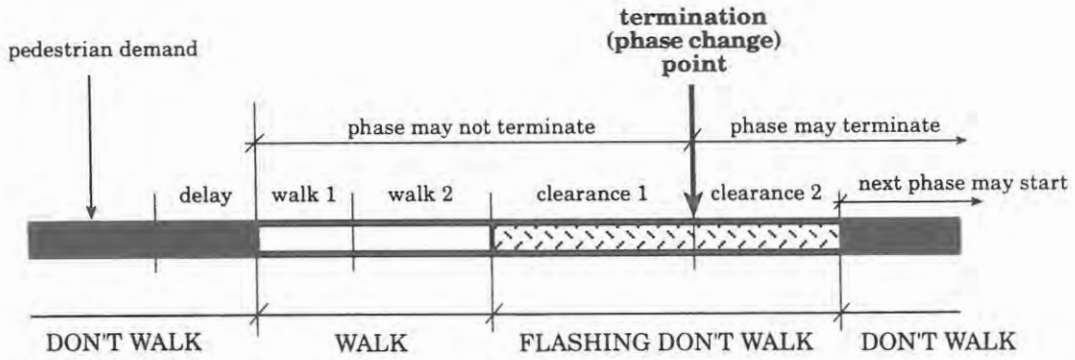


Fig. 1b – Definition of phase intervals for pedestrian traffic (Source: RTA 1991)

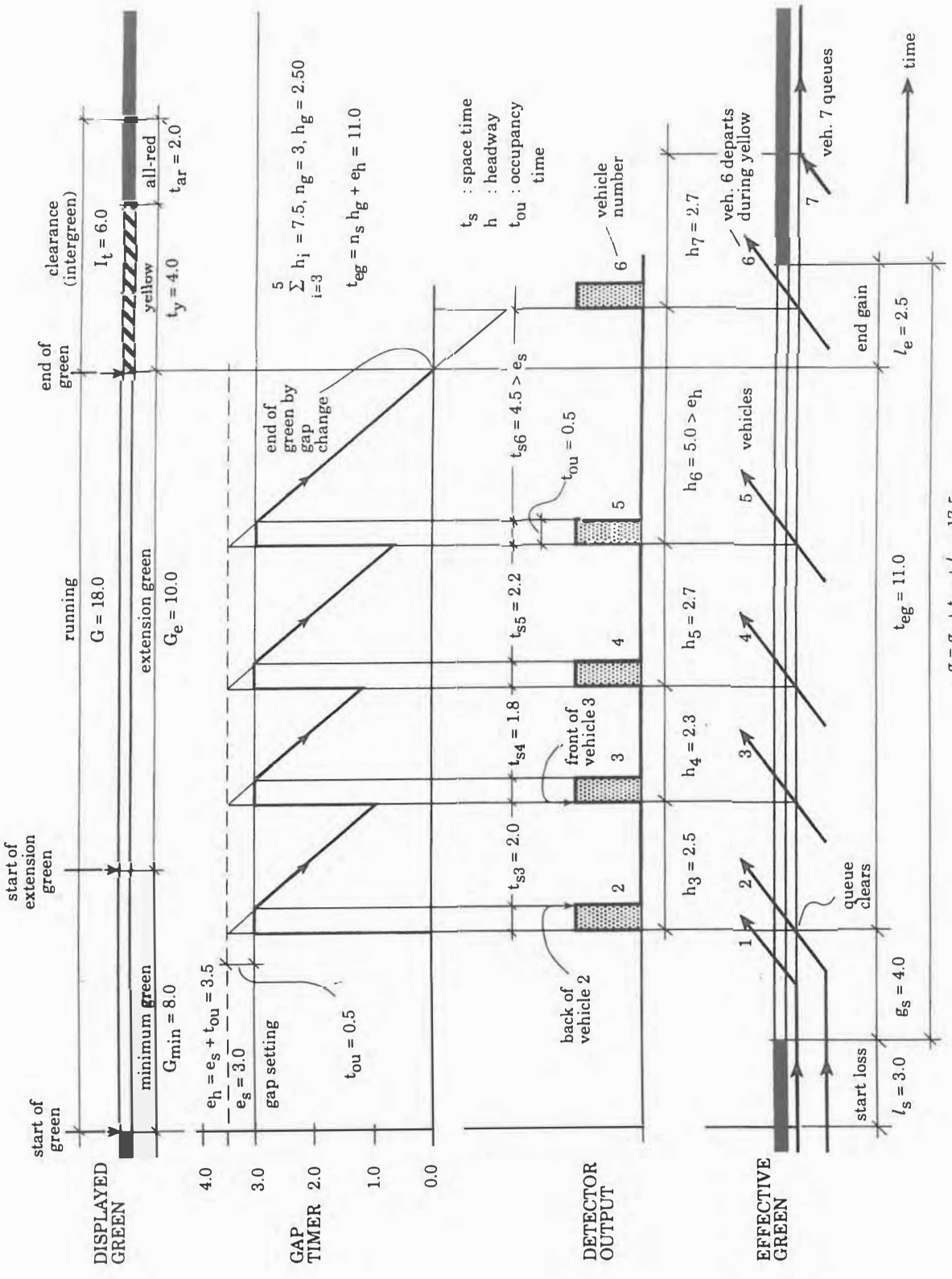


Fig. 2 – The operation of basic VA control that uses a fixed gap setting (an example)

In this paper, G_r' is used to denote the duration of the rest interval in a given cycle, or to denote the average rest time for all signal cycles with rest intervals, whereas G_r is used to denote the average rest time for all signal cycles with and without rest intervals.

The *extension green* interval is of variable length, and under isolated operation, its duration (G_e) is determined by extension settings, namely the *gap setting* (e_s, e_g or e_h), *headway* and *waste* settings (t_h, w_s) and *maximum extension green setting* (G_{emax}).

As discussed in *Section B.5 of Appendix B*, the gap setting can be a *space time* value (e_s) as used with presence detection, a *gap time* value (e_g) or a *headway time* value (e_h) as relevant to passage detection. In the analysis methods given in later sections of the report, the headway equivalent of the gap setting (e_h) is used always. The headway and waste settings are used as space time values as they are used with presence detection in the Australian practice.

A simple controller operation mode using a fixed value of the gap setting (also called vehicle interval, vehicle extension or unit extension) and a maximum extension green setting will be referred to as the ***basic VA control***. An example for the operation of basic VA control is shown in *Figure 2*. In this example, a single-lane movement is considered, and it is assumed that a conflicting demand is registered before the termination of the minimum green period, and therefore, the *extension period* starts immediately after the expiry of the *minimum green* period. The gap setting is $e_s = 3.0$ s (the occupancy time is $t_{ou} = 0.5$ s, and the headway equivalent of this setting is $e_h = e_s + t_{ou} = 3.5$ s).

The gap timing logic operates from the start of the green period to enable a green termination at the end of the minimum green period or the rest interval. It operates for the whole of the running part of the phase including the rest interval. The *gap timer* is loaded with the gap setting when a detector actuation occurs. With presence detection, the gap timer starts decrementing (from the initial value, e_s) when the detector actuation ceases. As shown in *Figure 2*, this is equivalent to loading the gap timer with the equivalent headway time (e_h) and decrementing it from the start of the detector actuation (this statement assumes that appropriate occupancy time values are used to relate e_s to e_h).

If the gap timer reaches zero before another detector actuation occurs (i.e. when $h \geq e_h$ or $t_s \geq e_s$ where $t_s = h - t_{ou}$), the timer is said to have *timed out* (in *Figure 2*, this occurs after the passage of Vehicle 5). When this occurs during the extension green interval, the green period is terminated. This point during the phase is called the *phase change (termination) time*. This type of extension green termination will be called a *gap change*. The phase change time starts the yellow signal display unless there is an early cut-off green interval.

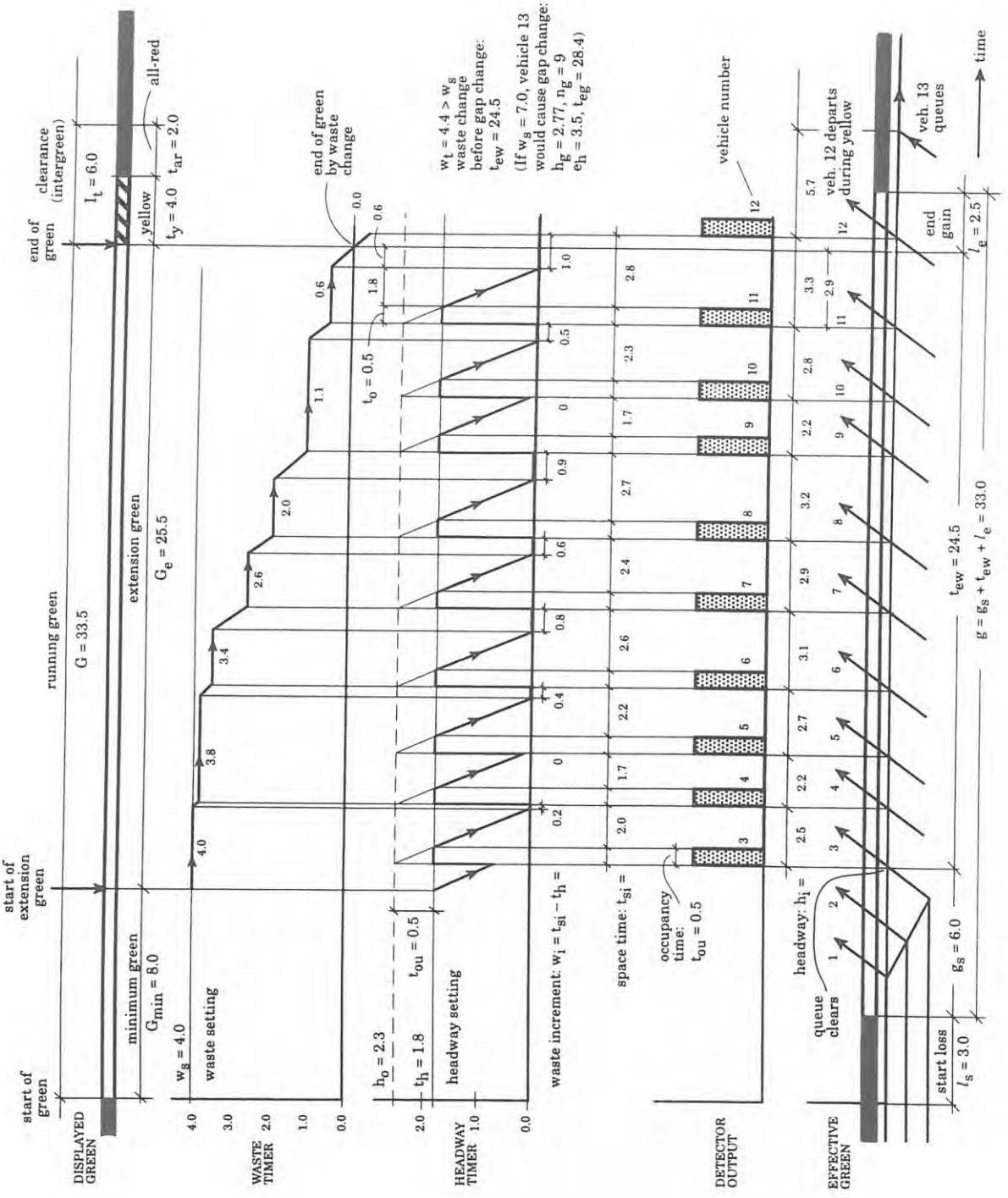


Fig. 3 – The operation of VA control that uses headway and waste settings (an example)

During the extension green interval, detection of each additional vehicle extends the green period by an amount equal to headway time of the vehicle ($h < e_h$). The extension time for the last vehicle before a gap change is e_h, i.e. the headway equivalent of the gap setting. In some controllers, a *passage time* setting (e_p) is used instead of the *gap setting* for the last vehicle to be able to travel the distance between the detector and the stop line before the start of yellow signal. Thus, the *terminating time* (e_t) at gap change is either the headway equivalent of the gap setting ($e_t = e_h$) or the passage time ($e_t = e_p$). In this report, the passage time is considered to be specified always as a headway value.

A ***variable gap setting*** method is employed in the USA using a linear function to reduce the gap setting between *maximum and minimum gap settings* (e_{max} and e_{min}) during a specified time duration. Thus, at the time of each detector actuation, a different gap setting applies until the minimum gap setting is reached. This is discussed in more detail in *Section 4.2*.

The current Australian control method employs ***headway and waste settings*** as additional extension settings. An example for the operation of VA control that uses headway and waste settings is shown in *Figure 3*. As in *Figure 2*, a single-lane movement is considered, and it is assumed that a conflicting demand is registered before the termination of the minimum green period, and therefore, the *extension period* starts immediately after the expiry of the *minimum green* period. The headway setting is $t_h = 1.8$ s, and the waste setting is $w_s = 4.0$ s.

The headway-waste control method aims to terminate the extension green interval *before gap change* if the headways are too small for a gap change but too large for efficient traffic operation. The efficiency is measured by difference of measured space times from the headway setting which is related to the saturation space time. This is equivalent to the difference between the measured headway and a headway corresponding to the saturation space time (see *Section B.6 of Appendix B*). The difference is called a *waste increment*.

The method is implemented as follows. A *waste timer* operates throughout the running part of the phase, but its operation is ignored until the start of the extension green interval (i.e. until the end of the minimum green period or until a conflicting phase demand is received, whichever comes later). At the start of the extension green interval, and at each detector actuation after that, the headway timer is loaded with the headway setting. When the detector actuation ceases (i.e. at the end of the occupancy time), the headway timer starts decrementing as seen in *Figure 3*. If the headway timer reaches zero before another actuation occurs, the timer is said to have *timed out*. The headway timer may time out many times during a phase.

The value of the waste timer at the start of the extension green interval equals the waste setting. Whenever the headway timer is timed out, the *waste timer* starts decrementing until a new detector actuation occurs. The amount of decrement equals the waste increment. When the waste timer reaches zero, the waste timer is said to have timed out. When this occurs before a gap

change, the phase will be terminated. This will be referred to as a *waste change*.

In addition to a gap change or waste change, the phase can be terminated by a *minimum change*, or a *maximum change*. The *minimum change* occurs when the phase ends at the end of the minimum green period when a conflicting phase demand has been received and the gap timer has timed out before the end of the minimum green period ($G = G_{min}$). The *maximum change* occurs when a gap change or waste change has not occurred during the extension green interval and the total green extension time equals the maximum extension green setting ($G_e = G_{emax}$).

In summary, the green period can be terminated by one of four methods:

(i) a *minimum change* ($G_r' = 0$, $G_e = 0$, $G = G_{min}$),

(ii) a *waste change* ($G_e < G_{emax}$, $G_r' \geq 0$, $G = G_{min} + G_r' + G_e$),

(iii) a *gap change* ($G_e < G_{emax}$, $G_r' \geq 0$, $G = G_{min} + G_r' + G_e$), or

(iv) a *maximum change* ($G_e = G_{emax}$, $G_r' \geq 0$, $G = G_{min} + G_r' + G_{emax}$).

The analysis of the rest and vehicle extension intervals (G_r', G_e) is key to the estimation of the duration of green time under vehicle-actuated control (see *Sections 3 and 4*).

The ***pedestrian phase intervals*** used in the Australian controller (RTA 1991) are shown in *Figure 1b*. Pedestrian movements can run *concurrently* with parallel vehicle movements, or can run in an *exclusive* pedestrian phase (see *Figure B.1* in *Appendix B*). Normally, they are introduced by pedestrian demand received before the relevant phase green starts (push button operation), or by automatic introduction in areas where heavy pedestrian movements exist.

Pedestrian displays are *walk*, *flashing don't walk* (clearance) and *don't walk*. Pedestrian walk interval (duration t_{pw}) consists of *walk 1* (a minimum time for walk display, t_{pwm}) and *walk 2* (an untimed interval of additional walk display which is terminated by the flashing don't walk display). The flashing don't walk display corresponds to *clearance intervals 1 and 2* (duration t_{c1} and t_{c2}). The clearance 2 interval overlaps with the vehicle clearance interval (including early cut-off period where applicable). The *delay* interval shown in *Figure 1.b* is rarely used (its purpose is to delay the registration of pedestrian demand).

Methods to determine pedestrian settings (durations of minimum walk and clearance periods) are discussed in *Section B.7* of *Appendix B*.

Minimum phase time requirement for a phase with concurrent pedestrian and vehicle movements is increased because of pedestrian walk and clearance time requirements. For the purpose of critical movement analysis to determine the phase time, it is convenient to use a *pedestrian minimum time* (G_{pmin}) which is equivalent to the sum of minimum walk and clearance 1 interval time as discussed in *Section 4.4 and Section B.7* of *Appendix B*.

For the purpose of performance analysis, i.e. to calculate delays, etc. to pedestrians, it is necessary to calculate effective green and red times for pedestrian movements. This is discussed in *Section B.7* of *Appendix B*.

Selecting appropriate values of many control settings for efficient operation of vehicle-actuated signals (for a given phasing system) is not an easy task. The location, number and other characteristics of detectors affect the choice of vehicle-actuated settings also. A discussion of basic parameters measured using presence and passage detectors, and the relationships among basic parameters are given in *Appendix A*. It is difficult to give general-purpose rules for choosing the values of vehicle-actuated controller settings, which can be used for any controller and location. The methods for determining VA control settings based on current Australian practice are described in *Appendix B*. Further research is needed on the selection of VA control settings for improved traffic performance as discussed in the conclusion section.

3. DISPLAYED AND EFFECTIVE GREEN, RED AND CYCLE TIMES

Before discussing the methods to estimate the duration of green period under vehicle-actuated control, it is necessary to clarify the relationships between the *displayed* (controller) and *effective* (movement) green, red and cycle time. These are shown in *Figure 4* together with the minimum green (G_{min}), rest time (G_r') and the extension green time (G_e) considering a signal cycle with rest interval (conflicting demand received after the end of the minimum green period). In *Figure 5*, the relationships are shown in a more complete example of a two-phase system where green, red and cycle times are averages for all signal cycles (thus, the rest time, G_r, and the red time adjustment, r_d, are averages for all cycles with and without rest intervals).

In this report, upper case letters (G, R, etc.) are used for displayed times and lower case letters are used to indicate effective times (g, r, etc.) .

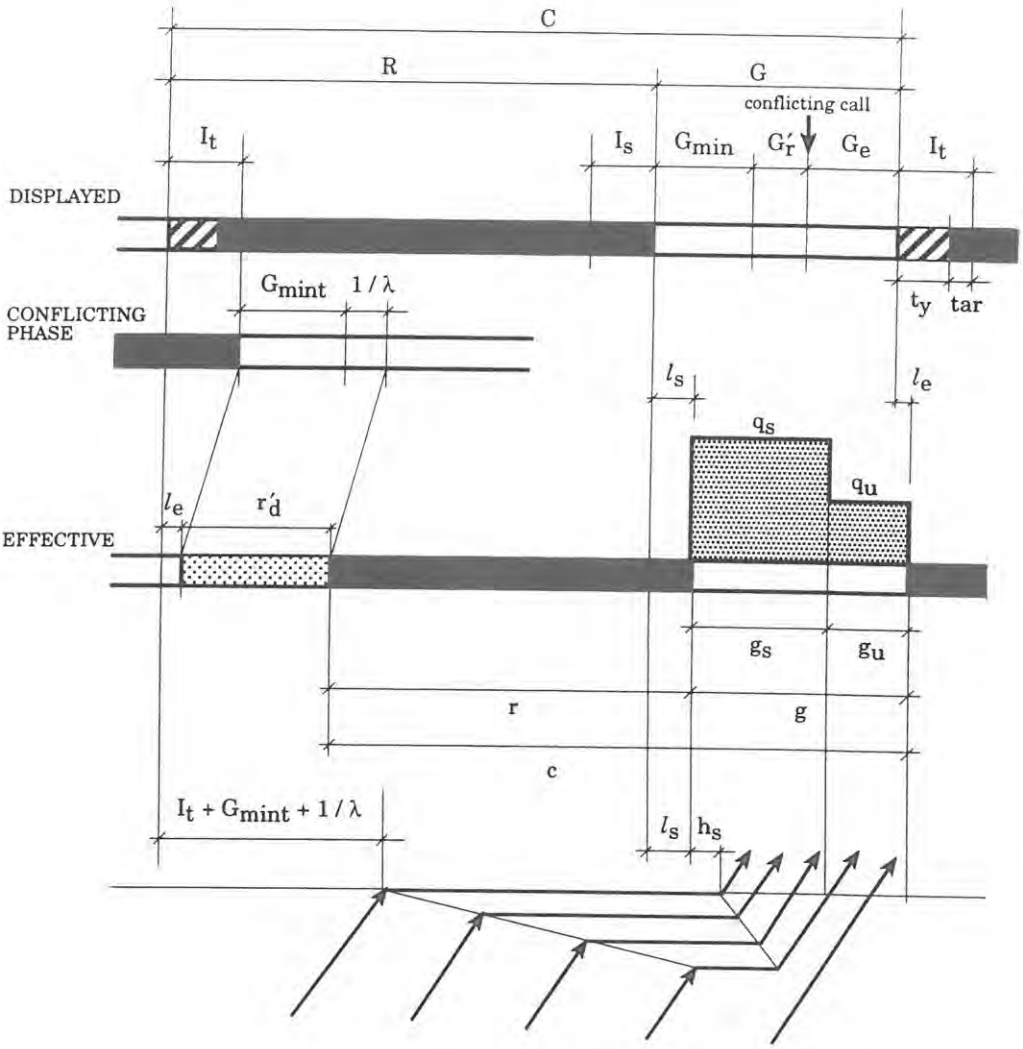


Fig. 4 – Displayed and effective green, red and cycle times for a given movement: the case with rest interval

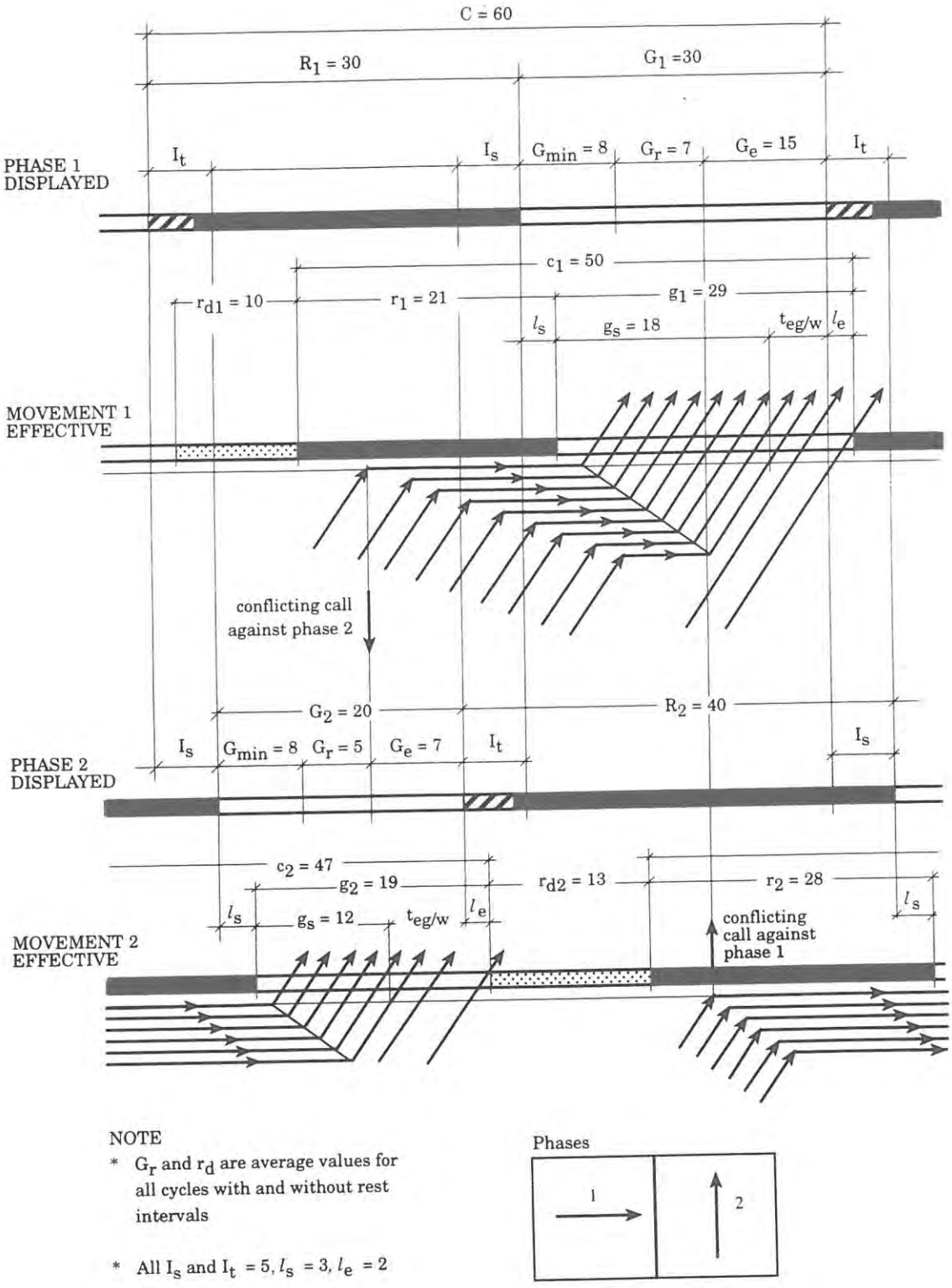


Fig. 5 – Displayed and effective green, red and cycle times for a two-phase system

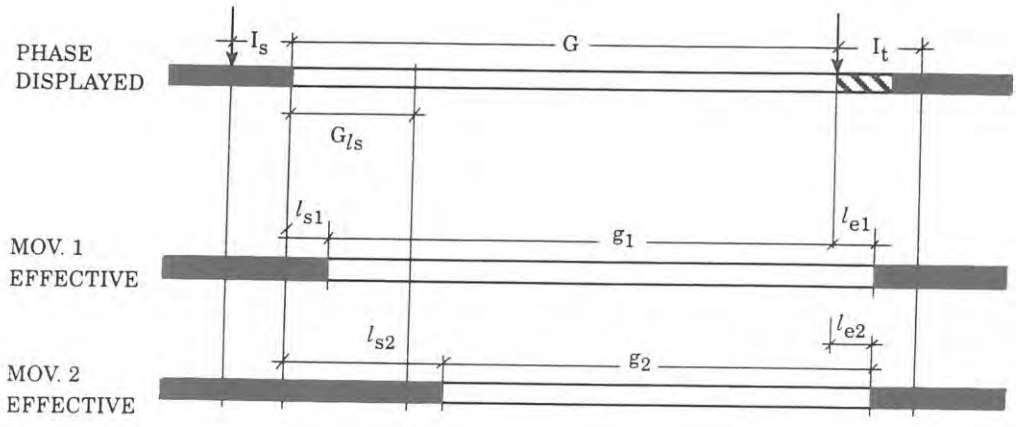


Fig. 6a – Treatment of the late start green time as part of start loss

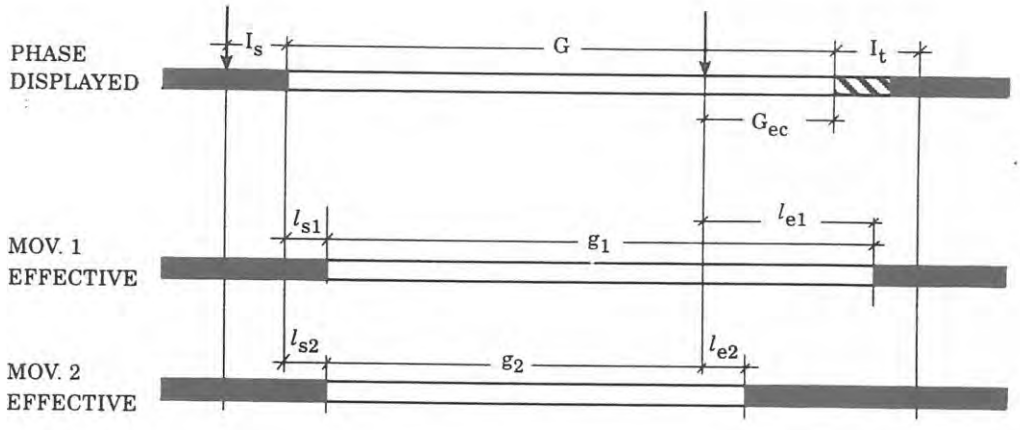


Fig. 6b – Treatment of the early cut-off green time as part of end gain

The displayed (phase) times are relevant to the controller operation whereas the effective (movement) times are needed for computational purposes as used traditionally for the analysis of fixed-time signal operations (Akçelik 1981). The difference between the displayed and effective times becomes more complex for vehicle-actuated control, particularly due to the existence of rest periods. This is discussed in more detail in the sections below presenting the basic formulae for effective green, red and cycle times.

The effective times can be calculated from displayed times by reference to *phase change times* and using the appropriate *intergreen*, *start loss* and *end gain times* for a given movement.

Phase change time is the time when the current phase is terminated, i.e. when the green period ends and clearance interval starts. The sum of the yellow and all-red times is called *intergreen time* ($I = t_y + t_{ar}$). The intergreen time that belongs to the current phase is called the *terminating intergreen time* (I_t). The terminating intergreen of one phase is the *starting intergreen time* (I_s) for the next phase. The *lost time* for a movement is calculated as the sum of its starting intergreen time and start loss time less end gain.

Start loss (l_s) for a movement is associated with driver reaction time and the acceleration delays until the steady saturation flow rate is reached. The *late start green time* (G_{ls}) is added to the start loss times of relevant movements (see *Figure 6a*). End gain (l_e) for a movement is associated with the departure of vehicles during the yellow signal display. Any *early cut-off green time* (G_{ec}) is added to the end gain times of relevant movements (see *Figure 6b*).

Effective green time

As seen in *Figures 4 to 6*, the effective green and lost times for a movement are calculated from:

$$g = G - l_s + l_e \quad (1)$$

$$l = I_s + l_s - l_e \quad (2)$$

where

g = effective green time (seconds),
G = displayed green time (seconds),
l_s = start loss (seconds),
l_e = end gain (seconds), and
l = lost time (seconds).

Therefore, the general relationship between effective and displayed green times, intergreen time and lost time is given by:

$$g + l = I_s + G \quad (3)$$

Cycle time

The signal cycle time is the sum of all phase green and intergreen times:

$$C = \Sigma (G + I_t) \quad (4)$$

This is a displayed cycle time which is also equivalent to the sum of effective green and lost times for critical movements (see Akçelik 1981):

$$C = \Sigma (g + l) \quad (4a)$$

For application of the critical movement identification method described by Akçelik (1981, 1990) to the case of actuated signals, average green time estimates can be used as *required green times*. The method compares the sum of effective green and lost times for all combinations of movements, and determines the critical movements as the set of movements that require the greatest sum of phase times. This total time is, in fact, equivalent to the cycle time as given by *Equation (4a)*.

The existence of rest periods requires that the effective red time of the current phase is calculated by applying a reduction for lack of arrivals at the start of the red period (associated with a rest interval for the conflicting phase). As a result of this, each movement will appear to be operating with a different effective cycle time although displayed cycle times are the same for all movements (see *Figures 4 and 5*). The effective cycle time for a given movement can be calculated from:

$$c = r + g \quad (5)$$

where r is the effective red time given by *Equation (7)* and g is the effective green time given by *Equation (1)*.

Red time

The displayed red time (R) is given by:

$$R = C - G \quad (6)$$

where C is the signal cycle time given by *Equation (4)* and G is the displayed green time.

For a two-phase system (see *Figure 5*), the displayed red time for a given phase can be calculated from the green time of the conflicting phase (G'):

$$R = I_t + G' + I_s \quad (6a)$$

where I_t and I_s are the terminating and starting intergreen times for the given phase. For more than two phases, green and intergreen times of all conflicting phases should be added to the displayed red time of the given movement.

A general equation for the effective red time is:

$$r = R + l_s - l_e - r_d \quad (7)$$

where R is the displayed red time, l_s and l_e are the start loss and end gain times, and r_d is the red time reduction for lack of arrivals during the early part of the displayed red period (see *Figure 4*) given by

$$r_d = (G_{mint} + \frac{1}{\lambda}) \varphi e^{-\lambda (I_t + G_{mint} - \Delta)} \quad (8)$$

where

G_{mint} = the minimum green time for the conflicting phase that corresponds to the first part of the subject red period (seconds), and
I_t = the terminating intergreen time of the subject movement,

and parameters φ, λ, Δ are for the subject movement. For the calculation of parameters φ, λ, Δ, see *Equations (15) and (15a)* in *Section 3*. The arrival flow rate to be used for this purpose is the average arrival flow rate during the red period (q_{ar}). Under the uniform arrivals model, use $q_a = q_{ar}$ (i.e. the same arrival flow rate applies during the red and green periods).

Rest time

The red time reduction for lack of arrivals during the early part of the displayed red period (r_d) given by *Equation (8)* corresponds to the rest time for the conflicting phase.

For a given phase, the average rest time considering all cycles with and without rest interval, G_r, can be calculated from:

$$G_r = G'_r \, p_o = G'_r \, \varphi_c \, e^{-\lambda_c (I_s + G_{min} - \Delta_c)} \qquad (9)$$

where

G'_r = average rest time for a given phase considering only the cycles with rest interval (seconds) given by:

$$G'_r = 1 / \lambda_c \qquad (9a)$$

and

p_o = the probability of no arrivals during the starting intergreen (I_s) and the minimum green (G_{min}) periods of the subject movement.

The general formula for calculating the probability of no arrivals during a given period of T seconds is:

$$p_o = \varphi \, e^{-\lambda (T - \Delta)} \qquad (10)$$

In *Equation (9)*, $T = I_s + G_{min}$ is for the subject movement, and parameters φ_c, λ_c, Δ_c are calculated considering all conflicting movements.

Equation (10) can be used for other purposes such as:

(i) allowing for the possibility of no vehicle arrivals resulting in phase-skipping under low flow conditions: in this case, use T = r where r is the red time for the subject movement, and

(ii) allowing for the effect of signal cycles with and without pedestrians: in this case, use $T = c - t_{pw}$ where t_{pw} is the duration of the pedestrian walk time (see *Section 4.4*).

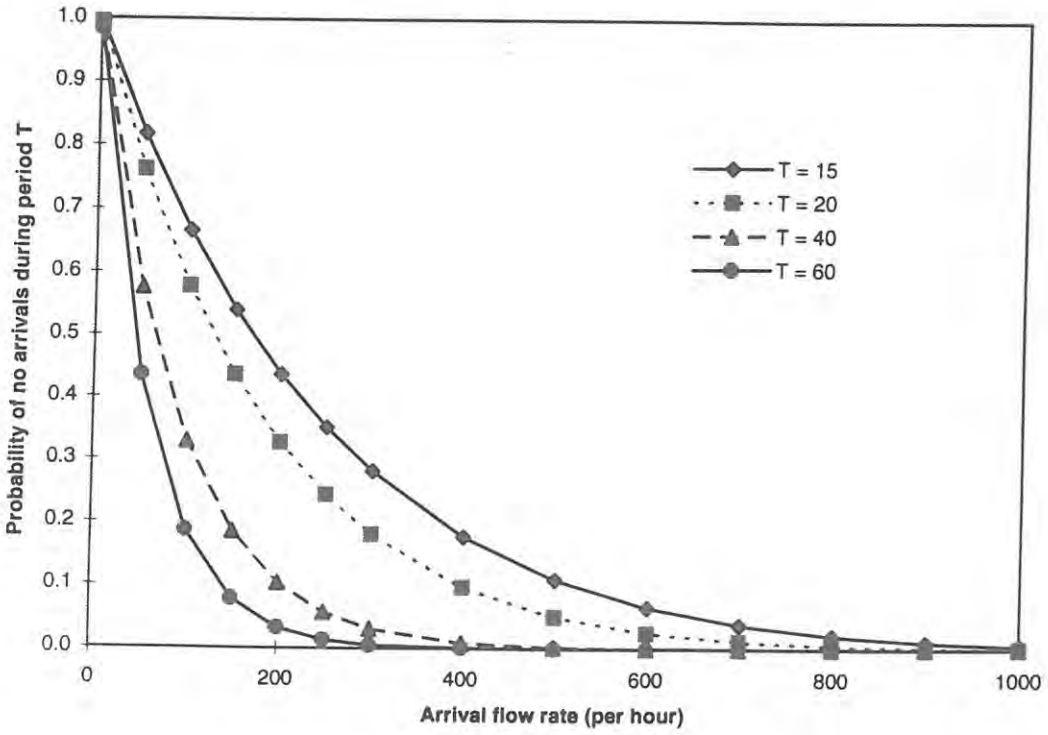


Fig. 7 – Probabilities of no arrivals during given periods of duration T = 15, 20, 40 and 60 seconds (p_o) as a function of the flow rate

The probability of no arrivals (p_o) during given periods of duration T = 15, 20, 40 and 60 seconds as a function of the total arrival flow rate is shown in *Figure 7* for a single-lane movement. It is seen that p_o is significant only for low flow rates when T is large, but can be significant up to medium flow levels when T is low (as in the case of estimating rest time). Therefore, for manual methods dealing with high flow conditions, the effect of the rest interval could be neglected, simplifying the method considerably. However, the formulation of a method to estimate the rest time is important for dealing with low to medium flow conditions in general, and with phase skipping, pedestrian movements and semi-actuated signals, in particular.

Saturated part of the green period

In addition to the estimation of the rest time, the estimation of the duration of the *saturated part of the green period* (g_s) is essential for the estimation of green time at VA signals. As seen in *Figures 4 and 5*, the saturated part of the green time represents the queue clearance time, i.e. the period when the vehicles depart from the queue at the departure flow rate, q_s. Under normal circumstances, this departure flow (queue discharge) rate equals the steady saturation flow rate, $q_s = s$. Thus, during the saturated part of the green period, the vehicles are modelled to depart with equal saturation headways, $h_s = 1 / s$, after the start loss time following the start of phase green period.

In multi-lane cases, the saturated portion of green should represent the time to clear the queue in the *critical lane* (i.e. the longest queue for any lane) considering all lanes of all movements (lane groups or approaches) using the phase. Ideally, g_s should be calculated for each lane in each approach using parameters relevant to each lane (e.g. flow, saturation flow and effective red time for the lane). This method requires the estimation of lane flows as used in SIDRA. Alternatively, g_s can be calculated for each movement (lane group or approach) by using a saturation flow rate that ensures that g_s for the lane group approximates the critical lane value. See *Section B.2* in *Appendix B* for a method which yields the same flow ratio (y value) for the movement and the critical lane.

The saturated portion of green period (g_s) can be estimated from the following formulae:

For platooned arrivals:

$$g_s = f_q \frac{y\,(c - P_A\,g)}{1 - P_A\,y} \tag{11}$$

For non-platooned (uniform) arrivals:

$$g_s = f_q \frac{y\,r}{1 - y} \tag{11a}$$

where

f_q = queue clearance time factor,

s = saturation flow rate (veh/s),

y = flow ratio ($y = q_{ac} / s$ where q_{ac} = average arrival flow rate during the cycle, for uniform arrivals, $q_{ac} = q_a$),

c = effective cycle time, $c = r + g$, (seconds),

r = red time (seconds),

P_A = platoon arrival ratio defined as the ratio of the average arrival flow rate during the green period, q_{ag}, to the average arrival flow rate during the cycle, q_{ac}:

$$P_A = q_{ag} / q_{ac} \tag{12}$$

The assumptions of the platooned arrival model used here is the same as the model used the US Highway Capacity Manual (TRB 1994) for progression factors as a simple model to allow for signal coordination effects on traffic performance (also see Olszewski 1990, Fambro, Chang and Messer 1991). The model assumes different arrival flow rates during the red and green periods (q_{ar} and q_{ac}). Given the average arrival flow rate during the cycle, q_{ac}, and the platoon arrival ratio, P_A, the arrival rates for the green and red periods are calculated from:

$$q_{ag} = P_A\,q_{ac} \tag{12a}$$

$$q_{ar} = \frac{q_{ac}\,c - q_{ag}\,g}{r} \tag{12b}$$

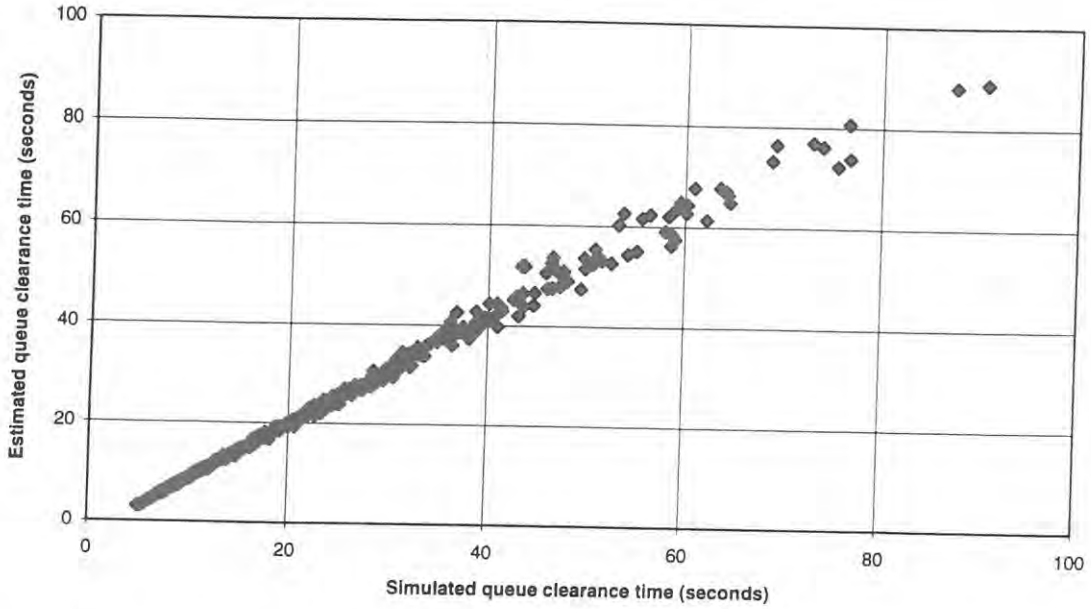


Fig. 8 – Comparison of estimated and simulated average queue clearance times by gap change without maximum green constraint

The more traditional uniform arrivals model assumes $P_A = 1.0$ and $q_{ar} = q_{ag} = q_{ac} = q_a$, and since $c - g = r$, *Equation (11a)* is seen to be a special case of *Equation (11)*. The most common form of the queue clearance time formula used in the literature is *Equation (11a)* based on uniform arrivals assumption with $f_q = 1.0$.

The queue clearance time factor was introduced in the formulae for fixed-time signals and unsignalised intersections in a paper by Akçelik and Chung (1994b). Through more recent simulation studies (to be reported in detail in a future report), the following formula has been developed for the calculation of the queue clearance time factor for actuated signals:

$$f_q = 1.05 + 0.0022\,\varphi_c\,(s_c\,g) \qquad (13)$$

where φ_c is the proportion of unbunched traffic in the critical lane (see *Equation 15*), and $(s_c\,g)$ is the cycle capacity (vehs) for the critical lane where s_c is the critical lane saturation flow (veh/s) and g is the effective green time (seconds).

Equation (13) is obtained through an iterative calibration process since the values of f_q affect the green times, and therefore the red times, which in turn affect the queue clearance times. As a result, f_q is dependent on the control method to some extent.

Figure (8) shows the comparison of the queue clearance times estimated using *Equations (11a) and (13)* with those obtained from the microscopic MODELC simulation model (Akçelik and Chung 1994b). The data points correspond to the simulation of a simple two-phase system as shown in *Figure 5*, using

single-lane movements with equal saturation flows (s = 1800 veh/h). The total arrival flow rate for both movements was varied between 800 and 1500 veh/h, and three ratios of the arrival flow rates for the two movements were applied (1:1, 1:1.5 and 1:2). For each combination of flows, basic VA control conditions with gap settings in the range e_s = 1.7 to 5.5 s were simulated using G_{min} = 10 s and no maximum green time constraint. *Equation (13)* was calibrated for this data set.

Unsaturated part of the green period

The duration of the *unsaturated portion of the green period* (g_u) is given by:

$$g_u = g - g_s \quad \textit{subject to } g_u \geq 0 \tag{14}$$

With actuated signals, $g_u = 0$ occurs when the phase is terminated before the end of the queue clearance by a maximum change or a waste change (assuming that a gap change never occurs during the saturated part of the green period as discussed in *Section 3*). In this case, $g < g_s$ results, and therefore $g_s = g$ is set.

The case of non-zero g_u represents the conditions when the phase terminated by a minimum change, a gap change, or a waste change after queue clearance.

During the unsaturated part of green period, which represents the uninterrupted flow conditions after queue clearance, the vehicles depart as they arrive at the average arrival flow rate during the green period, i.e. $q_u = q_{ag}$ (for uniform flow conditions, $q_u = q_a$). This is the arrival flow rate to be used in the estimation of the average extension time by gap change or waste change after queue clearance. This arrival flow rate corresponds to arrivals at the back of the queue during the saturated part of the green period. Note that the same arrival flow rate is used for calculations related to the red period (G_r and r_d) under the uniform arrivals model (q_a), but different arrival flows are used for the red and green periods under the platooned arrivals model ($q_{ar} \neq q_{ag}$).

The average departure headway during g_u is $h_u = 1 / q_u$. For an isolated intersection approach, the individual headways during this period are distributed randomly. The model used for the distribution of arrival headways for isolated intersection approaches is discussed below.

Distribution of Arrival Headways

The recommended arrival headway distribution model for isolated intersection approaches is based on a bunched exponential model proposed by Cowan (1975). A detailed discussion of this model and the results of its calibration using real-life data for single-lane traffic streams and simulation data for multi-lane streams are given in Akçelik and Chung (1994a). See *Appendix D*.

The bunched exponential model can be used either with a known (measured) value of φ, or more generally, with a value of φ estimated as a function of the flow rate. The following exponential relationship can be used for estimating the proportion of free (unbunched) vehicles in the traffic stream (φ):

$$\varphi = e^{-b \Delta q} \tag{15}$$

where b is a bunching factor, Δ is the minimum arrival (intra-bunch) headway, and q is the flow rate (veh/s). The recommended parameter values are:

Single-lane case: $\Delta = 1.5$ s and $b = 0.6$ ($\varphi = e^{-0.9\,q}$),

Multi-lane case (number of lanes = 2): $\Delta = 0.5$ s and $b = 0.5$ ($\varphi = e^{-0.25\,q}$), and

Multi-lane case (number of lanes > 2): $\Delta = 0.5$ s and $b = 0.8$ ($\varphi = e^{-0.4\,q}$).

An important parameter in the arrival headway distribution model is λ given by:

$$\lambda = \frac{\varphi q}{1 - \Delta q} \quad \textit{subject to } q \leq 0.98/\Delta \qquad (15a)$$

This parameter appears in most equations for the estimation of green times at VA signals.

The *simple negative exponential* and *shifted negative exponential* models of arrival headways and the Tanner (1962, 1967) model, which are more commonly used in the literature, can be obtained as special cases of the bunched exponential model:

for simple negative exponential model, use: $\Delta = 0$ and $\varphi = 1$ (therefore $\lambda = q$),

for shifted negative exponential model, use: $\varphi = 1$ and $\Delta = 1.5$, and

for the Tanner model, use $\varphi = 1 - \Delta q$ (therefore $\lambda = q$).

The shifted negative exponential model is suitable for single-lane traffic only. The shifted and simple negative exponential models assume no bunching ($\varphi = 1$) for all levels of flow which is not realistic.

Arrival headway distributions play a fundamental role in the estimation of green extension times at actuated signals. Significant differences were found in average green times predicted using different arrival headway distributions and different bunching levels (Akçelik 1994).

In the case of closely-spaced intersections, the assumption of random arrival headways is only partly applicable. The headways in platoons corresponding to saturation flows from upstream intersections are regular. A detailed platooned arrivals model would be needed for this purpose (see *Section 4.6*).

4. ESTIMATION OF GREEN TIMES

As discussed in *Section 2* (see *Figures 1a, 4 and 5*), the average displayed green time for a phase (G) is given by:

$$G = G_{ls} + G_{min} + G_r + min\ (G_e , G_{emax}) + G_{ec} \qquad (16)$$

where

G_{ls} = late start green time (seconds),
G_{min} = minimum green time (seconds),
G_r = average rest time (seconds),
G_e = average extension green time ($G_e \leq G_{emax}$) (seconds), and
G_{ec} = early cut-off green time.

The late start and early cut-off green times (G_{ls}, G_{ec}) are fixed as determined by the relevant control settings as discussed in *Section 2* (see *Figures 2a, 6a and 6b*). In most cases, the late start and early cut-off green intervals are not applicable ($G_{ls} = 0$, $G_{ec} = 0$).

The minimum green time with stop-line detection is fixed as determined by the minimum green setting ($G_{min} = G_{mf}$). The minimum green time with non-stopline detection ($G_{min} = G_{mf} + G_{iv}$ where G_{iv} is duration of the variable initial green interval) varies with the detector setback distance, the total arrival flow rate, critical lane saturation flow and start loss, and the red time. The formulae for the calculation of G_{min} in this case is given in *Section B.3 of Appendix B*.

As discussed in *Section 3*, the average duration of the rest interval (G_r) depends on the starting intergreen time and minimum green time for the subject phase and the total arrival flow rate considering all conflicting movements (*Equation 9*).

As seen from *Equation (16)*, the duration of the extension green interval cannot exceed the *maximum green extension setting* ($G_e \leq G_{emax}$). Before the test for maximum change is carried out, the value of the G_e based on a gap change or a waste change is determined:

$$G_e = \begin{cases} max\ [0, (l_s + g_s - G_{ls} - G_{min} - G_r + t_{eg/w})] & if\ w_{ts} < w_s \\ n_{ws}\ h_t & if\ w_{ts} \geq w_s \end{cases} \qquad (17)$$

$$t_{eg/w} = min\ (t_{eg}, t_{ew}) \qquad (17a)$$

where

l_s = start loss (seconds),
g_s = duration of the saturated part of the green period calculated from *Equation (11) or (11a)* (seconds),
$t_{eg/w}$ = average extension time by gap change or waste change *after queue clearance* (seconds),
t_{eg} = average extension time by gap change after queue clearance (seconds),
t_{ew} = average extension time by waste change before gap change *after queue clearance* (seconds),

w_{ts} = total waste time (sum of waste increments) during the saturated part of the green period (seconds),
w_s = waste setting (seconds),
n_{ws} = the number of vehicles with positive waste increments ($w_{is} > 0$) and causing a waste change before the end of the saturated part of green period, and
h_t = saturation headway for all lanes of the movement ($h_s = 1 / s_t$ where s_t is the total movement saturation flow) (seconds).

While the saturation headway for waste change purposes is for all lanes of the movement, the values of $l_s + g_s$ (time from the start of the phase green to the end of the queue clearance interval) is for the critical lane (longest for any lane).

For VA control by gap setting only, *Equation (17)* is equivalent to:

$$G_e = max\ [0, (l_s + g_s - G_{ls} - G_{min} - G_r + t_{eg})] \qquad (17b)$$

Equations (17) and (17b) give $G_e = 0$ when the queue has cleared and the gap timer has expired (see *Section 2*) before the end of the minimum and rest periods ($l_s + g_s + t_{eg} \leq G_{ls} + G_{min} + G_r$). In this case, $t_{eg/w} = t_{eg}$ is used in *Equation (17)* since waste method does not apply if the gap timer has expired before the end of the minimum and rest periods.

The formulae for the calculation of various parameters listed above as required for *Equation (17)* are described in the following sections.

4.1 Average Extension Time by Gap Change

Gap change during the saturated portion of green period

Before giving the formula for calculating the average extension time by gap change after queue clearance, it is necessary to discuss the condition of no gap change during queue clearance. A gap change during the saturated portion of the green period indicates inefficient operation (insufficient green to clear the queue). Therefore, the gap setting should be chosen to ensure that a gap change does not occur during the saturated portion of the green period.

During the saturated part of green period (g_s), i.e. while vehicles are departing from a queue, the headways are assumed to be equal $h_t = 1/s_t$, where s_t is the saturation flow rate (veh/s) for all lanes allowing for any lane underutilisation (see *Section B.2 in Appendix B*). As discussed in *Section B.5 in Appendix B*, as a general rule, it would be a good idea to select a gap setting (e_h) which is larger than the critical lane saturation headway ($h_s = 1/s_c$ where s_c is the critical lane saturation flow rate in veh/s). This would ensure for all conditions that a gap change will not occur during the saturated part of the green period because, for multi-lane cases, saturation headways are smaller than the single-lane saturation headway). Gap settings used in Australian VA control practice satisfy this general rule (see *Tables C.3 and C.4* in *Appendix C*).

With presence detection, gap setting is a space time value (e_s). In *Appendix B*, it is suggested that this can be related to the saturation headway of a single lane using the occupancy time during the unsaturated part of the green period

($e_s > e_h - t_{ou}$). However, this is relevant to the analysis of gap change after queue clearance. For the queue clearance period, the occupancy time during saturation (t_{os}) should be used. This is associated with saturation speeds which are lower than approach speeds relevant to the unsaturated part of green period (see *Appendix A*). At the same time, reduced saturation flows are associated with reduced saturation speeds. This is particularly important considering the possibility of *downstream queue interaction* which may cause severe reductions in saturation flow and saturation speed.

Thus, for a reduced saturation flow, the saturation headway increases, but the occupancy time also increases. The space time can be small under such circumstances (see *Figure A.11* in *Appendix A*). Similar considerations apply to opposed turns including cases of pedestrian interference (large saturation headways and small saturation space times). These relationships should be carefully considered when choosing the gap setting as a space time value.

The analyses presented in the rest of this paper assume that the *gap setting* is chosen to ensure that a gap change does not occur during queue clearance.

Gap change during the unsaturated part of green period

A gap change during the unsaturated portion of green, i.e. after queue clearance, corresponds to conditions when the vehicles in the arrival stream pass through the intersection without queueing. Under these conditions, the average extension time by gap change can be estimated from

$$t_{eg} = n_g h_g + e_t \tag{18}$$

where

n_g = average number of arrivals before a gap change after queue clearance (average number of consecutive arrivals with headways $h < e_h$ followed by an arrival with a headway $h \geq e_h$), given by:

$$n_g = \frac{e^{\lambda(e_h - \Delta)}}{\varphi} - 1 \tag{19}$$

h_g = average headway before a gap change after queue clearance (average headway corresponding to n_g), given by:

$$h_g = \frac{1}{n_g}\left[\frac{e^{\lambda(e_h - \Delta)}}{\varphi\, q_u} - e_h - \frac{1}{\lambda}\right] \tag{20}$$

e_h = gap time setting,

e_t = terminating time at gap change (equals the gap setting, $e_t = e_h$, or the passage time setting, $e_t = e_p$).

q_u = total departure flow rate during the unsaturated part of green period for all lane groups that actuate the phase under consideration (veh/s),

Δ = minimum arrival (intra-bunch) headway (seconds),

φ = proportion of unbunched (free) vehicles calculated from *Equation (15)*, and

λ = parameter in the arrival headway distribution (*Equation 15a*).

Parameters φ and λ are calculated using $q = q_u = q_{ag}$ (arrival rate during the green period; for uniform arrivals, $q_u = q_a$).

Figures 9 and 10 show the average number of vehicles (n_g) and average headway (h_g) before gap change from *Equations (19) and (20)* as a function of the flow rate (q_u) predicted for a single-lane case with gap settings of e_h = 3.0 s and 4.5 s.

When the terminating time at gap change is the headway corresponding to the gap setting ($e_t = e_h = e_s + t_{ou}$), *Equation (18)* becomes equivalent to:

$$t_{eg} = \frac{e^{\lambda(e_h - \Delta)}}{\varphi\, q_u} - \frac{1}{\lambda} \qquad (21)$$

This formula will give the same result as the formula by Lin (1982a,b) for single-lane cases using the shifted negative exponential model with Δ = 1 s recommended by Lin. For multi-lane cases, Lin recommends the use of Δ = 0 s, therefore his formula is equivalent to *Equation (21)* for the simple negative exponential model ($\Delta = 0$, $\varphi = 1.0$ and $\lambda = q_u$):

$$t_{eg} = \frac{e^{q_u e_h} - 1}{q_u} \qquad (22)$$

Figure 11 shows the average extension time (t_{eg}) by gap change from *Equation (21)* as a function of the flow rate (q_u) predicted for a single-lane case with gap settings of e_h = 3.0 s and 4.5 s (this figure corresponds to *Figures 9 and 10*). It is seen that the difference between extension times for gap settings of e_h = 3.0 and 4.5 s increases with increasing flows to substantial levels at very high flows.

With *non-stop-line detectors* ($L_d > 0$), it is necessary to reduce n_g by the number of vehicles which arrive early during the green period, cross over the detector and join the back of the queue downstream of the detection point. These vehicles are counted as part of the vehicles departing during queue clearance ($s_t\, g_s$ vehicles where s_t is the total saturation flow for all lanes in veh/s) as well as being counted as part of n_g (since the gap timer operates from the start of the green period, including the minimum green and rest periods). This occurs when $q_{ar}\, r < n_d / p_c$, where q_{ar} is the total arrival flow rate for all lanes during the red period (for uniform arrivals $q_{ar} = q_a$), r is the effective red time, n_d is the average number of vehicles between the detector and the stop line in the critical lane, and p_c is the proportion of total flow in the critical lane (see *Section B.2* in *Appendix B).* In this case, reduce n_g by ($s_t\, g_s - q_{ar}\, r$). In this process, ensure that ($s_t\, g_s - q_a\, r$) > 0 and $n_g' = n_g - (s_t\, g_s - q_a\, r) > 0$ for use in *Equation (18).*

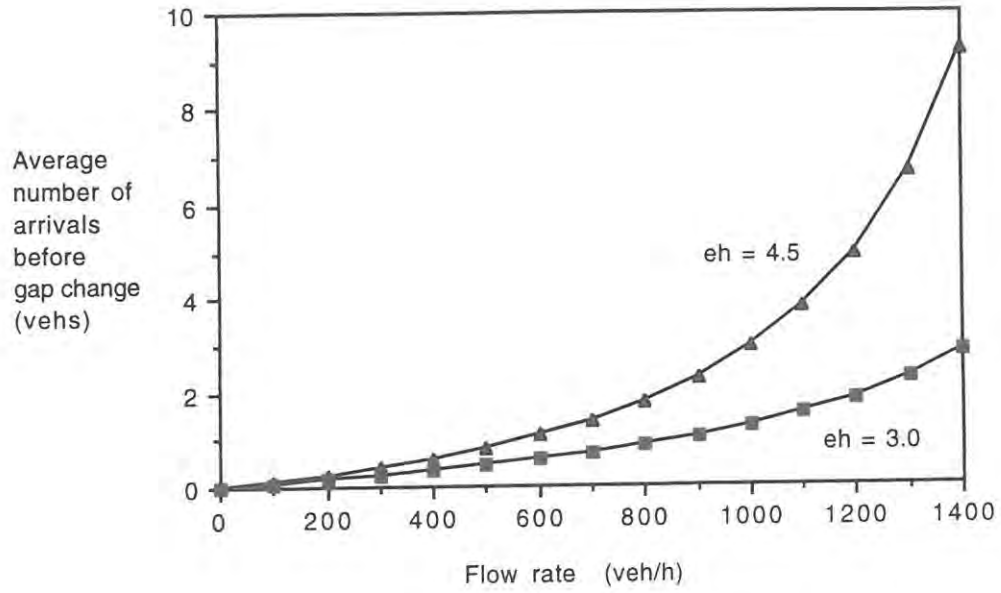


Fig. 9 – Average number of vehicles before gap change (n_g) as a function of the flow rate (q_w) predicted for a single-lane movement with gap settings of e_h = 3.0 s and 4.5 s

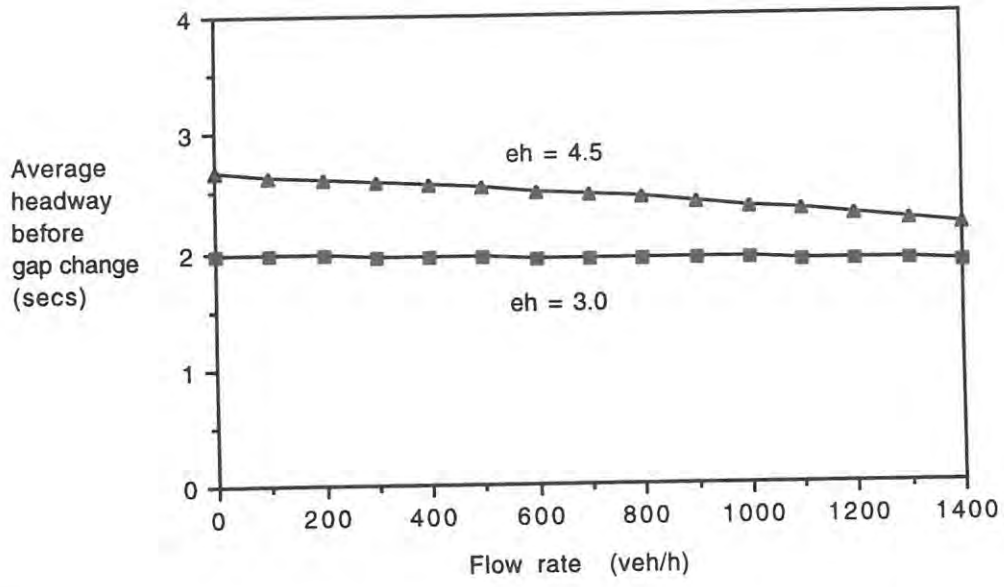


Fig. 10 – Average headway before gap change (h_g) as a function of the flow rate (q_w) predicted for a single-lane movement with gap settings of e_h = 3.0 s and 4.5 s

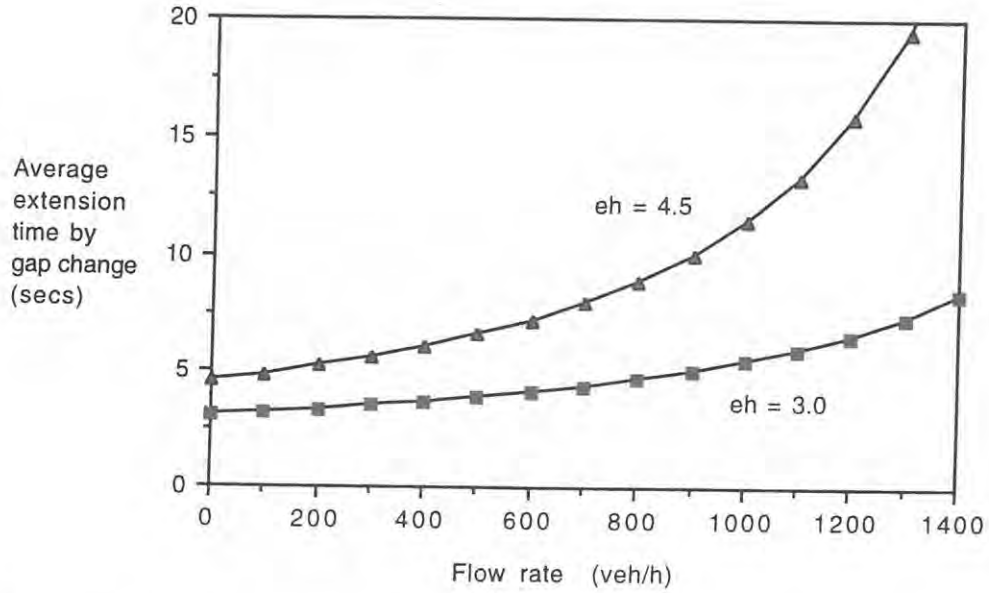


Fig. 11 – Average extension time by gap change after queue clearance (t_{eg}) as a function of the arrival flow rate (q_u) predicted for a single-lane movement with gap settings of e_h = 3.0 s and 4.5 s

Figures 12 and 13 show comparisons of the average green times estimated using *Equations (16) and (21),* and the resulting displayed cycle times, with those obtained from the microscopic MODELC simulation model for the same data base as that used for *Figure 8* (gap change without any maximum green time constraint).

Example

(i) For a single-lane movement with an average flow rate of q_u = 720 veh/h = 0.20 veh/s during the unsaturated part of the green period, and subject to VA control with a gap setting of e_h = 4.5 s, find the average number of arrivals and the average headway before a gap change, and the average extension time by a gap change after queue clearance using *Equation (21).*

From *Equations (15) and (15a),* $\varphi = \exp(-0.9 \times 0.20) = 0.8353$, $\varphi\, q_u = 0.8353 \times 0.20 = 0.1671$, $\lambda = 0.1671 / (1 - 1.5 \times 0.20) = 0.2387$ and $1/\lambda = 4.19$ s. From *Equations (19) and (20)*, the average number of arrivals before a gap change is $n_g = \exp(0.2387 (4.5 - 1.5)) / 0.8353) - 1 = 1.45$, and the average headway before a gap change is $h_g = (1/1.45)\ [(\exp(0.2387 (4.5 - 1.5)) / 0.1671) - 4.5 - 4.19] = 2.45$ s. The average extension time by a gap change after queue clearance from *Equation (21),* $t_{eg} = (\exp(0.2387 (4.5 - 1.5)) / 0.1671) - 4.19 = 8.1$ s. The same result is obtained from *Equation (18)* with $e_t = e_h = 4.5$ s: $t_{eg} = 1.45 \times 2.45 + 4.50 = 8.1$ s.

(ii) For the movement in (i), saturation flow is s = 1800 veh/h = 0.5 veh/s, effective red time is r = 30 s, start loss and end gain are l_s = 3 s, l_e = 2 s, minimum green time is G_{min} = 10 s, and the maximum extension green time is G_{emax} = 50 s. Find the queue clearance time (g_s), the extension green time (G_e), the displayed and effective green times (G, g) and the effective cycle time (c). Use f_q = 1.1 and assume uniform arrivals. The rest period is negligible. The late start and early cut-off intervals are not used.

The flow ratio is y = 720/1800 = 0.4 and the queue clearance time from *Equation (11a)* is $g_s = 1.1 \times 0.4 \times 30 / (1 - 0.4) = 22.0$ s. As found in (i), the extension time by gap change after queue clearance is t_{eg} = 8.1 s. From *Equation (17)*, the average extension green time is $G_e = 3.0 + 22.0 - 10.0 + 8.1 = 23.1$ s < G_{emax}. From *Equation (16)*, the displayed green time is G = 10.0 + 23.1 = 33.1 s (note that this is equal to $l_s + g_s + t_{eg} = 3.0 + 22.0 + 8.1 = 33.1$ s). The effective green time from *Equation (1)* is g = 33.1 – 3.0 + 2.0 = 32.1 s. The effective cycle time from *Equation (5)* is c = 30.0 + 32.1 = 62.1 s. The resulting degree of saturation is $x = (720 \times 62.1) / (1800 \times 32.1) = 0.774$.

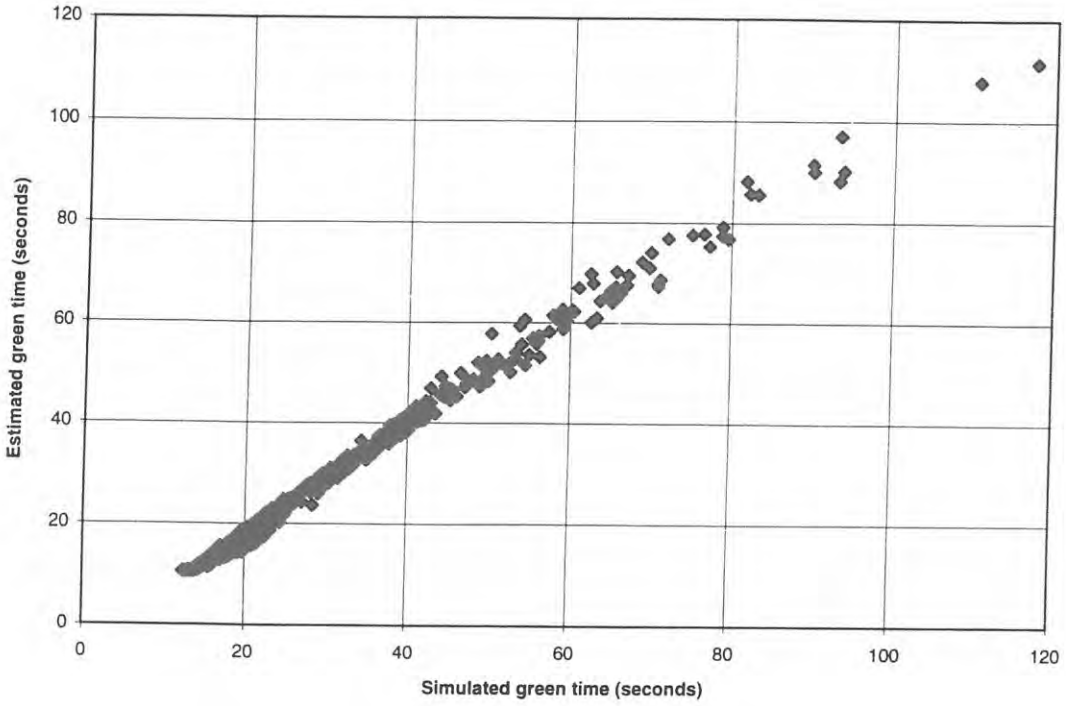


Fig. 12 – Comparison of estimated and simulated average green times by gap change without maximum green constraint

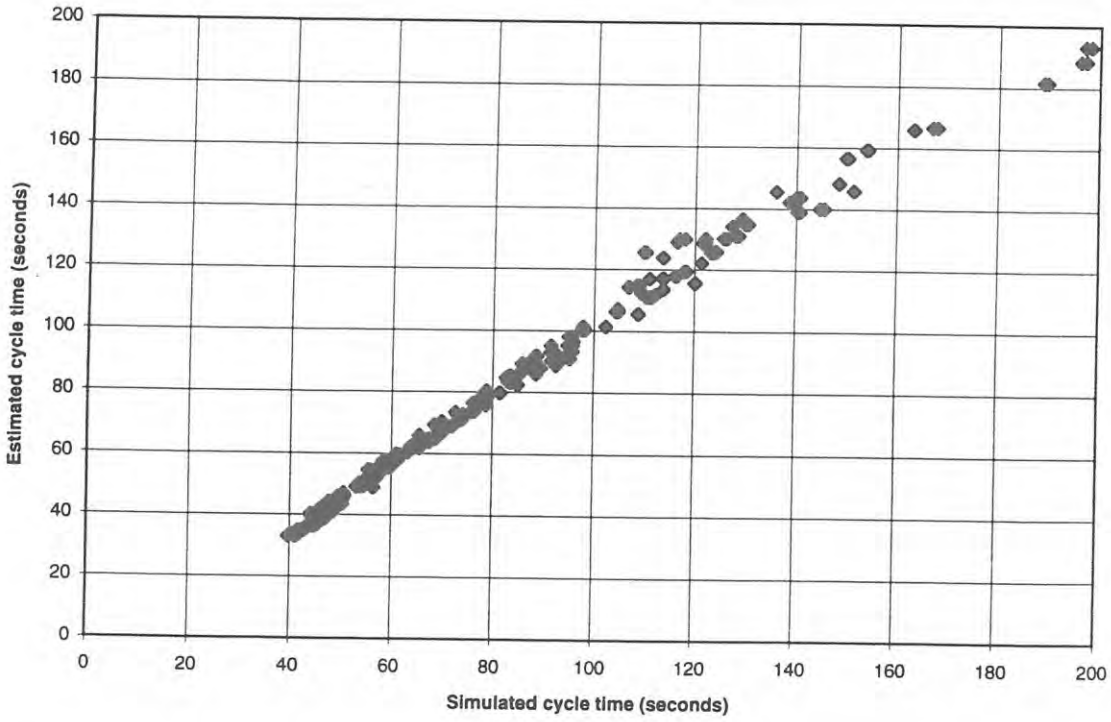


Fig. 13 – Comparison of estimated and simulated average displayed cycle times by gap change without maximum green constraint

4.2 Gap Reduction Methods

The choice of the *gap* setting is critical in determining the length of the green period, and hence the efficiency of operation. Vehicle-actuated control with a *fixed gap setting* (basic VA control) is not able to produce efficient operating conditions during both light and heavy flow periods. *Gap reduction* methods (often referred to as volume-density methods) are used for this reason.

Gap reduction methods help to reduce the *gap setting* automatically in response to measured traffic conditions, e.g. according to the traffic flow rate on the running phase (referred to as *density* control), waiting time on the red phase, the number of vehicles waiting on the red phase, or combinations and variations of these algorithms (e.g. *volume-density* and *headway-density* controllers). A summary of various gap reduction and other vehicle-actuated controller types can be found in Staunton (1976).

The formulae for estimating the average extension time after queue clearance (t_{eg}) given in *Section 4.1* can be used for gap-reducing controllers by replacing the fixed gap setting with a gap setting calculated according to the particular gap reduction technique used in the controller. Time-waiting and number waiting gap-reduction methods are discussed below.

Time-waiting gap reduction

The time-waiting gap-reduction method reduces the gap setting as a function of the time from the start of the current green period so that shorter gap settings are used for a gap change later during the green period. The vehicle-actuated signal controllers in the USA employ a linear gap reduction function (Lin and Courage 1995) which can be expressed as follows (see *Figure 14*) :

$$\begin{aligned} e &= e_{max} && \text{for } t \le e_{r1} \\ &= e_{max} - \frac{(e_{max} - e_{min})\,(t - e_{r1})}{e_{r2}} && \text{for } e_{r1} < t < e_{r1} + e_{r2} \\ &= e_{min} && \text{for } t \ge e_{r1} + e_{r2} \end{aligned} \qquad (23)$$

where

e = e_s if presence detection, e_g or e_h if passage detection,

e_{min} = minimum gap setting as a headway value for passage detection, or as a space time value for presence detection (seconds),

e_{max} = maximum gap setting (typically $e_{max} = e_p$ as a headway value for passage detection, or $e_{max} = e_p - t_{ou}$ as a space time value for presence detection) (seconds),

t = time since the start of the gap reduction period (for the case when the gap reduction starts at the start of the green period, use t = G where G is the displayed green time),

e_{r1} = time before reduction (seconds), and

e_{r2} = time to reduce (seconds).

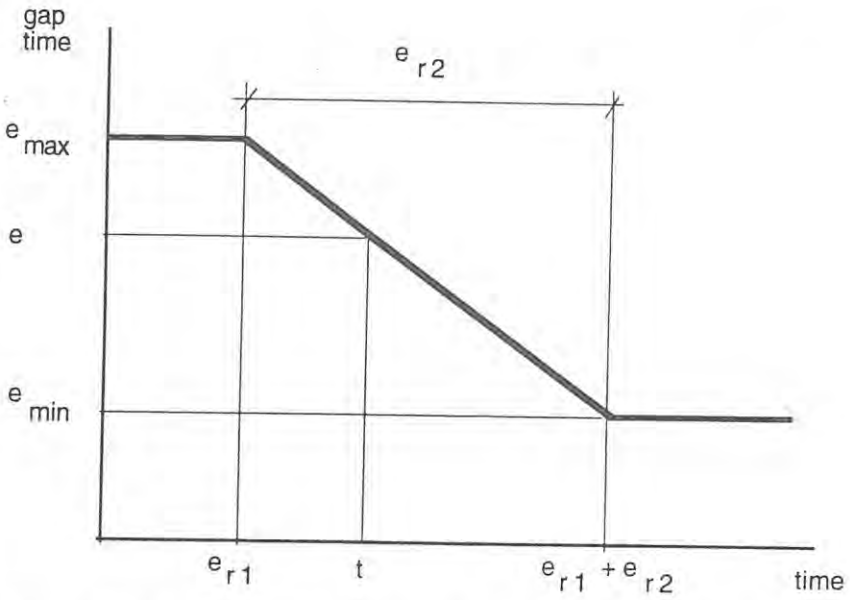


Fig. 14 – Time-waiting gap reduction method using a linear function

As seen in *Figure 14*, the gap setting remains at its maximum value, e_{max}, during the initial period, e_{r1}. It is then reduced from e_{max} to e_{min} during time to reduce, e_{r1}. If the controller settings e_{min} and e_{max} are specified as space time values (presence detection), *Equation (23)* gives $e = e_s$. For extension time estimation using *Equation (18) or (21)*, this needs to be converted to a headway value by adding the occupancy time, i.e. $e_h = e_s + t_{ou}$.

The minimum gap setting, e_{min}, should be greater than the critical lane saturation headway (or the corresponding space time) to ensure that no gap change occurs during queue clearance. For this purpose, use $e_{min} = 1.10\ h_s$ to $1.25\ h_s$ as a headway value, or $e_{min} = (1.10\ h_s$ to $1.25\ h_s) - t_{ou}$ as a headway value where $h_s = 1 / s_c$ (s_c is the critical lane saturation flow in veh/s).

An *exponential* time-waiting gap reduction function could be used instead of the linear function given in *Equation (23)*. To avoid repetition, an exponential function is used in *Equation (24)* for number-waiting gap reduction (similarly, a linear number-waiting gap reduction could be used).

Number-waiting gap reduction

This is similar to the time-waiting method except that the reduction in gap setting is related to the number of vehicles waiting on the red phase:

$$\begin{aligned} e &= e_{max}\, e^{-a\,(n_{vc}\,/\,n_r)} && \text{for } n_{vc} \le n_r \\ &= e_{min} && \text{for } n_{vc} > n_r \end{aligned} \qquad (24)$$

where

a = parameter calculated as $a = ln\ (e_{max} / e_{min})$,

n_{vc} = the number of vehicles (conflicting demand) waiting on the red phase, i.e. since the start of the current green period, and

n_r = the value of the number of vehicles waiting at the time when $e = e_{min}$.

The number of vehicles waiting on the red phase at the gap change time can be estimated from:

$$n_{vc} = q_{arc} G \qquad (24a)$$

where

q_{arc} = total conflicting demand flow (veh/s) arriving on the red phase, and

G = green time for the current phase (red time for the conflicting phases).

Example

Consider the single-lane movement in the example given in *Section 4.1* as subject to time-waiting gap reduction with the linear gap reduction method. The settings are e_{min} = 2.2 s and e_{max} = 4.5 s as headway values, e_{r1} = 0 s and e_{r2} = 30 s. The gap reduction period starts at the start of green period. Find e_h after 25 s after the start of the green period, and determine the displayed green time.

(i) Putting t = G = 20 s in *Equation (23)*, $e_h = 4.5 - (4.5 - 2.2) \times (25/30) = 2.6$ s is found.

(ii) The queue clearance time as found in the example in *Section 4.1* is g_s = 22.0 s. Let us use e_h = 2.6 s found in (i) as an initial guess. From *Equation (21)*, the extension time after queue clearance is $t_{eg} = (\exp(0.2387 (2.6 - 1.5)) / 0.1671) - 4.19 = 3.6$ s. From *Equation (17)*, the average extension green time is $G_e = 3.0 + 22.0 - 10.0 + 3.6 = 18.6$ s < G_{emax}. From *Equation (16)*, the displayed green time is G = 10.0 + 18.6 = 28.6 s.

Now recalculate $e_h = 4.5 - (4.5 - 2.2) \times (28.6/30) = 2.3$ s, and repeating the calculations, $t_{eg} = (\exp(0.2387 (2.3 - 1.5)) / 0.1671) - 4.19 = 3.1$ s, $G_e = 3.0 + 22.0 - 10.0 + 3.1 = 18.1$ s and G = 10.0 + 18.1 = 28.1 s are found. This can be accepted as the final solution.

4.3 Average Extension Time by Waste Change

The waste change method using the headway and waste settings (t_h, w_s) with presence detection as used in the Australian VA control practice is based on accumulating waste increments, i.e. the differences between the measured space time and the headway setting (as a space time value), $w_i = t_s - t_h$ (see *Section 2*, and *Section B.6* in *Appendix B*). The waste accumulation occurs during the extension green interval only, i.e. after the end of minimum green and rest periods, and the phase terminates when the total waste increment *before a gap change* equals the waste setting ($w_t = w_s$).

Waste accumulation can occur during both saturated and unsaturated parts of the green period. A waste change during the saturated part of the green period is relevant in view of the small values of the headway setting used in Australian practice (t_h = 0.3 to 2.0 s as seen in *Tables C.3 and C.4* in *Appendix C*). Even if a waste change does not occur, the total waste time accumulated during the saturated part of the green period must be estimated and taken into account in determining the total waste time accumulated during the unsaturated part of the green period.

Waste change during the saturated part of green period

The waste increment during the saturated part of green period, i.e. during queue clearance, w_{is}, can be calculated from:

$$w_{is} = max\,(0, t_{ss} - t_h) \qquad (25)$$

where t_h is the headway setting and t_{ss} is the space time during the saturated part of green period given by:

$$t_{ss} = h_t - t_{os} \qquad (25a)$$

where h_t is the saturation headway considering all lanes ($h_t = 1/s_t$), and t_{os} is the detector occupancy time during the saturated part of green period (see *Appendix A*). For a single-lane movement, $s_t = s_c$, $h_t = h_s = 1/s_c$.

The total waste accumulated during the saturated part of green period, w_{ts}, is given by:

$$w_{ts} = max\,[0, \frac{(l_s + g_s - G_{ls} - G_{min} - G_r)\, w_{is}}{h_t}] \qquad (26)$$

where l_s is the start loss, g_s is the duration of the saturated part of green period, G_{ls} is the late start time, G_{min} is the minimum green time, G_r is the average rest time, and w_{is} is the waste increment from *Equation (25)*.

In *Equation (25)*, w_{ts}= 0 if the queue is cleared before the end of the minimum green and rest periods ($l_s + g_s \le G_{ls} + G_{min} + G_r$).

A waste change would occur during queue clearance if $w_{ts} \ge w_s$ where w_s is the waste setting, and as seen in *Equation (17)*, the extension green time would equal:

$$G_e = n_{ws}\, h_t \qquad (27)$$

where n_{ws} is the number of vehicles with positive waste increments ($w_{is} > 0$) causing a waste change before the end of the saturated part of green period:

$$n_{ws} = \frac{w_s}{w_{is}} \qquad for\ w_{is} > 0 \tag{27a}$$

Waste change during the unsaturated part of green period

If a waste change does not occur during the saturated part of green period ($w_{ts} < w_s$), possibility of a waste change *before gap change* during the unsaturated part of green period needs to be checked. For this purpose, calculate the total waste time from

$$w_t = w_{ts} + w_{tu} \tag{28}$$

where w_{ts} is the total waste accumulated during queue clearance calculated from *Equation (26)*, and w_{tu} is the total waste accumulated before a gap change after queue clearance given by:

$$w_{tu} = n_g\, p_w\, w_{iu} \tag{29}$$

where

n_g is the average number of arrivals before a gap change after queue clearance from *Equation (19)*,

p_w is the proportion of headways that give positive waste increment amongst all headways before gap change; calculated as the ratio of the probability of a headway between the headway setting and gap setting ($h_o < h < e_h$, or $t_h < t_s < e_s$) to the probability of all headways less than the gap setting ($h < e_h$, or $t_s < e_s$):

$$p_w = max\left[\,0\,,\ \frac{\varphi\, e^{\lambda \Delta}\,(e^{-\lambda h_o} - e^{-\lambda e_h})}{1 - \varphi\, e^{-\lambda (e_h - \Delta)}}\,\right] \tag{30}$$

and w_{iu} is the waste increment during the unsaturated part of green period, i.e. after queue clearance:

$$w_{iu} = t_{sw} - t_h \tag{31}$$

where t_h is the headway setting and t_{sw} is the average space time corresponding to arrival headways between the headway setting and gap setting, $t_h < t_s < e_s$, or $h_o < h < e_h$.

In *Equation (30)*, $h_o = t_h + t_{ou}$ is the headway setting as a headway value, $e_h = e_s + t_{ou}$ is the gap setting as a headway value, $h = t_s + t_{ou}$ is the headway, and t_{ou} is the detector occupancy time during the unsaturated part of green period. The value of t_{sw} for use in *Equation (31)* can be calculated from:

$$t_{sw} = h_w - t_{ou} \tag{32}$$

where h_w is the average of headways between the headway setting and the gap setting:

$$h_w = max\left[\,0\,,\ \left(\frac{1}{\lambda} + \frac{h_o\, e^{-\lambda h_o} - e_h\, e^{-\lambda e_h}}{e^{-\lambda h_o} - e^{-\lambda e_h}}\right)\right] \tag{33}$$

If the total waste time from *Equation (28)* is less than the waste setting ($w_t < w_s$), a waste change before gap change will not occur. If $w_t \geq w_s$, calculate the average extension time by waste change before a gap change during the unsaturated portion of green from:

$$t_{ew} = n_{wu}\, h_g \tag{34}$$

where h_g is the average headway before a gap change after queue clearance calculated from *Equation (20)* and n_{wu} is the number of vehicles with positive waste increments ($w_{iu} > 0$) and causing a waste change after the end of the saturated part of green period and before a gap change:

$$n_{wu} = \frac{w_s - w_{ts}}{p_w\, w_{iu}} \qquad for\ p_w > 0\ and\ w_{is} > 0 \tag{35}$$

where w_s is the waste setting, w_{ts} is the total waste time accumulated during the saturated part of green period calculated from *Equation (26),* w_{iu} is the waste increment calculated from *Equation (31)* and p_w is the proportion of headways that give positive waste increment calculated from *Equation (30).*

For a waste change before gap change during the unsaturated part of green period, $t_{ew} < t_{eg}$ is needed, i.e. $t_{eg/w} = t_{ew}$ in *Equation (17).* As mentioned before, other conditions for this to occur are:

(i) $w_{ts} < w_s$: there must not be a waste change during the saturated part of green period, and

(ii) $l_s + g_s + t_{eg} > G_{ls} + G_{min} + G_r$: there must not be a gap change before the end of the minimum and rest periods.

Figure 15 shows a comparison of the average cycle times estimated using the analytical method described in this section with those obtained from the microscopic MODELC simulation model. The data is the same as in *Figure 8* except that a maximum extension green time of G_{emax} = 30 s, a headway setting of t_h = 1.8 s and a waste setting of w_s = 4.0 s are used in addition to the gap settings in the range e_s = 1.7 s to 5.5 s (e_h = 2.2 s to 6.0 s) and G_{min} = 10 s. Note that $w_s = 0.1\, G_{max}$ was used where $G_{max} = G_{min} + G_{emax}$ = 40 s.

Figure 16 shows a comparison of simulated average green times from MODELC for the same data set using gap control only (e_s = 1.7 s to 5.5 s) and using headway and waste settings (t_h = 1.8 s, w_s = 4.0 s) in addition to the gap settings (gap and waste control). G_{min} = 10 s and G_{emax} = 30 s were used in both cases. It is seen that the use of headway and waste settings made little difference in this example, i.e. phase durations were mostly determined by gap change rather than waste change. Further work on the impact of headway and waste settings is in progress.

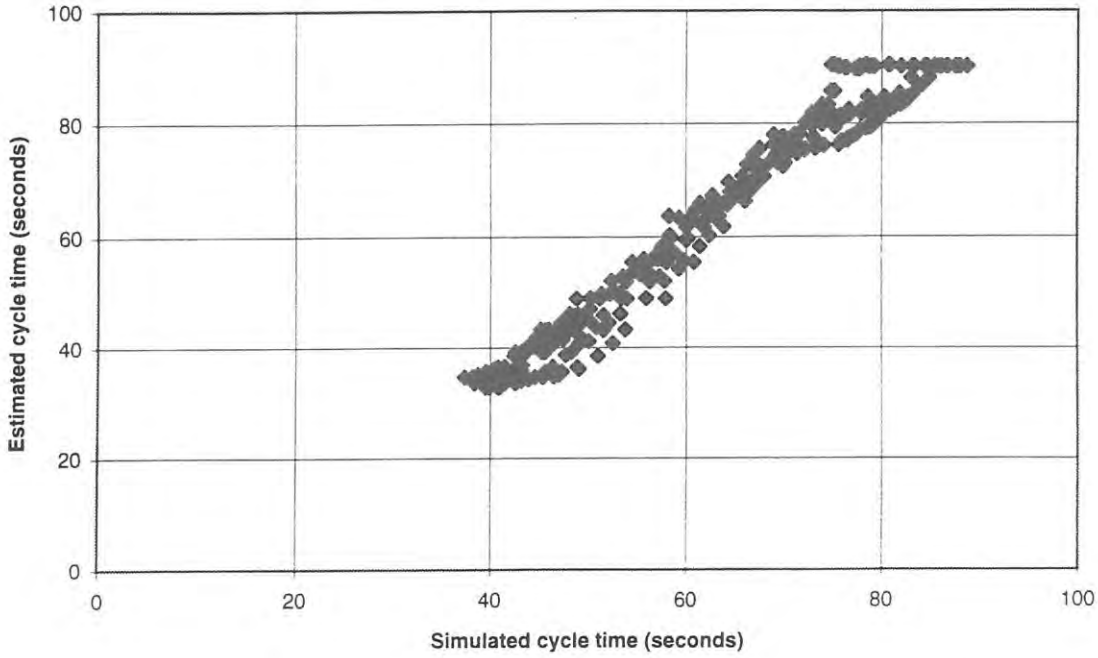


Fig. 15 – Comparison of estimated and simulated average cycle times obtained using headway and waste settings in addition to gap setting

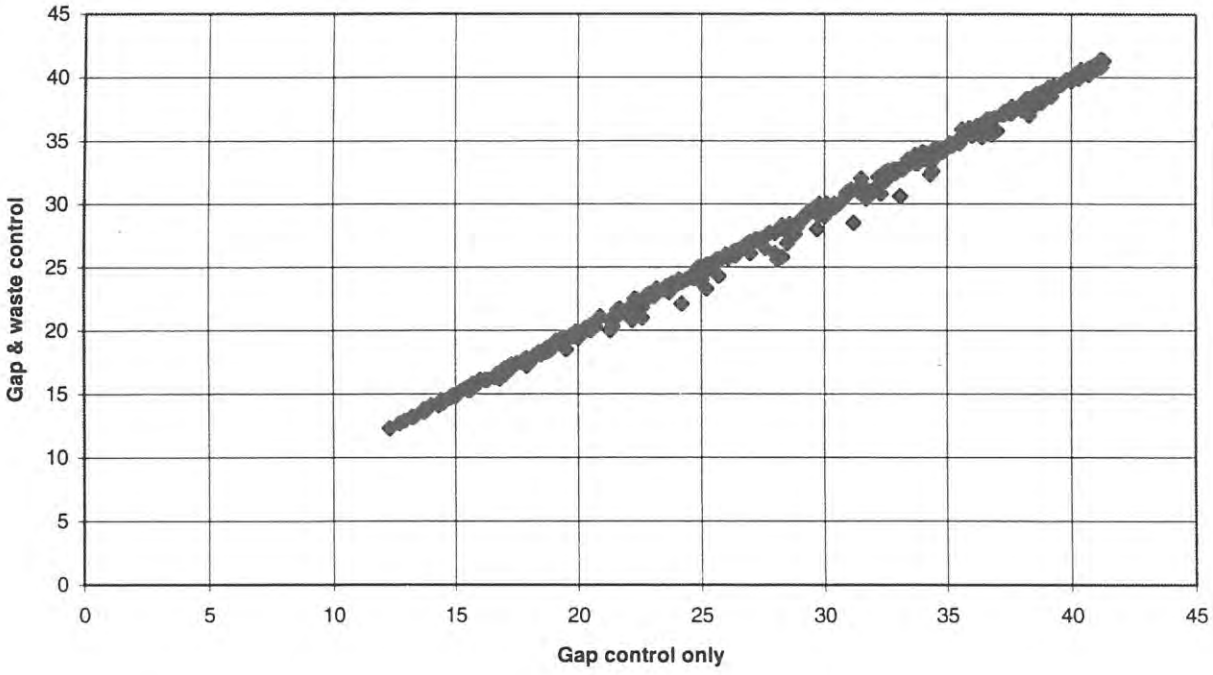


Fig. 16 – Comparison of simulated average green times using gap control only vs gap and waste control

Example

For the single-lane movement in the example given in *Section 4.1*, determine if a waste change before gap change would occur using headway and waste settings of (i) t_h = 0.8 s, w_s = 3.0 s, (ii) t_h = 1.8 s, w_s = 4.0 s, and (iii) t_h = 1.8 s, w_s = 0.5 s. Minimum green and maximum extension green times are G_{min} = 10 s and G_{emax} = 50 s. There is no late start or early cut-off interval. Ignore the rest period. The average occupancy times during the saturated and unsaturated parts of green period are t_{os} = 0.6 s and t_{ou} = 0.5 s. Using $q_s = s_t = s_c$ = 1800 veh/h = 0.5 veh/s and $q_u = q_a$ = 720 veh/h = 0.20 veh/s, the average departure headways during the saturated and unsaturated parts of green period are $h_t = h_s = 1 / s_c = 1/0.5$ = 2.0 s and $h_u = 1 / 0.20$ = 0.5 s. As determined in the example given in *Section 4.1*, g_s = 22.0 s, $l_s + g_s$ = 25.0 s, and t_{eg} = 8.1 s.

(i) Using t_h = 0.8 s and w_s = 3.0 s: From *Equation (25a)*, the average space time during the saturated part of green period is t_{ss} = 2.0 – 0.6 s = 1.4 s. From *Equation (25)*, the corresponding waste increment is w_{is} = 1.4 – 0.8 s = 0.6 s. From *Equation (26)*, the total waste during queue clearance is w_{ts} = ((25.0 – 10.0) x 0.6) / 2.0 = 4.5 s > w_s. From *Equation (27a)*, n_{ws} = 3.0 / 0.6 = 5.0 vehs, and from *Equation (27)*, G_e = 5.0 x 2.0 = 10.0. From *Equation (16)*, G = 10.0 + 10.0 = 20.0. The effective green and cycle times are g = 20.0 – 3.0 + 2.0 = 19.0 s and c = 30.0 + 19.0 = 49.0 s. The resulting degree of saturation is x = (720 x 49.0) / (1800 x 19.0) = 1.03 (oversaturated due to a waste change before queue is cleared).

(ii) Using t_h = 1.8 s and w_s = 4.0 s: Since t_{ss} = 1.4 < t_h, the waste increment during the saturated part of green period is w_{is} = 0, therefore there will not be any waste accumulation during queue clearance, w_{ts} = 0. Using e_h = 4.5 s, $h_o = t_h + t_{ou}$ = 1.8 + 0.5 = 2.3 s, λ = 0.2387, φ = 0.8353 and Δ = 1.5 s in *Equations (30) and (33)*, p_w = 0.477 and h_w = 3.3 s are found. From *Equation (32)*, t_{sw} = 3.3 – 0.5 s = 2.8 s, and from *Equation (31)*, w_{iu} = 2.8 – 1.8 = 1.0 s. The total waste time is $w_t = w_{tu}$ = 1.45 x 0.477 x 1.0 ≈ 0.7 s < w_s = 4.0 s, therefore a waste change will not occur before the gap change. From *Equation (35)*, n_{wu} = 4.0 /(0.477 x 1.0) = 8.4 vehs, and using h_g = 2.45 in *Equation (34)*, t_{ew} = 8.4 x 2.45 = 20.6 s > t_{eg} = 8.1 s, therefore $t_{eg/w} = t_{eg}$ = 8.1 s (gap change). The green time will be the same as in the example given in *Section 4.1*.

(iii) Using t_h = 1.8 s and w_s = 0.5 s: As found in (ii), w_t = 0.7 s, therefore $w_t > w_s$, i.e. a waste change will occur before gap change after queue clearance. From *Equation (35)*, n_{wu} = 0.5 /(0.477 x 1.0) = 1.05 vehs, and using h_g = 2.45 in *Equation (34)*, t_{ew} = 1.05 x 2.45 = 2.6 s < t_{eg} = 8.1 s, therefore $t_{eg/w} = t_{ew}$ = 2.6 s (waste change). From *Equation (17)*, the average extension green time is G_e = 25.0 – 10.0 + 2.6 = 17.6 s < G_{emax}. From *Equation (16)*, the displayed green time is G = 10.0 + 17.6 = 27.6 s. The effective green time from *Equation (1)* is g = 27.6 – 3.0 + 2.0 = 26.6 s. The effective cycle time from *Equation (5)* is c = 30.0 + 26.6 = 56.6 s. The resulting degree of saturation is x = (720 x 56.6) / (1800 x 26.6) = 0.851.

4.4 Pedestrian Considerations

Pedestrian crossings

A signalised pedestrian crossing operates as a simple two-phase system. The pedestrian phase is introduced by pedestrian actuation only. When introduced, this phase receives a pedestrian minimum time, G_{pmin}, which is the sum of minimum walk time and clearance 1 time (see *Equation B.23* in *Section B.7* of *Appendix B*).

A pedestrian crossing can be vehicle-actuated or not. If vehicle-actuated, the vehicle phase will have the minimum, rest and extension green intervals as in the general case (see *Figure 1a* in *Section 2*). If not, the vehicle phase will have the minimum and rest intervals, and will be terminated as soon as a pedestrian call is received. In both cases, there is no rest interval for the pedestrian phase since it is terminated as soon as the pedestrian clearance 1 time expires (a call for the vehicle phase is registered automatically when the pedestrian phase is introduced).

A single stage pedestrian crossing will be considered here treating pedestrian flows in two directions as one pedestrian movement. Vehicle streams in two directions of the main road will also be treated as one movement.

The general method for estimating average green times applies to pedestrian crossings. The simple negative exponential model is appropriate for pedestrian flows ($\Delta = 0$, $\varphi = 1$ and $\lambda = q_p$ where q_p is the total pedestrian arrival flow rate; see *Equations 15 and 15a* in *Section 3*).

Vehicle-actuated pedestrian crossing

A vehicle-actuated pedestrian crossing can be analysed using the general method by calculating the pedestrian phase green time as

$$G_p = G_{pmin} \tag{36}$$

and calculating the vehicle phase green time from *Equation (16)* with the average rest time estimated from

$$G_r = \frac{e^{-q_p (I_s + G_{min})}}{q_p} \tag{37}$$

where q_p is the total pedestrian flow in both directions (ped/s), I_s and G_{min} are the starting intergreen time and the minimum green time (seconds) for the vehicle movement.

The extension green time is estimated using *Equation (17)* and all subsequent equations given in *Sections 4.1 to 4.3* as relevant.

To determine the effective red time for the pedestrian movement, the red time reduction for lack of arrivals during the early part of the displayed red period is calculated from:

$$r_d = (G_{min} + \frac{1}{q_p}) e^{-q_p (I_s + G_{min})} \quad (38)$$

where G_{min} and I_s are the minimum green and starting intergreen times for the vehicle movement.

Pedestrian crossing with no vehicle actuation

In this case, the average green time for the vehicle movement is estimated without an extension green time:

$$G = G_{min} + G_r \quad (39)$$

where G_r is calculated from *Equation (37)*. It is seen that the queue clearance time does not effect the green time. Unless G_r is large (pedestrian flow is low), the performance of the vehicle movement may not be satisfactory under this method of control.

Example

The total pedestrian flow at a pedestrian crossing on a two-lane one-way street is q_P = 90 ped/h = 0.025 ped/s and the minimum pedestrian time is G_{pmin} = 20 s. The total vehicle arrival flow is q_t = 1350 veh//h = 0.375 veh/s, the critical lane flow and saturation flow are q_c = 900 veh//h = 0.250 veh/s and s_c = 1800 veh//h = 0.500 veh/s (y = 90 / 1800 = 0.50). The minimum green time for the vehicle phase is G_{min} = 10 s. Intergreen times are I = 5 s. The start loss and end gain times for the vehicle movement are l_s = 3 s and l_e = 2 s. The start loss and end gain times for the pedestrian movement are l_{ps} = 1 s and l_{pe} = – 11 s. Find the average phase green time and cycle time for two types of control: (i) vehicle phase is actuated (use a gap setting of e_h = 3.5 s, G_{emax} = 50 s, queue clearance factor of 1.10), and (ii) pedestrian actuation only.

(i) Pedestrian green time is always $G_p = G_{pmin}$ = 20 s. This means that the red time for the vehicle movement is fixed and R = 5 + 20 + 5 = 30.0 s (see *Equation 6a*). From *Equation (37)*, the average rest time for the vehicle phase is G_r = (1 / 0.025) x exp (– 0.025 x (5 + 10)) = 27.5 s.

From *Equation (11a)*, the queue clearance time is g_s = 1.1 x 0.50 x 30 / (1 – 0.50) = 33.0 s. The vehicle movement is in two lanes, therefore Δ = 0.5 s, φ = exp (– 0.25 x 0.375) = 0.9105, φ q = 0.9105 x 0.375 = 0.3414, λ = 0.3414 / (1 – 0.5 x 0.375) = 0.4202 and 1 / λ = 2.38 s (see *Equations 15 and 15a)*. From *Equation (21)*, the average extension time by gap change is t_{eg} = (exp (0.4202 (3.5 – 0.5)) / 0.3414) – 2.38 = 5.6 s. To find the average extension green time from *Equation (17)*, $l_s + g_s + t_{eg}$ = 3.0 + 33.0 + 5.6 = 41.6 s > $G_{min} + G_r$ = 10.0 + 27.5 = 37.5 s, therefore G_e = 41.6 – 37.5 = 4.1 s < G_{emax}. *From Equation (16)*, the average green time for the vehicle phase is G = 10.0 + 27.5 + 4.1 = 41.6 s.

From *Equation (4)*, the displayed cycle time is C = 41.6 + 5 + 20.0 + 5 = 71.6 s. The effective green and red times for the vehicle movement from *Equations (1) and (7)* are g = 41.6 – 3.0 + 2.0 = 40.6 s and r = 30.0 + 3.0 – 2.0 = 31.0 s (r_d = 0 since there are no rest intervals for the pedestrian movement). Therefore, the

effective cycle time from *Equation (5)* is c = 31.0 + 40.6 = 71.6 s = C, and the degree of saturation (for the critical lane) is x = (900 x 71.6) / (1800 x 40.6) = 0.882.

For the pedestrian movement, the effective green time is g_p = 20.0 – 1.0 – 11.0 = 8.0 s. The displayed red time from *Equation (6)* is $R_p = C - G_{pmin}$ = 71.6 – 20.0 = 51.6 s. The red time reduction for lack of arrivals during the early part of the displayed red period is calculated from *Equations (38)* as r_d = (10.0 + (1 / 0.025)) x exp (– 0.025 x (5 + 10)) = 34.4 s, and the effective red time from *Equation (7)* is r = 51.6 + 1.0 + 11.0 – 34.4 = 29.2 s. The effective cycle time from *Equation (5)* is c = 29.2 + 8.0 = 37.2 s. Note that $c + r_d$ = 37.2 + 34.4 = 71.6 s = C).

(ii) If the vehicle phase is not actuated, the displayed green time for the vehicle movement from *Equation (39)* is G = 10.0 + 27.5 = 37.5 s. Other statistics for the vehicle movement are C = 37.5 + 5 + 20.0 + 5 = 67.5 s, g = 37.5 – 3.0 + 2.0 = 36.5 s and r = 31.0 s, c = 67.5 s = C, x = (900 x 67.5) / (1800 x 36.5) = 0.925. As expected, this method is seen to be less efficient for the vehicle movement while it is slightly more efficient for the pedestrian movement (reduced red time).

Pedestrians at signalised intersections

Pedestrian movements at signalised intersections can have a significant effect on signal operations through their minimum time requirements. Pedestrian *walk* and *clearance times* are used to calculate the *pedestrian minimum time* (G_{pmin}) as discussed in *Section B.7 of Appendix B*. If pedestrian actuation (push button operation) is not allowed, then the minimum green times for the parallel vehicle movements should be long enough so that the analysis allows for pedestrian requirements for safe crossing. If pedestrian actuation is allowed, then pedestrian movements are included in the critical movement analysis using the associated required minimum time values.

For low pedestrian flows, the number of signal cycles with *no pedestrian arrivals* can be significant. This would result in a shorter average cycle time than the case when pedestrian actuation occurs in all cycles because pedestrian minimum time requirements are usually longer than vehicle minimum green times. This effect could be allowed by calculating the probability of no pedestrian arrivals:

$$p_o = e^{-q_p (C - t_{pw})} \qquad (40a)$$

where p_o is the probability of no arrivals during the period $C - t_{pw}$ (cycle time less pedestrian walk time) obtained from *Equation (10)* in *Section 3* with $\Delta = 0$, $\varphi = 1$ and $\lambda = q$.

If the green time calculated for the concurrent vehicle movement (G_v) is less then the pedestrian minimum time (e.g. $G_v < G_{pmin}$), then the phase green time (e.g. G_A) would be estimated from

$$G_A = p_o G_v + (1 - p_o) G_{pmin} \qquad (40b)$$

If $G_v \geq G_{pmin}$, then $G_A = G_v$.

This method requires iterative calculations since p_o, C and G in the above equations are interdependent.

The same method can be used when a pedestrian movement runs during more than one cycle and overlaps with several vehicle movements. For example, considering vehicle movements 1 and 2 that run in phases A and B, respectively, and a pedestrian movement that runs in phases A through B, the sum of the two phase green times and the intergreen between them is given by $G_A + I_{A,B} + G_B = p_o (G_{v1} + I_{A,B} + G_{v2}) + (1 - p_o) G_{pmin}$.

Furthermore, the rest times should be estimated considering the cycles with pedestrians and without pedestrians, an average of these should be used in the analysis. In simple manual calculations, this complication could be avoided by neglecting pedestrian flows in rest interval calculations.

4.5 Semi-Actuated Signals

The semi-actuated signal operation is common in the USA, but rarely used in Australia. Considering a simple two-phase system controlling a major-minor road intersection, the operation of semi-actuated signals is similar to signalised pedestrian crossings with pedestrian actuation only (see *Section 4.4)*. For the sake of discussion, the minor road will be referred to as the *side road*, and the major road will be referred to as the *main road*.

The side road vehicles are detected and controlled as in the case of full VA control as discussed in previous sections. On the other hand, the main road has no detections, hence no extension interval. It consists of the minimum green and rest intervals, i.e. the main road receives a minimum green time and is terminated as soon as a conflicting demand is registered on the side road. Therefore, this type of operation is suitable only when the side road flows are very low.

As in *Equation (39)*, the average green time for the *main road* phase is estimated from:

$$G = G_{min} + G_r \qquad (41)$$

where the rest time G_r is calculated from *Equation (9)* with the side road movement treated as the conflicting flow. It is seen that, unless G_r is large (side-road flow is low), the performance of the vehicle movement may not be satisfactory under this method of control.

The average green time for the side road can be estimated using the general method except that the rest time for the side road $G_r = 0$ since a call for the main road phase is registered automatically when the side road phase is introduced.

Example

At an intersection controlled by a simple two-phase semi-actuated signal system, the main road has $G_{min} = 20$ s. Intergreen times are I = 5 s, all start loss and end gain times are $l_s = l_e = 3.0$ s. The side-road has a single-lane movement with an arrival flow rate of 360 veh/h = 0.10 veh/s, saturation flow rate of $s_c = 1800$ veh//h = 0.50 veh/s (y = 360 / 1800 = 0.20), and is subject to basic VA control with a gap setting of $e_h = 3.0$ s, $G_{min} = 10$ s, and $G_{emax} = 50$ s. Use queue clearance factor $f_q = 1.10$.

For the side road, $\Delta = 1.5$ s, $\varphi = \exp(-0.9 \times 0.10) = 0.914$, $\varphi q = 0.0914$, $\lambda = 0.0914 / (1 - 1.5 \times 0.10) = 0.1075$ and $1 / \lambda = 9.3$ s (see *Equations 15 and 15a)*. From *Equation (21)*, the average extension time by gap change is $t_{eg} = (\exp(0.1075 (3.0 - 1.5)) / 0.0914) - 9.3 = 3.6$ s.

From *Equation (9)*, the rest time for the main road is $G_r = 9.3 \times 0.914 \times \exp(-0.1075 (5 + 20 - 1.5) = 0.7$ s. From *Equation (41)*, the average green time for the main road is $G = 20 + 0.7$ s $= 20.7$ s.

The displayed red time for the side road is $R = 20.7 + 5 + 5 = 30.7$ s. The red time reduction for lack of arrivals during the early part of the displayed red period is calculated from *Equation (8)* as $r_d = (20.0 + 9.3) \times 0.914 \times \exp(-0.1075 \times (5 + 20 - 1.5)) = 2.1$ s. The effective red time from *Equation (7)* is $r = 30.7 + 3.0 - 3.0 - 2.1 = 28.6$ s. From *Equation (11a)*, the queue clearance time is $g_s = 1.1 \times 0.20 \times 28.6 / (1 - 0.20) = 7.9$ s. From *Equation (17)*, $l_s + g_s + t_{eg} = 3.0 + 7.9 + 3.6 = 14.5$ s $> G_{min} + G_r = 10.0 + 0 = 10.0$ s, therefore $G_e = 4.5$ s $< G_{emax}$. *From Equation (16)*, the average green time for the side road is $G = 10.0 + 0 + 4.5 = 14.5$ s.

From *Equation (4)*, the displayed cycle time is $C = 14.5 + 5 + 20.7 + 5 = 45.2$ s. The effective green and red times for the main road movement from *Equations (1) and (7)* are $g = G = 20.7$ s and $r = R = 45.2 - 20.7 = 24.5$ s. For the side road, $g = G = 14.5$ s, $r = 28.6$ as found above, the effective cycle time from *Equation (5)* is $c = 28.6 + 14.5 = 43.1$ s, and the degree of saturation is $x = (360 \times 43.1) / (1800 \times 14.5) = 0.595$.

4.6 Linked Vehicle-Actuated Signals

Although the method for the analysis of VA signal operations described in previous sections is primarily applicable to *isolated* intersections, many aspects of the method also apply to *linked VA* operations, i.e. to the operation of actuated signals as part of a coordinated system. However, there are some important issues which need to be taken into consideration. Some of these will be discussed briefly.

For linked signals, the *platooned* arrivals model need to be used instead of the uniform arrivals model. A simple model of platooned arrivals has been used in this paper, applying separate arrival flow rates to the green and red periods as discussed in *Section 3* (see *Equation 12*). For example, in a well coordinated system, a higher proportion of traffic will arrive during green. While this high flow rate is relevant to the analysis of extension times, the lower arrival flow rate during the red period is relevant to the determination of rest times.

A more direct modelling of platooned arrivals could be used when detailed arrival profiles are available, e.g. using a stepwise model as in the well-known TRANSYT program. Such models should identify the platoon and non-platoon parts of the arrival profile (as determined by upstream departure profiles). The arrival headways within platoons could be used as constant headways, but they are not necessarily the same as the minimum arrival headway (Δ) used in various equations. The headway distribution models based on the assumption of *random* arrivals described in *Section 3* could then be applied to the non-platoon part of the arrival profile.

Importantly, linked VA systems work under a master cycle time with certain amounts of green time pre-allocated to some phases to guarantee signal offsets for uninterrupted progression of main platoons. Methods given in previous sections need to be extended to deal with this type of operation. One particular extension of the method is the allocation of any *excess green time* to nominated movements (phases). This excess green time would result from the imposition of a master cycle which may be longer than the cycle time under isolated VA operation. The *green split priority* used in the SIDRA program is suitable for this purpose Akçelik (1990).

The semi-actuated signals and pedestrian-actuated crossings provide simple cases where all excess green time is allocated to the main road. For example, at a semi-actuated signal site, the main road green time, G_M , would be given by:

$$G_M = G_{min} + G_r + G_x \quad (42)$$

and the master cycle time would equal

$$C = G_M + G_S + \Sigma I \quad (43)$$

where G_S is the side-road green time calculated taking into account the increased red times due to the excess green time allocated to the main road, and ΣI is the sum of intergreen times.

This method assumes that a positive excess green value can be achieved. Otherwise, the master cycle time is insufficient for the operation of this site, but this could be accommodated by adjusting the side street green time down, e.g. by setting a lower maximum green time.

Similarly, all excess green time at a pedestrian-actuated crossing can be allocated to the main road, but a negative excess time is not acceptable since the pedestrian movement operates at minimum green time.

4.7 Signal Timings for Fixed-Time Signals

For the purpose of discussing the implications of vehicle actuated signal settings in terms of resulting degrees of saturation, the analysis method for fixed-time signal timings is summarised here (Akçelik 1981, 1990). This method tries to achieve specified target (practical) degrees of saturation (x_p) for critical movements. Usually, the principle of equal degrees of saturation (the same x_p value for all movements) is used. A more general method allows for different x_p values to be specified for different movements (e.g. 0.90 for the main road and 0.95 for the side road).

The practical cycle and green time method can be expressed in terms of effective green and red times:

$$g_i = u_{pi} c_p \quad (44)$$
$$\textit{subject to } g_{imin} \leq g_i \leq g_{imax}$$

$$c_p = \frac{L + G_m}{1 - U_p'} \quad \textit{subject to } U_p' < 1.0 \tag{45}$$

where

c_p = the practical cycle time;

G_m = the sum of effective minimum and maximum green times (g_{imin}, g_{imax}) for critical movements whose green times are set to $g_i = g_{imin}$ or $g_i = g_{imax}$ (therefore not included in the summation for U_p'):

$$G_m = \Sigma (g_{imin} + g_{imax}) \tag{46}$$

U_p' = an adjusted *green time ratio* for the intersection (sum of the *required* green time ratios for critical movements *excluding* those with green times set to g_{imin} or g_{imax}):

$$U_p' = \Sigma u_{pi} \tag{47}$$

where $u_{pi} = y_i/x_{pi}$ is the green time ratio for *ith* critical movement (y_i = flow ratio, x_{pi} = practical degree of saturation).

Examples given in this paper and other analyses of actuated signal timings indicate that equal degrees of saturation do not necessarily result in actuated signal cases, and the minor movements may get lower degrees of saturation than the major movements, indicating inefficient operation.

Example

A simple two-phase case is considered with a single movement in each phase. This occurs at the intersection of two one-way streets with three lanes each. Equal conditions are assumed for both approaches. The critical lane saturation flows are s_c = 1800 veh/h, and the total approach saturation flows are s_t = 4500 veh/h (proportion of flow in the critical lane is p_c = 1800 / 4500 = 0.40). Intergreen times are I = 5 s, all start loss and end gain times are $l_s = l_e$ = 3.0 s, minimum and maximum green times are G_{min} = 10 s and G_{emax} = 45 s, gap setting is e_h = 4.0 (e_s = 3.5 s), headway setting is t_h = 0.6 s and waste setting is w_s = 3.0 s. The practical degree of saturation for fixed-time settings is x_p = 0.90. Therefore, the intersection lost time is L = 2 x 5 = 10 s, the minimum cycle time is C_{min} = 2 x (G_{min} + I) = 2 x (10 + 5) = 30 s, and the maximum cycle time is C_{max} = 2 x (G_{min} + G_{emax} + I) = 2 x 60 = 120 s. Total intersection flow (twice the approach flow) varies between 400 veh/h and 4000 veh/h. With all control methods in this example, the green times for the two phases are equal.

Figure 17 shows the displayed cycle time (C) as a function of the total intersection flow for (i) vehicle-actuated control with gap setting only, (ii) vehicle-actuated control with gap, headway and waste settings, and (iii) the practical cycle time with x_p = 0.90.

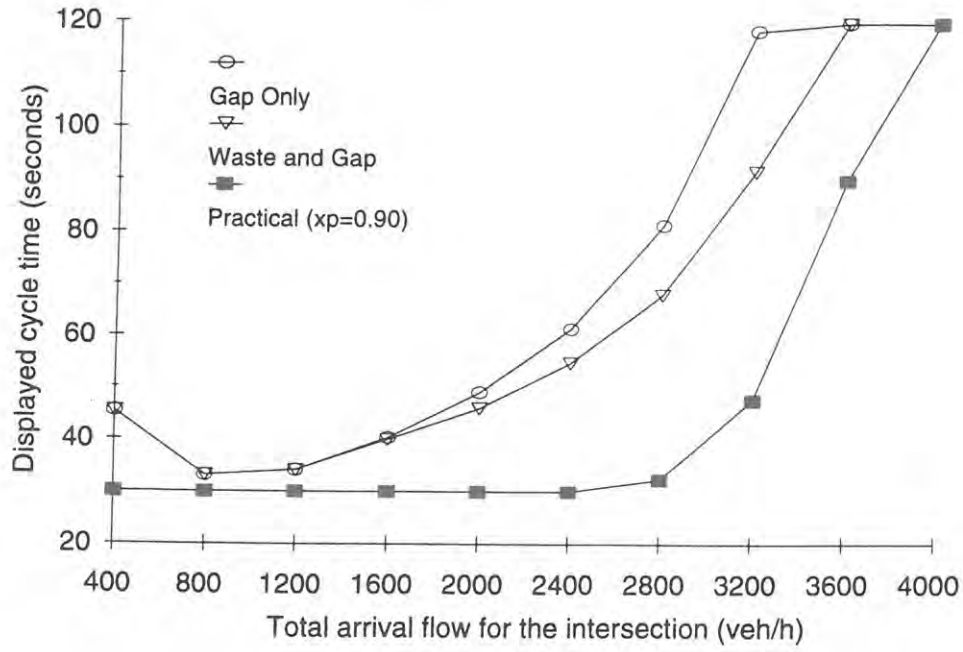


Fig. 17 – An example of the displayed cycle time (C) as a function of the total intersection flow for vehicle-actuated control with gap setting only (e_h = 4.0 s), with gap and waste settings (e_h = 4.0 s, t_h = 0.6 s, w_s = 3.0 s) and practical cycle time (x_p = 0.90)

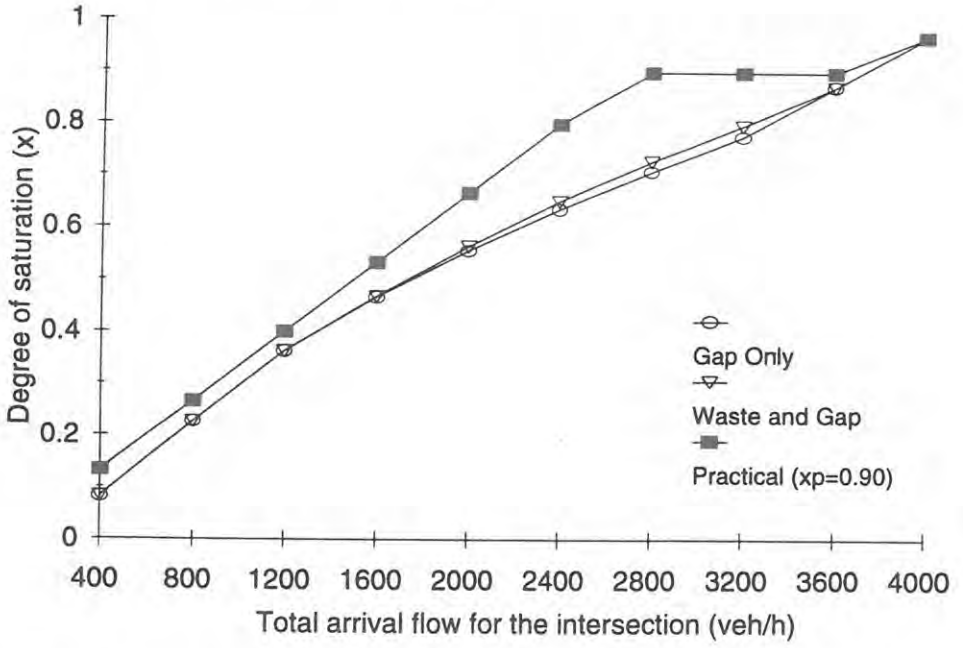


Fig. 18 – An example of the intersection degree of saturation (x) as a function of the total intersection flow for vehicle-actuated control with gap setting only (e_h = 4.0 s), with gap and waste settings (e_h = 4.0 s, t_h = 0.6 s, w_s = 3.0 s) and practical cycle time (x_p = 0.90)

It is seen in *Figure 17* that, under VA control, the cycle times for low flow conditions exceed the minimum cycle time significantly due to the effect of rest intervals. Under high flow conditions, all curves merge at maximum cycle time of 120 s. The intersection degrees of saturation corresponding to *Figure 17* are shown in *Figure 18*. It is seen that the degrees of saturation from the two methods of VA control are very close although the cycle times differ significantly.

5. CONCLUDING REMARKS

The method given in this paper provides an analytical tool for investigating the optimisation of VA signal operations (to minimise delay, queue length, number of stops, or a performance index). This can be done with relative ease compared with the use of a simulation model which is the most common method used for this purpose to date.

The analysis method presented in this paper can be implemented manually. However, implementation through computer software such as SIDRA (Akçelik 1990) is useful for dealing with complex intersection geometry and phasing arrangements, and importantly, because an iterative method is needed.

The reasons for an iterative method include (a) the queue clearance time depends on red time, which is a function of the green times of conflicting movements, (b) the cycle capacity in the calculation of the queue clearance factor for the critical lane depends on the green time, (c) the probability of no arrivals in a cycle and the cycle and green times are interdependent, (d) minimum green time with non-stop line detection changes with red time, (e) in the case of a controller with time-reducing gap-reduction method, the gap setting depends on green time, (f) critical lane flow can change as the saturation flows for individual lanes change with green time and cycle time due to such factors as opposed turns, lane blockages and short lanes and (g) critical movement combinations may change as the cycle time changes.

This indicates a strong need for iterative calculations. However, an iterative method is also needed for the analysis of fixed-time signal operations, and such a method is already implemented in SIDRA.

Extensive analysis of vehicle-actuated signal operations is needed considering various combinations of arrival flow and control conditions, to evaluate effectiveness of VA control settings in terms of delay, queue length, and other performance measures. There are many VA control parameters that can be considered for optimisation, namely minimum green, gap, maximum extension, headway and waste settings, the location and type of detectors and so on.

Some suggestions are available on the effect of gap setting in a basic VA controller based on a limited amount of research published in the literature (Grace, Morris and Pak Poy 1964; Webster and Cobbe 1966; Morris and Pak-Poy 1967; Pak-Poy and Associates 1975; Staunton 1976). These can be summarised as follows:

(a) There is an optimum value of the gap setting which minimises delay.

(b) The optimum gap setting becomes smaller and its effect becomes more critical as traffic flows increase; hence, for relatively high flows, it is important that the gap time setting chosen is not too large.

(c) The effect of increasing the gap setting is to increase the cycle time and to reduce the percentage of vehicles stopped; using large values of gap time setting is advantageous from this viewpoint especially for low flows where delays are not very sensitive to the value of the vehicle interval.

Analyses can be undertaken using the method given in this paper to assess the above findings, and to explore optimisation of VA controller settings, considering a wider range of impacts including queue length, fuel consumption and operating cost. Complex intersection geometries and flow conditions (including different flow levels at different times during the day) can be analysed for this purpose.

Preliminary analyses of the waste-headway control method indicate that, with near optimal gap settings, a gap change is more likely to occur than a waste change. When the gap setting is large, the headway and waste settings appear to be effective in reducing the cycle time at high flow conditions provided they are reasonably low. Further work is needed to investigate optimum combinations of the gap, headway and waste settings.

Work is in progress on calibrating the performance functions (delay, queue length, queue move-up rate, etc.) for actuated signals. The results will be useful in investigating optimum settings for VA control using the analytical methods given in this paper.

The issue of the effective detection zone length in relation to the physical length of presence loop needs to be resolved as discussed in *Appendix A*.

A major US research project (NCHRP 3-48) on vehicle actuated signals is also in progress (Courage and Akçelik 1994). Under this project, it has been established that the analytical method given in this paper produces satisfactory estimates of green times and cycle times compared with the results of the NETSIM and EVIPAS simulation models for basic VA control with a fixed gap setting (Courage, et al. 1995).

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APPENDICES

APPENDIX A

Fundamental Relationships

A.1 Basic Parameters

Basic parameters *headway*, *occupancy time*, *space time* and *gap time* measured using presence and passage detection are shown in *Figures A.1 to A.3*. *Figure A.1* has been developed from Figure 7.1 of AUSTROADS (1993).

Headway (h) is the time between passage of the front ends of two successive vehicles. *Spacing* (L_h) is the distance corresponding to the headway time, i.e. the distance between the front ends of two successive vehicles. *Vehicle passage time* (t_v) is the time between the passage of the front and back ends of a vehicle from a given point along the road, e.g. at a passage detector, or at the leading edge of a presence detection zone. *Gap time* (t_g) is the time between the passage

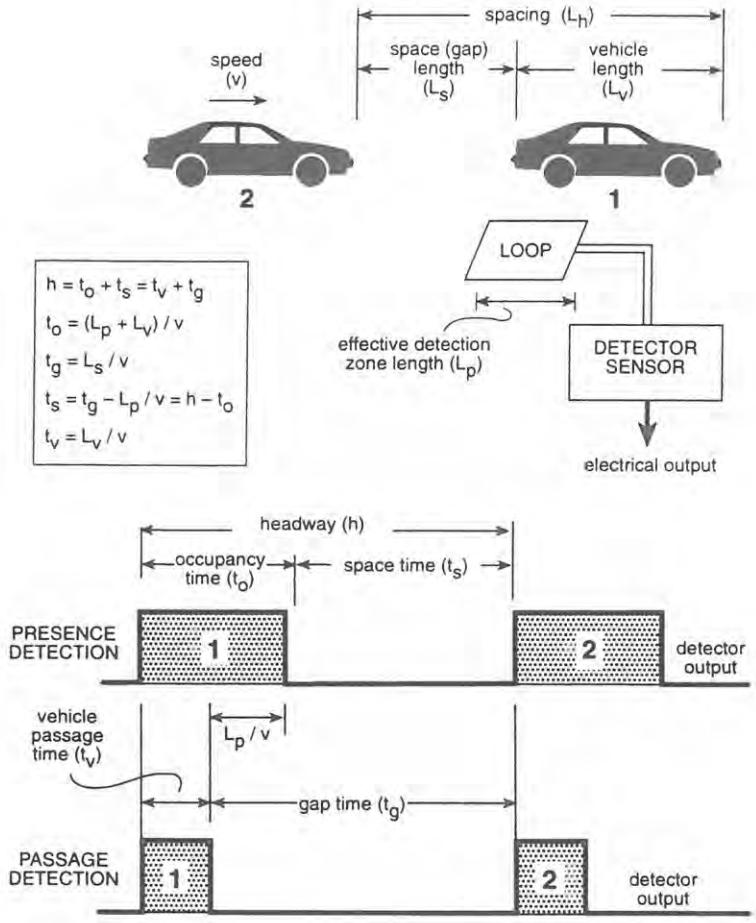


Fig. A.1 – Parameters and basic relationships in presence and passage detection for vehicle-actuated signal control

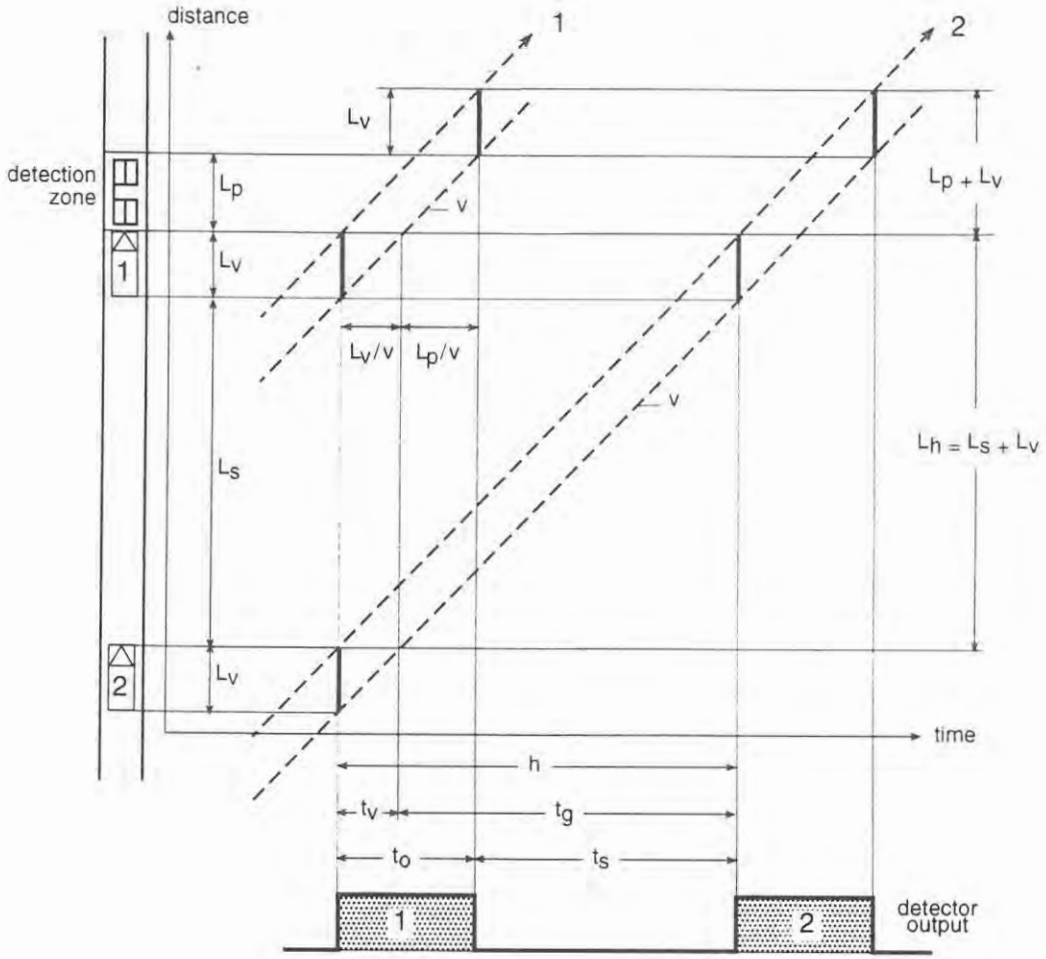


Fig. A.2 – Time-distance diagram explaining the parameters and basic relationships in presence and passage detection for vehicle-actuated signal control operation

of the back end of one vehicle and the front end of the next vehicle, measured at a given point along the road, and is equivalent to headway time less vehicle passage time. The space (gap) length, L_s, is the following distance between two successive vehicles as measured between the *back* end of one vehicle and the *front* end of the next vehicle (spacing less vehicle length).

Space time (t_s) is the duration of the time between the detection of two consecutive vehicles when the presence detection zone is not occupied. It is equivalent to gap time less the time taken to travel the effective detection zone length, L_p. *Occupancy time* (t_o) starts when the front of a vehicle enters the detection zone and finishes when the back of the vehicle exits the detection zone. Thus, it is the duration of the period when the detection zone is *occupied* by a vehicle, and is equivalent to the sum of the vehicle passage time and the time to travel the effective detection zone length.

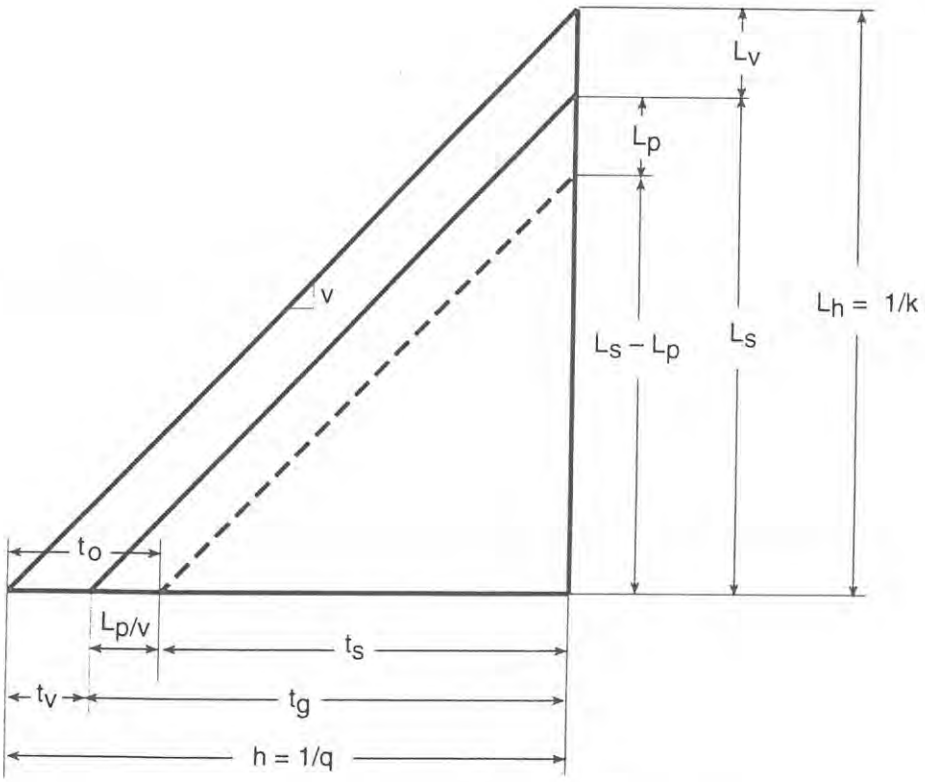


Fig. A.3 – The relationships between spacing, headway, space length, gap time, space time and speed

The relationships between headway, gap time, space time, occupancy time, spacing, space (gap) length and speed can be expressed as follows:

$$h = \frac{L_h}{v} = \frac{L_v + L_s}{v} = t_v + t_g = t_o + t_s \quad \text{(A.1)}$$

$$t_v = \frac{L_v}{v} \quad \text{(A.2)}$$

$$t_g = h - t_v = h - \frac{L_v}{v} = \frac{L_s}{v} \quad \text{(A.3)}$$

$$t_s = h - t_o = t_g - \frac{L_p}{v} = \frac{L_s - L_p}{v} \quad \text{(A.4)}$$

$$t_o = h - t_s = t_v + L_p / v = \frac{L_v + L_p}{v} \quad \text{(A.5)}$$

where

h = headway (seconds),
t_v = vehicle passage time (seconds),
t_g = gap time (seconds),
t_s = space time,

t_o = occupancy time (seconds),
L_v = vehicle length (m /veh),
L_h = spacing (headway distance), $L_h = L_v + L_s$, (m),
L_s = space (gap) length, $L_s = L_h - L_v$, (m),
L_p = detection zone length (m), and
v = vehicle speed (m/s).

In the calculations relating to average traffic conditions, the vehicle length should represent the actual traffic composition. Where the traffic stream is represented as a mixture of light vehicles (LVs) and heavy vehicles (LVs), the average vehicle length can be calculated as:

$$L_v = (1 - p_{HV})\, L_{vm} + p_{HV}\, L_{vHV} \qquad (A.6)$$

where

p_{HV} = proportion of heavy vehicles in the traffic stream,
L_{vm} = average vehicle length for light vehicles (passenger car units, *pcus*) (m/LV),
L_{vHV} = average vehicle length for heavy vehicles (m/HV),

Typical stop-line presence loop length used in Australia is 4.5 m. The effective detection zone length for presence detection is not necessarily the same as the detector loop length. Morris, Dean and Hulscher (1984) suggest that due to the *spill-over sensitivity* at the ends of the loop, the effective length of a loop is usually greater than its physical length. Figure 3 of Morris, et al. (1984) shows that excess of effective over physical length is about 0.5 m for the commonly-used symmetripole loops with high sensitivity (Leschinski 1994). Thus, for a loop length of 4.5 m, the effective detection zone length is $L_p = 5.0$ m. On the other hand, RTA (1991) suggests that for a loop length of 4.5 m, the effective detection zone length is $L_p = 4.0$ m. This matter needs to be resolved by testing typical loop configurations and lengths, and with due consideration to differences in vehicle types and speeds in relevant traffic situations. In the meantime, it is suggested that the effective detection zone length is assumed to be equal to the physical loop length.

In the analysis of signal timings under VA control, the average *saturation speed* (i.e. the average speed while vehicles depart from a queue) and the average *approach speed* after queue clearance (at the average arrival flow rate) should be used for the saturated and unsaturated parts of the green period, respectively.

The *detector occupancy time* should not be confused with the *occupancy* parameter used in the literature. The latter is defined as the proportion of road space covered by vehicles, or as the proportion of time a road section is occupied by vehicles (e.g. see AUSTROADS 1988, TRB 1994). Occupancy based on space is $\Omega_s = L_v / L_h$. This is related to the *density* parameter (k) discussed in *Section A.2*: $\Omega_s = k\, L_v$ (see *Equation A.8*). Occupancy based on time is equivalent to occupancy based on space when a passage detector is used, i.e. $\Omega_t = t_v / h = L_v / L_h = \Omega_s$ as seen from *Equations A.1 and A.2*. For presence detection, occupancy based on time is $\Omega_t = t_o / h = (L_v + L_p) / L_h$.

Example

Given presence detection zone length L_p = 4.5 m, average vehicle lengths L_{vm} = 4 m and L_{vHV} = 10 m, 5 % heavy vehicles (p_{HV} = 0.05), v_s = 50 km/h = 13.9 m/s during queue clearance with an average saturation headway of h_s = 2.0 s (single lane), and v_u = 65 km/h = 18.1 m/s after the queue clearance with an average arrival headway of h_u = 2.6 s.

The average vehicle length is found as $L_v = 0.95 \times 4 + 0.05 \times 10 = 4.3$ m. During queue discharge, the vehicle passage time is $t_{vs} = 4.3 / 13.9 = 0.3$ s, the average gap time is $t_{gs} = 2.0 - 0.3 = 1.7$ s, the average space time is $t_{ss} = 1.7 - 4.5 / 13.9 = 1.4$ s, and the occupancy time is $t_{os} = 2.0 - 1.4 = 0.6$ s. This can also be found as $t_{os} = (4.3 + 4.5) / 13.9 = 0.6$ s. From *Equation (A.3)*, the average space length of can be found as $L_{ss} = t_{gs} v_s = 1.7 \times 13.9 = 23.6$ m between successive vehicles. Thus, average spacing during the saturated part of green period is L_{hs} = 4.3 + 23.6 = 27.9 m/veh. Occupancies are $\Omega_s = 4.3 / 27.9 = 0.154$ and $\Omega_t = (4.3 + 4.5) / 27.9 = 0.315$.

After queue clearance, the vehicle passage time is $t_{vu} = 4.3 / 18.1 = 0.2$ s, the average gap time is $t_{gu} = 2.6 - 0.2 = 2.4$ s, the average space time is $t_{su} = 2.4 - 4.5 / 18.1 = 2.2$ s, and the occupancy time is $t_{ou} = (4.3 + 4.5) / 18.1 = 0.5$ s. The average space length is $L_{su} = t_{gu} v_u = 2.4 \times 18.1 = 43.4$ m between successive vehicles. Thus, average spacing during the unsaturated part of green period is L_{hu} = 4.3 + 43.4 = 47.7 m/veh. Occupancies are $\Omega_s = 4.3 / 47.7 = 0.090$ and $\Omega_t = (4.3 + 4.5) / 47.7 = 0.184$.

A.2 Speed-Flow-Density Relationships

As shown in *Figure A.3*, the basic parameters in the analysis of signal timings with VA control (*headway*, *spacing*, *occupancy time, space time* and *gap time*) can be related to the fundamental parameters of traffic flow (*flow rate, speed* and *density*). In establishing these relationships, it is necessary to distinguish between conditions during the saturated and unsaturated parts of the green period (see *Figure A.4*).

Flow rate

The *headway* is related to the *flow rate* (veh/s) through:

$$h = 1/q \qquad (A.7)$$

As seen in *Figure A.4*, the flow rate measured at the signal stop line can be:

(i) the departure flow rate (q_s) during the *saturated* part of green period (g_s), or

(ii) departure flow rate (q_u) during the *unsaturated* part of green period (g_s).

The average departure rate during the green period is $q_{dg} = (q_s g_s + q_u g_u) / g$, and the total number of vehicles which depart during the green period is $n_{vg} = q_{dg} g = q_s g_s + q_u g_u$.

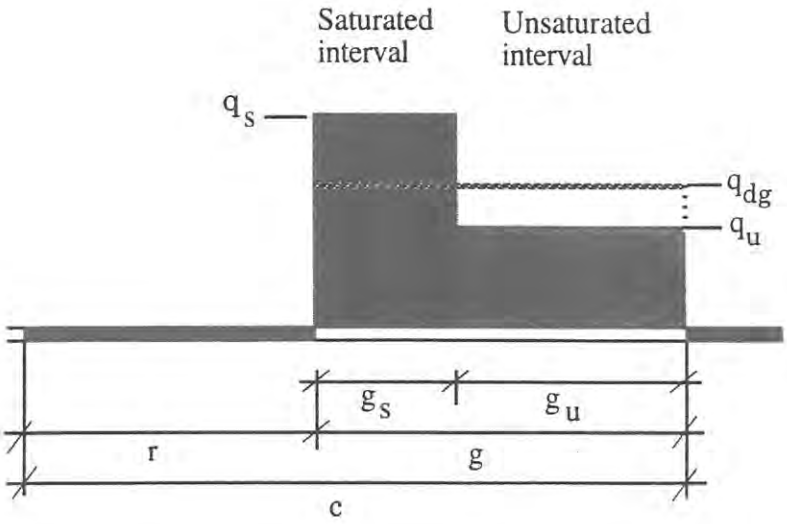


Fig. A.4 – Saturated and unsaturated portions of the green period

The departure flow rate during the unsaturated part of green period, i.e. after queue clearance, corresponds to the *arrival* flow rate as measured under uninterrupted conditions at a point upstream of the back of queue, $q_u = q_a$.

As discussed in *Section 3* in the main text of this report (see *Equation 12*), a simple platooned arrivals model used in signal timing and performance analyses assumes two different arrival rates during the red and green periods (q_{ar}, q_{ag}). This is specified through a known platoon arrival ratio, $P_A = q_{ag} / q_{ac}$.

In this case, the departure flow rate during the unsaturated part of green period is $q_u = q_{ag} = P_A\, q_{ac}$, where q_{ac} is the arrival flow rate during the signal cycle. With uniform (non-platooned) arrivals, the arrival flow rate during the signal cycle is constant ($P_A = 1.0$, $q_{ac} = q_{ag} = q_{ar} = q_a$) and $q_u = q_a$.

The departure flow rate, q_s, corresponds to the average queue discharge rate, i.e. the saturation flow rate. This can be the full saturation flow, s, if there is no downstream queue interference, or a reduced saturation flow, s_r, resulting from downstream queue interference. The reduced saturation flow is associated with a reduced saturation speed, v_r, and this condition occurs when a downstream queue results in insufficient space for the departing vehicles to accelerate to full saturation speed.

The saturation flow rates s and s_r are for the actual traffic mix including heavy vehicles. The saturation flow for all lanes of traffic is denoted s_t and the saturation flow for the critical lane is denoted s_c (see *Section B.2* in

Appendix B). The single-lane saturation flow for a stream consisting of passenger car units (or light vehicles) only is denoted s_m.

Due to rounding errors, the departure rate during the saturated part of the green period, q_s, needs to be adjusted to ensure conservation of flows ($q_{ac}\ c = q_{dg}\ g = q_s\ g_s + q_u\ g$). This is discussed in *Section A.3*.

Spacing and Density

The distance that corresponds to headway time, i.e. the distance between the front ends of two successive vehicles, is called *spacing*. This consists of the space (gap) length and the vehicle length, $L_h = L_v + L_s$. The *density* (concentration) is the number of vehicles per unit distance, and is related to the average spacing thorough:

$$k = \frac{1000}{L_h} \qquad (A.8)$$

where L_s and L_v are in metres and k is in veh/km (or $k = 1 / L_h$ in veh/m).

The average spacing in a stationary queue, L_{hj} (average queue space per vehicle) equals the vehicle length, L_v, and the average space length in queue, L_{sj}:

$$L_{hj} = L_v + L_{sj} \qquad (A.8a)$$

The *jam density*, i.e. the number of vehicles per unit distance in a stationary queue, can be calculated from the average space length in queue (L_{sj}):

$$k_j = \frac{1000}{L_{hj}} = \frac{1000}{L_v + L_{sj}} \qquad (A.8b)$$

where L_{hj}, L_v and L_{sj} are in metres and k_j is in veh/km. For example, L_v = 4.0 m and L_{sj} = 2.0 m means L_{hj} = 6.0 m, and k_j = 1000/6 = 167 veh/km.

As seen in *Figures A.1 to A.3*, the space (gap) length is given by:

$$L_s = L_h - L_v = \frac{1}{k} - L_v \qquad (A.9)$$

As the vehicles speed up from a stationary queue, the space length between vehicles increases gradually, and therefore the spacing increases and the density decreases. The corresponding flow rate increases to a maximum saturation flow value and then decreases as the speed increases towards the free flow speed. The relationships between flow, speed, density and spacing are discussed in detail in the following pages.

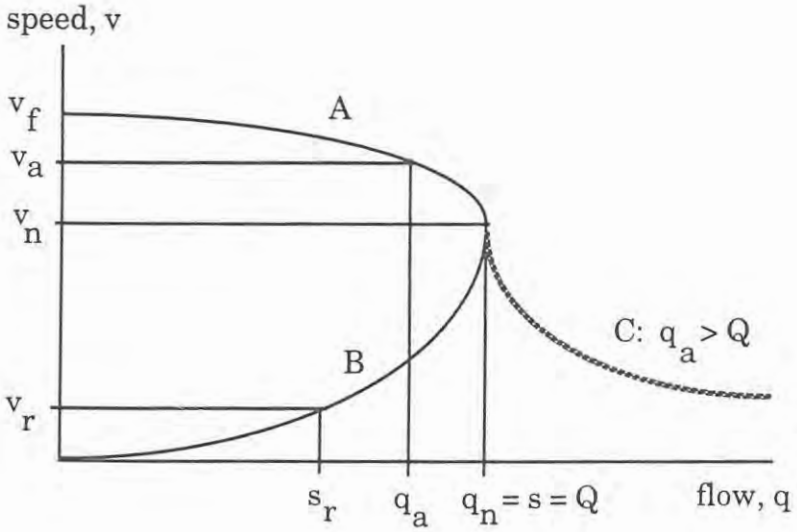


Fig. A.5a – Speed as a function of flow rate: uninterrupted flow conditions

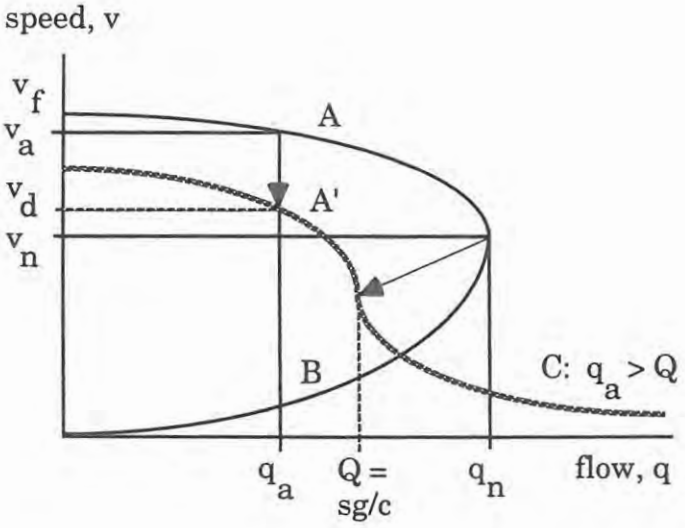


Fig. A.5b – Speed as a function of flow rate: interrupted flow conditions

Flow-Speed-Density Relationship

The fundamental relationship of traffic flow is:

$$q = v\,k \qquad \text{(A.10)}$$

where q is the flow rate (veh/h or veh/s), v is the speed (km/h or m/s) and k is the density (veh/km or veh/m). This relationship is often discussed in the context of uninterrupted flows, e.g. as observed on freeways and rural roads, but is also valid for departures across the stop line of a signalised intersection as observed during the saturated and unsaturated parts of the green period.

Since $k = 1 / L_h$, *Equation (A.10)* is equivalent to:

$$q = \frac{v}{L_h} \qquad \text{(A.10a)}$$

The speed-flow relationships for uninterrupted and interrupted conditions are depicted in *Figures A.5a and A.5b*, respectively. As seen in *Figure A.5a*, the capacity is the maximum saturation flow rate ($Q = q_n = s$). Region A represents arrival flows below capacity ($q_a \le Q$) associated with uninterrupted speeds ($v_f \ge v_a \le v_n$ where v_f is the free-flow speed and $v_n = v_s$ is the saturation speed). Region B represents the flow rates below the capacity rate ($q = s_r \le s$) associated with reduced speeds ($v_r \le v_s$). Region C represents arrival flows above capacity ($q_a > Q$) associated with average speeds based on travel time through the section. In this case, the flow represents the demand flow rate which can exceed the capacity value (as measured upstream of the capacity point), and therefore the speed-flow curve is valid for degrees of saturation in excess of 1.0. A speed-flow relationship consisting of regions A and C (used in SIDRA for uninterrupted movements) is given in Akçelik (1990).

In *Figure A.5b* for interrupted conditions, the capacity ($Q = sg/c$) is less than the average saturation flow (s reduced by the available green time ratio, g/c), and average section speeds (regions A' and C) are seen to be reduced from the approach speeds due to delays experienced due to signal or other control ($v_d < v_a$). The average speeds estimated by SIDRA for interrupted movements correspond to the speed-flow relationship consisting of regions A' and C in *Figure A.4b*.

Importantly, it has been established under the ARRB Paired Intersections Project that the flow rate for the congested region (B) at a signalised intersection stop line is the rate of departure from the queue. This corresponds to the instantaneous saturation flow rate during the green period that increases to a maximum saturation flow rate (q_n) when there is no downstream queue interference. This consideration brings an explanation to the speed-flow models for uninterrupted flows as well: the flow rate measured for forced-flow (congested) conditions represents a *reduced capacity flow* (not an arrival flow). This is a result of vehicles (or moving queues) in the same stream interfering in the same way as the downstream queue interferes with the departure rate at the signal stop line, resulting in reduced vehicle spacings, speeds and flow rates. Wardrop's (1965) speed-flow surveys of traffic in a circular track is interesting in this respect.

The maximum saturation flow rate (q_n) is higher than the average saturation flow rate (s) during the green period used in capacity calculation by about 5 to 20 per cent depending on the saturation flow measurement method (s includes the effects of the lower departure flow rates during the early part of green period).

For the analysis of departures during green period at traffic signals, modelling of both regions A and B of the speed-flow relationship is necessary. Once a relationship between two of the variables in *Equation (A.10)*, i.e. v-k, q-k or v-q, is established, all other relationships as expressed in *Equations (A.1) to (A.9)* are determined.

There is a vast amount of literature on the flow-speed-density relationships for uninterrupted traffic flow offering empirical as well as theoretical models based on car-following, hydrodynamic and kinetic theories of traffic flow. Based on theoretical approaches, various speed-density models (linear, parabolic, exponential) can be derived from $dv/dk = - c\ v^m\ k^{l-2}$ where parameters m and l are selected to satisfy boundary conditions such as $v = v_f$ when $q = 0$ and $k = 0$, $v = 0$ when $k = k_j$, $dq/dk = 0$ when $k = k_n$, etc. (Akçelik 1974). However, it has not been possible to obtain a satisfactory model, including two-regime models, that gives realistically high values of $v_f / v_n = 0.6\text{-}0.9$ (e.g. see TRB 1994) associated with low values of $k_n / k_j = 0.15\text{-}0.30$ where v_n and k_n are the speed and density corresponding to the maximum saturation flow rate. For example, $q_n = 1800$ veh/h and $v_n = 45$ km/ h give $k_n = q_n / v_n = 40$ veh/km, and using $k_j = 1000/6.3 = 159$ veh/km, $k_n / k_j = 0.25$.

A useful method to derive the fundamental speed-flow-density relationships is to start by modelling vehicle spacing as a function of speed (L_h-v) as discussed in detail by Lay (1979, 1985) who showed that this approach is well related to driver behaviour (reaction times, stopping distances, out-of-lane factors, and so on). Lay (1979) proposed a polynomial spacing-speed model which produced all desirable characteristics of a speed-flow relationship except one condition: speeds exceeded the free-flow speed as the flows approached zero in Region A ($v > v_f$ when $q = 0$).

The following spacing-speed model developed under the ARRB Paired Intersections Project produces a speed-flow model that satisfies the condition $v = v_f$ when $q = 0$ in Region A, and its validity has been confirmed using real-life data collected at signalised intersection stop lines in Melbourne (a detailed paper is to be published in near future):

$$L_h = L_{hj} + \frac{p_1\, v}{(1 - \rho_v)\, p_2} \qquad \text{(A.11)}$$

The resulting speed-flow and speed-density models are:

$$q = \frac{1000\, v}{L_{hj} + \dfrac{p_1\, v}{(1 - \rho_v)\, p_2}} \qquad \text{(A.11a)}$$

$$k = \frac{1000}{L_{hj} + \frac{p_1 v}{(1 - \rho_v) p_2}} \quad \text{(A.11b)}$$

where L_h (m/veh), q (veh/h) and k (veh/km) are the average spacing, flow rate and density at speed v (km/h) , L_{hj} is the jam spacing ($L_{hj} = 1000 / k_j$), p_1, p_2 are the model calibration parameters, and ρ_v is the speed ratio defined as the ratio of a given speed to the free-flow speed:

$$\rho_v = \frac{v}{v_f} \qquad \textit{subject to } \rho_v \leq 1.0 \quad \text{(A.11c)}$$

The observed speed ratios at maximum saturation flow ($\rho_{vn} = v_n / v_f$) increase as the facility type improves, e.g. freeways have higher values than signalised arterials. As a rough guide where information is not available, the speed ratio can be estimated from:

$$\rho_{vn} = 0.3 + 0.005\, v_f \qquad \textit{subject to } \rho_v \geq 0.5 \quad \text{(A.11d)}$$

where the free-flow speed v_f is in km/h (e.g. v_f = 100-120 km/h for freeways, 60-90 km/h for arterials, 40-50 km/h for sub-arterials).

Similarly, a density ratio is defined as the ratio of the density at a given speed to the jam density:

$$\rho_k = \frac{k}{k_j} = \frac{L_{hj}}{L_h} \qquad \textit{subject to } \rho_k \leq 1.0 \quad \text{(A.11e)}$$

Parameter p_1 is related to driver reaction times (including out-of-lane effects) and determines the extra spacing required under low to medium flow conditions in Region A. Parameter p_2 is important in determining the characteristics of the speed-flow model for conditions around the maximum flow rate and in Region B. The maximum flow rate (q_n), the corresponding speed (v_n), the free-flow speed (v_f) and jam density ($k_j = 1000 / L_{hj}$) are the parameters that can be established externally. If these parameters are known, then model parameter p_2 and p_1 can be determined from:

$$p_2 = \frac{(1 - \rho_{vn})\, q_n}{v_f k_j (\rho_{vn})^2 - \rho_{vn}\, q_n} \quad \text{(A.11f)}$$

$$p_1 = \frac{(L_{hn} - L_{hj})(1 - \rho_{vn})\, p_2}{v_n} \quad \text{(A.11g)}$$

where $\rho_{vn} = v_n / v_f$ is calculated using the known values of v_n and v_f, or estimated from *Equation (A.11d)*.

The spacing that corresponds to the maximum saturation flow rate is given by:

$$L_{hn} = \frac{L_{hj}\,[1 - (1 - p_2)\,\rho_{vn}]}{p_2\, \rho_{vn}} \quad \text{(A.11h)}$$

and the density ratio at maximum saturation flow is given by:

$$\rho_{kn} = \frac{k_n}{k_j} = \frac{L_{hj}}{L_{hn}} = \frac{p_2 \rho_{vn}}{1 - (1 - p_2) \rho_{vn}} \qquad \text{(A.11i)}$$

In the above formulae, spacings are in m/veh, speeds are in km/h, flow is in veh/h, and densities are in veh/km.

Example

Graphs of spacing-speed (L_h-v), speed-flow (v-q), speed-density (v-k), flow-density (q-k), headway-speed (h-v), gap time-speed (t_g-v), occupancy time-speed (t_o-v), space time-speed (t_s-v) relationships are shown in *Figures A.6 to A.11* for the following example:

v_f = 80 km/h, v_n = 56 km/h = 15.56 veh/s ($\rho_{vn} = v_n / v_f = 0.70$),
q_n = 1975 veh/h (0.549 veh/s)
detection zone length, L_p = 4.5 m
average vehicle length, L_v = 4.0 m
jam spacing, L_{hj} = 6.0 m/veh (jam density, k_j = 1000 / 6.0 = 166.7 veh/km)

From *Equations (A.11f to (A.11i)*, p_2 = 0.115, p_1 = 0.348, L_{hn} = 28.4 m/veh, $\rho_{kn} = k_n / k_j = 0.212$ and k_n = 35.3 veh/km are found.

Other parameters for the maximum saturation flow point can be calculated from *Equations (A.1) to (A.9)*:

headway $h_n = 1 / q_n = 1 / 0.549 = 1.82$ s
occupancy $t_{on} = (L_v + L_p) / v_n = 8.5 / 15.56 = 0.55$ s
space time $t_{sn} = h_n - t_{on} = 1.82 - 0.55 = 1.27$ s
vehicle passage time $t_{vn} = L_v / v_n = 4.0 / 15.56 = 0.26$ s
gap time $t_{gn} = h_n - t_{vn} = 1.82 - 0.26 = 1.56$ s
space (gap) length $L_{sn} = L_{hn} - L_v = 28.4 - 4.0 = 24.4$ m/veh

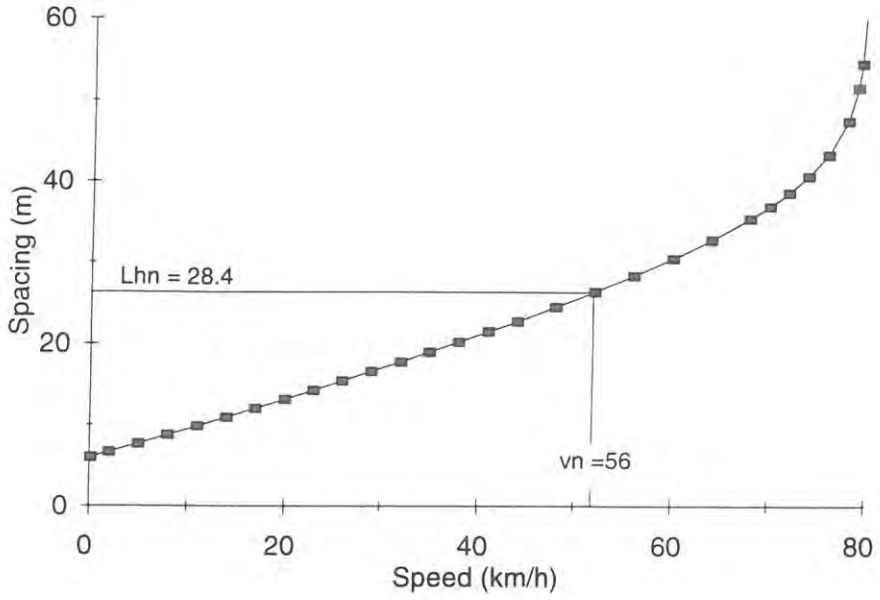


Fig. A.6 – Spacing as a function of speed

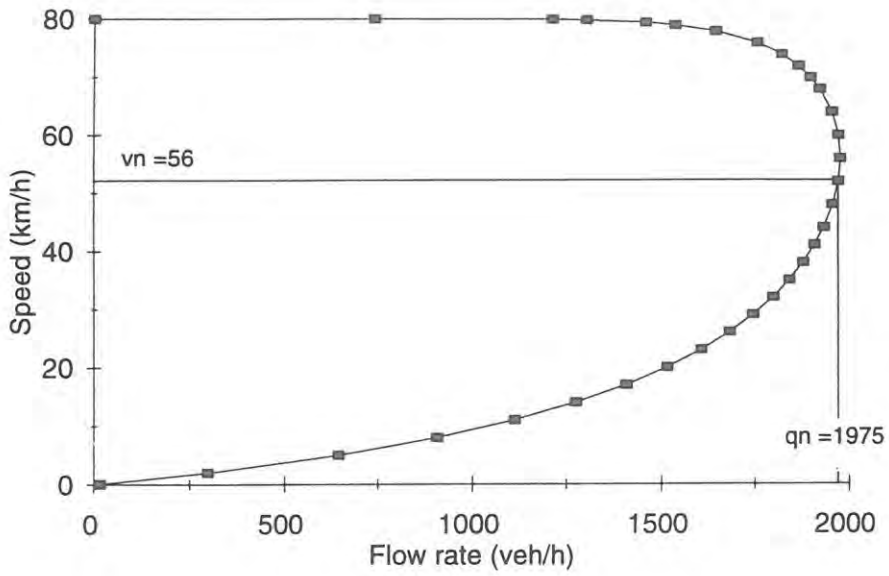


Fig. A.7 – Speed as a function of flow rate

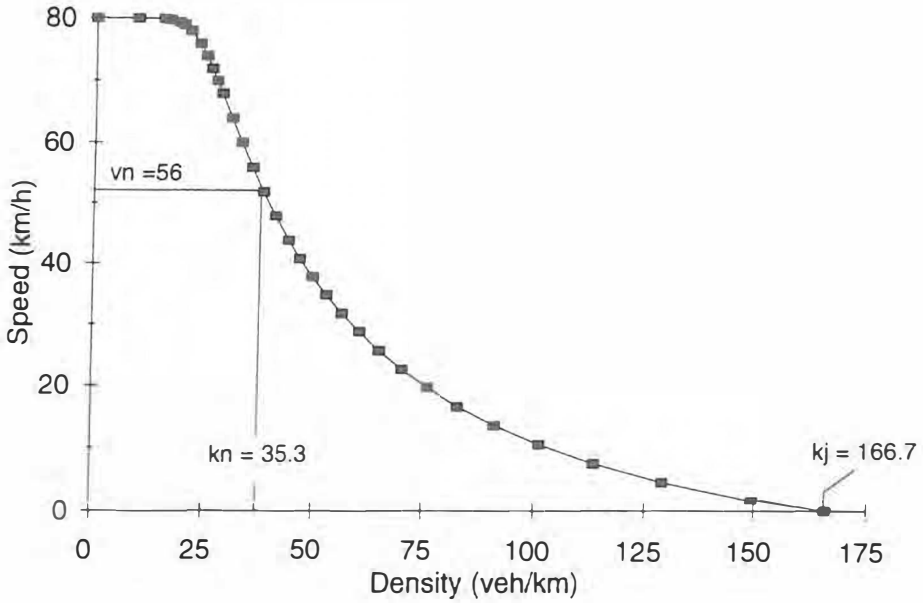


Fig. A.8 – Speed as a function of density

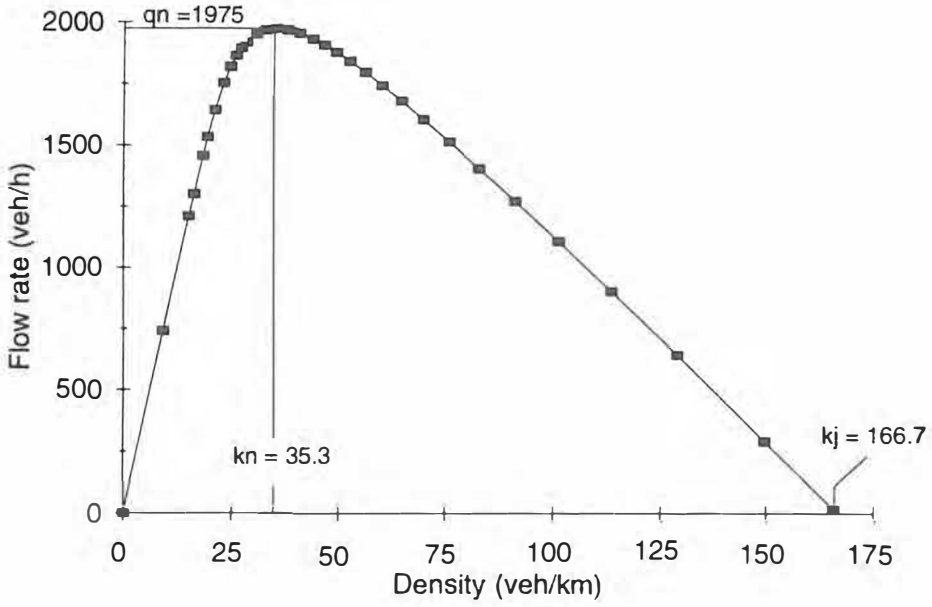


Fig. A.9 – Flow rate as a function of density

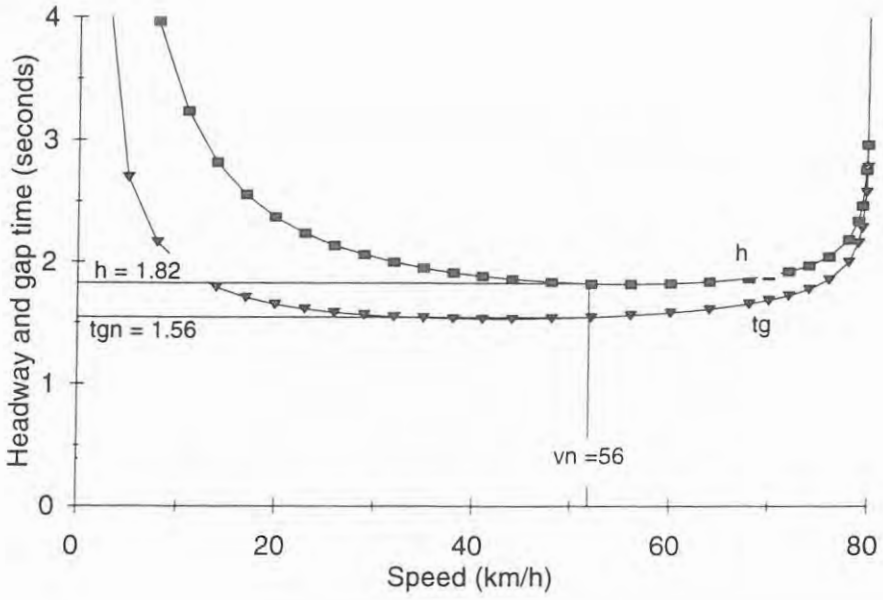


Fig. A.10 – Headway and gap time as a function of speed

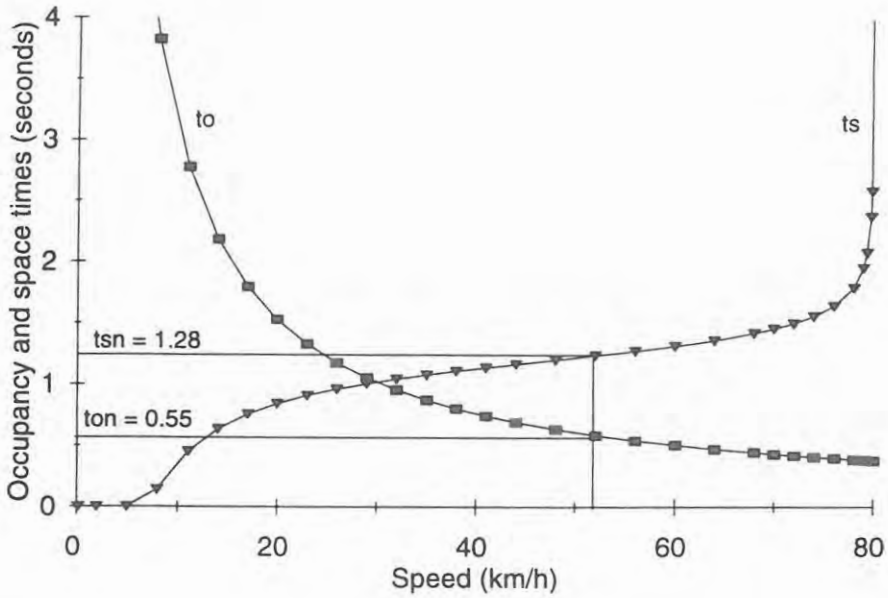


Fig. A.11 – Occupancy time and space time as a function of speed

A.3 SCATS DS Parameter

An important parameter in Australian traffic control practice is the *SCATS DS parameter*. This is the basic strategic control parameter in the operation of the SCATS dynamic signal control system (Lowrie 1982, 1990), including the control of an isolated intersection. The DS parameter is defined as "the ratio of the effectively used green time to the total available green time", and is formulated in terms of presence loop occupancy and volume data measured by the system. Lowrie (1982, 1990) qualifies the SCATS DS parameter as "*analogous to degree of saturation*".

In this section, the DS parameter is expressed using the notation of this paper, and its relation to the *degree of saturation* used in the traditional traffic signal analysis methods (e.g. Akçelik 1981, 1990; AUSTROADS 1993) is discussed. The analysis is presented as applicable to a critical lane. The emphasis in the discussion presented here is the *meaning* of the DS parameter rather than the role of this parameter in actual SCATS operations which is outside the scope of this paper. Some documentation on SCATS includes a calibration parameter (NF) in the formulation of the DS parameter. This calibration factor will be ignored in discussing the basic meaning of the DS parameter.

The definition of the SCATS DS parameter as the ratio of the effectively used green time to the total available green time (Lowrie 1982, 1990) can be expressed as:

$$DS = \frac{g - \Sigma\, t_{si} + n_{vg}\, t_{sm}}{g} \qquad (A.12)$$

where g is the total available green time, $\Sigma\, t_{si}$ is the total space time during the green period including both saturated and unsaturated intervals (t_{si} is the space time of the *ith* vehicle that departs during the green period), n_{vg} is the number of vehicles that depart during the green period, and t_{sm} is the average space time at *maximum flow* (SCATS MF parameter, i.e. the saturation flow for *pcu*s only, s_m).

As discussed in *Section A.2* (*Figure A.3*), the average number of vehicles that depart during the green period is $n_{vg} = q_s\, g_s + q_u\, g_u = q_{dg}\, g$, where g_s and g_u are the durations of the saturated and unsaturated parts of green period (calculated from *Equations (11) to (14)* in *Section 3* of the main text of this report), q_s and q_u are the corresponding departure flow rates, and q_{dg} is the average departure flow rate during the green period, g.

The departure flow rate during the *unsaturated* part of green period corresponds to the average arrival flow rate during the green period, q_{ag} (for the critical lane): $q_u = q_{ag} = P_A\, q_{ac}$, where q_{ac} is the average arrival flow rate during the cycle and P_A is the platoon arrival ratio (for uniform arrivals, $P_A = 1.0$, $q_u = q_{ag} = q_a$).

The departure flow rate during the *saturated* part of green period corresponds to the critical lane saturation flow rate (the full saturation flow, s_c, or the reduced saturation flow, s_r, for the actual traffic mix). Due to rounding errors and as a results of the use of $f_q > 1.0$ in calculating g_s (*Equation 13* in *Section 3*

of the main text of this report), the departure rate during the saturated part of the green period, q_s, needs to be adjusted to ensure conservation of flows:

$$\begin{aligned} q_s &= q_{ac}\,\frac{c - P_A\,g_u}{g_s} && \text{for } q_{ac}\,c \le s_c\,g \\ &= s_c && \text{for } q_{ac}\,c > s_c\,g \end{aligned} \qquad \text{(A.13)}$$

where c is the cycle time and other variables are as explained above.

For an undersaturated cycle ($q_{ac}\,c \le s_c\,g$), n_{vg} equals the number of vehicles arriving during the signal cycle, $q_{ac}\,c$, and for an oversaturated cycle, ($q_{ac}\,c > s_c\,g$), n_{vg} equals the cycle capacity, $s_c\,g$. Thus, the following formula can be used to calculate the average number of vehicles that depart during the green period:

$$\begin{aligned} n_{vg} &= q_{ac}\,c && \text{for } q_{ac}\,c \le s_c\,g \\ & s_c\,g && \text{for } q_{ac}\,c > s_c\,g \end{aligned} \qquad \text{(A.13a)}$$

The average departure flow rate during the green period is $q_{dg} = n_{vg} / g$, and the average departure headway during the green period is

$$h_{dg} = 1 / q_{dg} = g / n_{vg} \qquad \text{(A.13b)}$$

and the average space time during the green period is:

$$t_{sg} = \Sigma\, t_{si} / n_{vg} \qquad \text{(A.13c)}$$

Using these relationships, *Equation (A.12)* can be written as:

$$DS = \frac{h_{dg} - t_{sg} + t_{sm}}{h_{dg}} \qquad \text{(A.14)}$$

or putting $h_{dg} - t_{sg} = t_{og}$ (average occupancy time during the green period):

$$DS = \frac{t_{og} + t_{sm}}{h_{dg}} \qquad \text{(A.15)}$$

The average occupancy time during the green period can be calculated from:

$$t_{og} = \frac{\Sigma\, t_{oi}}{n_{vg}} = \frac{(q_s\,g_s)\,t_{os} + (q_u\,g_u)\,t_{ou}}{q_s\,g_s + q_u\,g_u} \qquad \text{(A.16)}$$

where $\Sigma\, t_{oi}$ is the sum of all occupancy times during the green time (t_{oi} is the occupancy time of the *ith* vehicle), t_{os} and t_{ou} are the average occupancy times during the saturated and unsaturated portions of the green period, and ($q_s\,g_s$), ($q_u\,g_u$) are the numbers of vehicles departing during the saturated and unsaturated portions of the green period, respectively.

For an undersaturated cycle ($g_u > 0$, $n_{vg} = q_a\,c$):

$$t_{og} = \frac{(q_s\,g_s)\,t_{os} + (q_u\,g_u)\,t_{ou}}{q_a\,c} \qquad \text{(A.16a)}$$

For an oversaturated cycle ($g_u = 0$, $g = g_s$, $n_{vg} = s_c\,g$):

$$t_{og} = t_{os} \qquad \text{(A.16b)}$$

The average occupancy times during the saturated and unsaturated portions of the green period, t_{os} and t_{ou}, are calculated from *Equation (A.5)* using the average departure speeds across the signal stop line during the saturated and unsaturated intervals, v_s and $v_u = v_a$, respectively. The uninterrupted approach speed, v_a, corresponds to the arrival flow rate as measured at a point upstream of the back of queue. As a rough relation between the saturation speed and the approach speed, $v_s / v_a \approx 0.6$ to 0.8. For the calculation of occupancy times, an average vehicle length, L_v, for the actual traffic mix including heavy vehicles should be calculated using *Equation (A.6)*.

For the calculation of the average space time at *maximum flow* ($t_{sm} = h_m - t_{om} = 1/s_m - t_{om}$), the occupancy time t_{om} should be calculated using the average vehicle length for a pcu ($L_{vm} \le L_v$) and the average speed at saturation for a traffic lane consisting of *pcu*s only ($v_m \ge v_s$).

The saturation headway for *pcu*s ($h_m = 1/s_m$) is smaller than the saturation headway for the actual traffic mix ($h_s = 1/s_c$). Given s_m, the saturation flow for the actual traffic mix s_c can be calculated from:

$$s_c = s_m / f_c \qquad \text{(A.17)}$$

where f_c is the traffic composition factor given by:

$$f_c = p_{HV} (f_{HV} - 1) + 1 \qquad \text{(A.17a)}$$

where f_{HV} is a heavy vehicle factor for saturation flow purposes (typically f_{HV} = 1.5 to 2.0 veh/pcu).

Let us now define a slightly different parameter (DS') by replacing parameter t_{og} (average occupancy time during the green period) in *Equation (A.15)* by t_{os} (average occupancy time during the saturated part of the green period), and t_{sm} (average space time at *maximum flow*, s_m) by t_{ss} (average space time at the actual saturation flow, s_c):

$$DS' = \frac{t_{os} + t_{ss}}{h_{dg}} \qquad \text{(A.18)}$$

Since $t_{os} + t_{ss} = h_s = 1 / s_c$ (average headway during the saturated part of the green period), and $h_{dg} = g / n_{vg}$:

$$DS' = \frac{h_s}{h_{dg}} = \frac{n_{vg}}{s_c\ g} \qquad \text{(A.18a)}$$

For undersaturated conditions ($x \le 1.0$), the average number of departures equals the average number of arrivals per cycle, $n_{vg} = q_a c$ and therefore DS' is equivalent to the *degree of saturation:*

$$DS' = \frac{q_a c}{s_c\ g} = x \qquad \textit{for } q_{ac}\, c \le s_c\, g \qquad \text{(A.19)}$$

Comparing *Equations (A.15) and (A.18)*, it can be seen that, under normal circumstances, DS is less than x (DS < x). This is because (i) $t_{og} < t_{os}$: during the saturated part of the green period, the average speed is lower, and therefore the occupancy time is larger as seen from *Equation (A.5)*, and

(ii) t_{sm} (= $h_m - t_{om}$) ≈ t_{ss} (= $h_s - t_{os}$) since $s_c < s_m$, $h_s > h_m$ and the average occupancy times, $t_{om} < t_{os}$ ($L_v > L_{vm}$ and $v_s < v_m$).

For oversaturated conditions, i.e. when the demand exceeds capacity (x > 1.0), $g_s = g$, $g_u = 0$, and therefore $h_{dg} = h_s$ and DS' = 1.0 (less than actual degree of saturation). Thus, DS' would fail to indicate the actual (demand-related) degree of saturation. This also applies to the SCATS DS parameter.

However, DS can be greater than 1.0 when downstream queue interaction results in a low departure flow rate (reduced saturation flow rate) and speed, producing very large occupancy times, hence very low space times as seen in *Figure A.11*. This results in $t_{sm} > t_{sg}$, and therefore DS > 1.0 as seen from *Equation (A.14)*. On the other hand, DS' could never be greater than 1.0 since under all conditions of oversaturation, $h_s = h_{dg}$ and DS' = 1.0.

It can be shown that, for undersaturated conditions, the SCATS DS parameter is close to the analytical degree of saturation (DS ≈ x). The percentage difference between DS and x can be expressed as:

$$DSE = 100 \left(\frac{DS}{x} - 1 \right) \qquad (A.20)$$

An example is given below to indicate possible differences using typical values of relevant parameters. The results are summarised in *Table A.1*.

Example

Given: $p_{HV} = 0.05$ (5 per cent heavy vehicles), $f_{HV} = 2.0$, $s_m = 1800$ pcu/h = 0.5 pcu/s, $L_{vm} = 4$ m/pcu, $L_{vHV} = 10$ m/HV, $L_p = 4.5$ m, $v_u = 60$ km/h = 16.7 m/s, v_s = 40 km/h = 11.1 m/s, v_m = 45 km/h = 12.5 m/s, arrival flow rate $q_a = 900$ veh/h = 0.250 veh/s (uniform arrivals), cycle time c = 90 s, green time is varied to obtain various levels of degree of saturation. To calculate the saturated part of green for this example, *Equation (11a)* in *Section 3* of this report is used with $f_q = 1.0$. The results are summarised in *Table A.1*. Calculations are explained below (results summarised in *Table A.1*).

The saturation flow for the given traffic composition is calculated from *Equation (A.17)* as $f_c = 0.05\ (2 - 1) + 1 = 1.05$ and $s_c = 1800 / 1.05 = 1714$ veh/h = 0.4762 veh/s. The corresponding saturation headways are $h_m = 1 / 0.5 = 2.0$ s/pcu and $h_s = 1 / 0.4762 = 2.1$ s/veh.

The average vehicle length is found from *Equation (A.6)* as $L_v = (1 - 0.05)$ x 4 + 0.05 x 10 = 4.3 m/veh. The occupancy and space times are found from *Equations (A.4) and (A.5)* as $t_{os} = (4.3 + 4.5)/11.1 = 0.79$ s, $t_{ou} = (4.3 + 4.5)/16.7$ = 0.53 s, $t_{om} = (4.0 + 4.5)/12.5 = 0.68$ s, $t_{sm} = h_m - t_{om} = 2.0 - 0.68 = 1.32$ s and $t_{ss} = h_s - t_{os} = 2.1 - 0.79 = 1.31$ s ($t_{ss} \approx t_{sm}$).

The number of vehicles arriving per cycle is $q_a\ c = 0.250$ x 90 = 22.5 veh. The flow ratio is y = 900/1714 = 0.525, and the duration of the saturated part of green period is $g_s = 0.525$ x r /(1 – 0.525) = 1.1053 r. The departure rates during the saturated and unsaturated parts of the green period are $q_s = s_c = 0.4762$ veh/s and $q_u = q_a = 0.250$ veh/s.

Table A.1

SCATS DS parameter and the degree of saturation: an example (c = 90 s, q_a = 900 veh/h, s_c = 1714 veh/h)

Case	g	r	g_s	g_u	n_{vg}	h_{dg}	t_{og}	DS	x	DS'	DSE
Low x	80	10	11.1	68.9	22.5	3.56	0.59	0.537	0.591	0.591	– 9.1 %
Medium x	60	30	33.2	26.8	22.5	2.67	0.71	0.723	0.788	0.788	– 8.2 %
High x	50	40	44.2	5.8	22.5	2.22	0.77	0.941	0.945	0.945	– 0.4 %
Oversat.	45	45	45	0	21.4	2.1	0.79	1.005	1.050	1.00	– 4.3 %
High oversat.	40	50	40	0	19.0	2.1	0.79	1.005	1.181	1.00	– 14.9 %
Downstr.* queue int.	60	30	60	0	20.0	3.0	2.11	1.143	1.125	1.00	+ 1.6 %

* Reduced saturation flow s = s_r = 1200 veh/h.

The results for various levels of degree of saturation obtained by varying the green time (cycle time constant) are given in *Table A.1*. The calculations for the medium degree of saturation case are as follows: g = 60 s, r = 90 – 60 = 30 s, g_s = 1.1053 x 30 = 33.2 s, g_u = g – g_s = 60 – 33.2 = 26.8 s, therefore n_{vg} = q_a c = 22.5 veh. From *Equation (A.13b)*, h_{dg} = 60 / 22.5 = 2.67 s. From *Equation (A.16a)*, t_{og} = (0.4762 x 33.2 x 0.79 + 0.250 x 26.8 x 0.53) / 22.5 = 0.71 s. From *Equation (A.15)*, DS = (0.71 + 1.32) / 2.67 = 0.723. From *Equation (A.19)*, DS' = x = 22.5 / (0.4762 x 60) = 0.788. From *Equation (A.20)*, DSE = 100 x (0.723 / 0.788 – 1) = – 8.2 %.

The last case in *Table A.1* assumes a reduced saturation flow rate of s = s_r = 1200 veh/h = 0.333 veh/s, (therefore h_s = 3.0 s) and a reduced saturation speed of v_s = v_r = 15 km/h = 4.17 m/s. In this case, r = 90 – 60 = 30 s, y = 900 / 1200 = 0.75, g_s = 0.75 x 30 / (1 – 0.75) = 90.0 s > g, therefore g_s = g = 60.0 s, g_u = 0, n_{vg} = 0.333 x 60 = 20.0 veh (< q_a c), h_{dg} = 60 / 20.0 = 3.0 s = h_s, t_{og} = t_{os} = (4.3 + 4.5) / 4.17 = 2.11 s, DS = (2.11 + 1.32) / 3.0 = 1.143, DS' = 3.0 / 3.0 = 1.0, x = 22.5 / 20.0 = 1.125 < DS', and DSE = 100 x (1.143 / 1.125 – 1) = + 1.6 %.

As seen in the examples given in *Table A.1*, the difference between the SCATS DS parameter and the degree of saturation (x) gets smaller as the degree of saturation increases towards 1.0, and then increases again with increasing x values above 1.0. The DS parameter is seen to be able to predict oversaturation when downstream queue interaction occurs (low saturation flows and speeds). The alternative measure DS' is equivalent to x for undersaturated conditions (x < 1), but it is not able to predict oversaturation with or without downstream queue interaction. However, downstream congestion effects (low saturation flows and speeds) could be identified directly through much reduced space time values (e.g. by comparing against the space time at maximum flow, t_{sm}). In the example in *Table A.1*, t_{og} = t_{os} = 2.11, h_{dg} = hs = 3.0, and the average space time t_{sg} = t_{ss} = 3.0 – 2.11 = 0.89 s.

A wider range of operating conditions (speed, arrival flow, saturation flow, heavy vehicle percentage, etc.) should be analysed to establish the range of possible differences between DS and x. Large differences between DS and x at low degrees of saturation is not a serious problem as the performance measures (delay, queue length, etc.) are not very sensitive to the degree of saturation at low degrees of saturation. On the other hand, a small underestimation of the degree of saturation near capacity can be serious in terms of performance. In the example given in *Table A.1*, the difference between DS and the actual x appears to be negligible for conditions just below capacity.

Theoretically, the use of DS' instead of DS would be possible in practice by finding a way of measuring the average saturation headway h_s. Although DS' would give a better measure of the real degree of saturation for undersaturated conditions ($x \leq 1.0$), it would fail to indicate oversaturation ($x > 1$) directly. As mentioned above, downstream congestion effects (low saturation flows and speeds) could be identified directly through low space time values, i.e. as the case when the average space time during the green period is less than the space time at full saturation flow ($t_{sg} < t_{ss}$, or $t_{sg} < t_{sm}$).

APPENDIX B

Methods to Determine VA Control Settings

B.1 Introduction

Methods to determine the minimum, maximum, gap, headway and waste settings for vehicle-actuated signal control are given here. These methods are based widely on guidelines given in existing Australian documents (AUSTROADS 1993, RTA 1991), extending and clarifying some aspects of them. It is expected that better guidelines based on the results of the current ARRB project on actuated signals will be available in the near future.

The typical values of the settings used in Australian practice are mentioned throughout this Appendix. These values are based on a survey carried out under the current ARRB VA signal project (see *Appendix C*).

In the case of gap and headway settings, it is important that the meanings of the settings are understood clearly. The terminology used for controller settings reflects the historical development of VA controllers, and is not consistent with the terms gap time and headway time as used in *Appendix* A. In particular, the gap setting can take several meanings (space time, gap time or headway) according to the detection type, historical usage of the term, and the requirements of analytical formulation. The headway setting, on the other hand, is a saturation space time value in the context of presence detection used with modern controllers. For this reason, controller settings have been referred to as gap setting, headway setting, etc. without the word *time* (e.g. as *gap setting* rather than *gap time setting* which has a more specific meaning).

For the purpose of determining VA controller settings, various formulae have been expressed in terms of displayed green and red times assuming that they are approximately the same as the effective green and red times.

B.2 Determining the Critical Lane

The method given in the main text of this report for determining the extension green times as well as the methods given in this Appendix for calculating VA controller settings are based on information related to the *critical lane*. In general terms, critical lane is defined as the lane with the *highest degree of saturation* considering all movements that run in the phase. The best method to determine the critical lane is a full lane flow analysis as described in ARR No. 123 (Akçelik 1981) and implemented in the SIDRA package (Akçelik 1990). For the purpose of simpler analysis, the following formula can be used to calculate the proportion of flow in the critical lane (p_c):

$$p_c = s_c / s_t \qquad \text{(B.1)}$$

where s_c is the critical lane saturation flow (veh/h), and s_t is the total movement saturation flow (veh/h) given by

$$s_t = \Sigma \rho_i s_i \quad \text{(B.1a)}$$

where i denotes the lane number, s_i is the lane saturation flow, and ρ_i is the *lane utilisation ratio* given by

$$\rho_i = x_i / x_c \quad \text{(B.1b)}$$

where x_i is the degree of saturation of the *ith* lane ($x_i = q_{ai} / Q_i$ where q_{ai} is the arrival flow rate and Q_i is the capacity) and x_c is the degree of saturation of the *critical lane* (the largest degree of saturation in any lane).

Thus, the lane utilisation ratio is defined as the ratio of the degree of saturation of a given lane to the degree of saturation of the critical lane. An underutilised lane has $\rho_i < 1.0$ since $x_i < x_c$. A reasonable assumption for simple analysis purposes is *equal lane utilisation*, in which case, all lanes have $x_i = x_c$, therefore $\rho_i = 1.0$, and the total saturation flow is simply:

$$s_t = \Sigma s_i \quad \text{(B.1c)}$$

However, the use of *Equation (B.1a)* which allows for unequal lane utilisation would be better in practice to reflect lower flow rates in left-turn and right-turn lanes. A reasonable general assumption would be to use $\rho_i = 0.8$ to 0.9 for such lanes although lower values would be justified to reflect the conditions in short lanes and shared lanes with opposed turns.

Using p_c from *Equation (B.1)*, the arrival flow rate in the critical lane (q_c) can be calculated as $q_c = p_c q_{at}$ where q_{at} is the total arrival flow for the movement, i.e. sum of all lane flows, $q_{at} = \Sigma q_{ai}$.

The general method allows for different effective green times for individual lanes that belong to the same movement, reflecting time losses to traffic in some lanes due to pedestrian interference, opposed turns waiting for gaps, and lane blockages in shared lanes. To explain the meaning of *Equation (B.1)*, let us assume that all lanes have the same effective green time. Under this assumption, the lane utilisation ratio is equivalent to:

$$\rho_i = y_i / y_c \quad \text{(B.1d)}$$

where y_i is the flow ratio of the *ith* lane ($y_i = q_{ai} / s_i$ where q_{ai} is the arrival flow rate and s_i is the saturation flow) and y_c is the flow ratio of the *critical lane* (the largest flow ratio in any lane).

Using this definition and *Equation (B.1a)*, the flow ratio for the movement is calculated as $y = q_{at} / s_t = y_c$, i.e. the movement flow ratio equals the critical lane flow ratio. Thus, $q_{at} / s_t = q_c / s_c$ and $p_c = q_c / q_{at} = s_c / s_t$.

The arrival flow for any lane can be estimated from:

$$q_{ai} = x_c \rho_i Q_i \quad \text{(B.1e)}$$

Using the simplified definition of ρ_i given in *Equation (B.1d)*, and putting $y_c = y = y = q_{at} / s_t$ (movement flow ratio), the arrival flow rate for the *ith* lane is given by:

$$q_{ai} = y \rho_i s_i \quad \text{(B.1f)}$$

It can be seen that the lane with the highest ($\rho_i s_i$) value will have the highest flow, and therefore can be identified as the critical lane. In the case of equal lane utilisation ($\rho_i = 1.0$), the lane with the highest saturation flow will have the highest flow rate, and should therefore be treated as the critical lane.

In the case of equal lane utilisation ($\rho_i = 1.0$) and when all lane saturation flows are equal ($s_i = s_c$), arrival flow rates for all lanes will be equal, and therefore $q_{ai} = q_c = q_{at} / n_l$ where n_l is the number of lanes available to the movement. The movement saturation flow in this case is $s_t = n_l s_c$, and the proportion of arrival flow in the critical lane is

$$p_c = 1 / n_l \quad \text{(B.1g)}$$

It is seen that the use of the number of lanes in the calculation of various control settings (as used in the current guides) is valid for the simple case when all lane saturation flows are equal and all lanes are used equally. The method given here generalises this simpler approach by allowing for different saturation flows and utilisation ratios for individual lanes. This provides a simple method for determining the proportion of flow in the critical lane without carrying out lane flow calculations. Movement saturation flow can be determined through the use of a manual method such as described in ARR No. 123 (Akçelik 1981), or a package such as SIDRA (Akçelik 1990).

Example

A movement has three lanes with saturation flows of $s_1 = 1400$ veh/h, $s_2 = 1800$ veh/h, $s_3 = 1500$ veh/h, and a total arrival flow rate of $q_{at} = 2000$ veh/h. Lane 1 is known to be underutilised due to the effects of buses and adjacent parking. The utilisation ratio is estimated as $\rho_1 = 0.50$ (for other lanes, $\rho_2 = \rho_3 = 1.0$). The movement saturation flow is therefore found as 0.50 x 1400 + 1.00 x 1800 + 1.00 x 1500 = 4000 veh/h. The critical lane is determined as Lane 2 since it has the highest ($\rho_i s_i$) value, thus $s_c = 1800$ veh/h. The proportion of flow in the critical lane is found from *Equation (B.1)* as $p_c = 1800/ 4000 = 0.45$, and the critical lane flow is $q_c = q_2 = p_c q_{at} = 0.45$ x 2000 = 900 veh/h. The flow ratios are found as y = 2000/4000 = 0.50 = y_c = 900/1800. Other lane flows can be found from *Equation (B.1f)* as $q_{a1} = 0.50$ x 0.50 x 1400 = 350 veh/h, and $q_{a3} = 0.50$ x 1.0 x 1500 = 750 veh/h. Total flow rate is $q_{at} = 350 + 900 + 750 = 2000$ veh/h as given.

B.3 Minimum Green Setting

The minimum green time consists of a *fixed minimum green time* (G_{mf}) and a *variable initial green time* (G_{iv}):

$$G_{min} = G_{mf} + G_{iv} \quad (B.2)$$

The *fixed minimum green time* is determined as a safe minimum green which should be long enough for the first vehicle to start moving and enter the intersection. Typically:

$$G_{mf} = 4 \text{ to } 10 \text{ seconds} \quad (B.3)$$

This reflects the current Australian practice (see *Tables C.3 and C.4* in *Appendix C*).

With stop-line presence detectors, the variable initial green facility is not used. In general, for controllers which do not have a *variable initial green* facility, $G_{iv} = 0$, $G_{vmax} = 0$, and therefore, simply use:

$$G_{min} = G_{mf} \quad (B.4)$$

The *variable initial green* time is an additional variable period determined by the number of vehicle actuations (after the first vehicle) during the red period. *Vehicle increment* (e_i) and *maximum initial green* (G_{vmax}) settings are used in relation to this.

The values of vehicle increment settings used in Australian practice are in the range $e_i = 0.8$ to 4.0 s, and the maximum initial green settings are in the range $G_{vmax} = 8.0$ to 14.0 s.

The sum of the *fixed minimum green time* and *maximum initial green time* must be long enough to clear the vehicles waiting in the *critical lane* between the detection point and the stop line (i.e. within the setback distance).

It may be assumed that each vehicle between the detection point and the stop line in the *critical lane* will take h_s seconds to clear after a start loss time of l_s seconds. Thus:

$$G_{mf} + G_{vmax} = l_s + h_s n_d \quad (B.5)$$

where

l_s = start loss in seconds (typically 2 or 3 s),

h_s = saturation headway for a single-lane traffic stream in seconds: $h_s = 1 / s_c$ where s_c is the saturation flow rate in veh/s; e.g. $s_c = 1800$ veh/h = 0.5 veh/s gives $h_s = 2.0$ s,

n_d = average number of vehicles between the detection point and the stop line in one lane:

$$n_d = L_d / L_{hj} \quad (B.6)$$

L_d = distance between the detection point and the stop line (setback distance), and

L_{hj} = average spacing in queue (typically 6 m/car).

The *maximum initial green setting* can therefore be calculated from:

$$G_{vmax} = max \{ [(l_s + h_s n_d) - G_{mf}], 0\} \quad (B.7)$$

The *vehicle increment* setting can be calculated from:

$$e_i = \frac{G_{vmax}}{(n_d / p_c) - 1} \quad subject\ to\ n_d > 1 \quad (B.8)$$

where p_c is the proportion of total flow in the critical lane considering multi-lane cases where detections in more than one lane (in one or more approaches) contribute to the length of the variable initial green period. Through the use of this parameter, the formula allows for unequal lane flows.

In *Equation (B.8)*, $[(n_d / p_c) - 1)]$ is the total number of vehicles, excluding the first vehicle, that are stored between the detection point and the stop line in all lanes when the queue in the critical lane equals n_d. For equal lane utilisation and equal lane saturation flows, $p_c = 1 / n_l$, therefore $n_d / p_c = n_l n_d$, where n_l is the number of lanes (see *Section B.2*). For the case of a single lane:

$$e_i = \frac{G_{vmax}}{n_d - 1} \quad subject\ to\ n_d > 1 \quad (B.9)$$

For given flow conditions, the actual minimum green time as defined by *Equation (B.2)*, will depend on the value of *variable initial green* time which can be less than the maximum value for low arrival flows:

$$G_{iv} = min \{G_{vmax}, max [0, e_i (q_a r - 1)]\} \quad (B.10)$$

where

q_{at} = total arrival flow in all lanes during the red period (veh/s),

r = the red time (s).

The formula will give $G_{iv} = 0$ for very low arrival flows and $G_{iv} = G_{vmax}$ for high arrival flows.

Example

Given: $G_{mf} = 5$ s, $L_d = 30$ m, $L_j = 6$ m, $r = 30$ s, total arrival flow $q_{at} = 1500$ veh/h = 0.417 veh/s in $n_l = 3$ lanes with equal saturation flows, $s_c = 1800$ veh/h and $h_s = 2$ seconds, and $l_s = 3$ s. Find the appropriate *maximum initial green* and *vehicle increment* settings, and determine the values of *variable initial green time* and *minimum green time* for:

(i) equal lane flows: $q_{a1} = q_{a2} = q_{a3} = 500$ veh/h,

(ii) unequal lane flows: $q_{a1} = 300$ veh/h, $q_{a2} = 750$ veh/h and $q_{a3} = 450$ veh/h, and

(iii) total flow in one lane: $q_{a1} = 0$, $q_{a2} = 1500$ veh/h and $q_{a3} = 0$.

Case (i): From *Equation (B.6)*, $n_d = 30 / 6 = 5$ veh/lane. From *Equation (B.7)*, the maximum initial green is $G_{vmax} = (3 + 2 \times 5) - 5 = 8$ s. Since all lane flows are equal, $p_c = 1 / n_l = 1/3$ and $n_d / p_c = 3n_d = 15$ veh. From *Equation (B.8)*, the vehicle increment is $e_i = 8 / (15 - 1) \cong 0.6$ s. From *Equation (B.10)*, $e_i (q_{at} r - 1) = 0.6 (0.417 \times 30 - 1) = 0.6 \times 11.5 = 6.9$ s $< G_{vmax} = 8$ s, therefore $G_{iv} = 6.9$ s. From *Equation (B.2)*, $G_{min} = 5 + 6.9 = 11.9$ s. The minimum green is sufficient to clear all vehicles queued between the detection point and the stop line during the red period (in each lane: $q r = (500/3600) \times 30 = 4.2$ veh $< n_d = 5$ veh).

Case (ii): Lane 2 is the critical lane and $p_c = p_2 = 750/1500 = 0.5$, $e_i = 8 / [(5 / 0.5) - 1] = 0.9$ s, $e_i (q_{at} r - 1) = 0.9 \times 11.5 = 10.4$ s $> G_{vmax} = 8$ s, therefore $G_{iv} = G_{vmax} = 8$ s, and $G_{min} = 5 + 8 = 13$ s. In the critical lane, $q_{a2} r = (750/3600) \times 30 = 6.3$ veh are queued during the red period and g_{min} is sufficient to clear only $n_d = 5$ veh stored between detection point and the stop line as indicated by *Equation (B.5)*.

Case (iii): $p_c = p_2 = 1.0$, $n_d / p_c = n_d = 5$ veh and $e_i = 8 / (5 - 1) = 2.0$ s . Since $e_i (q_{at} r - 1) = 2.0 \times 11.5 = 23$ s $> G_{vmax} = 8$ s, $G_{iv} = 8$ s and $G_{min} = 5 + 8 = 13$ s. As in Case (ii), G_{min} is sufficient to clear only $n_d = 5$ vehicles.

B.4 Maximum Green Setting

Maximum green time is very important in determining the performance of traffic under vehicle-actuated signal control. Traditionally, the green time calculation methods for fixed-time signals are used for determining suitable maximum green settings. The *practical* signal timing method (Akçelik 1981, 1990) based on the use of a target (highest acceptable) degree of saturation (x_p) can be used for this purpose (see *Section 4.7*). This method can be adopted in a simple way as follows. To simplify the method, the calculations are carried out using the displayed rather than effective green and red times.

Calculate the required maximum green time (sum of the minimum green and the maximum extension green times) for a movement from:

$$G_{max} = \frac{y R_{max}}{x_p - y} \qquad \text{(B.11)}$$

where

y = flow ratio for the movement or the critical lane, $y = q_{at} / s_t = q_{ac} / s_c$, where q_{at} is the total arrival (demand) flow rate during the signal cycle for the movement and s_t is the saturation flow rate for the movement (all lanes), q_{ac} is the average arrival (demand) flow rate during the signal cycle in the critical lane and s_c is the saturation flow rate for the critical lane,

R_{max} = maximum red time that is acceptable to drivers (e.g. 60 seconds), and

x_p = practical (target) degree of saturation.

If $G = G_{max}$ and $R = R_{max}$ were to result in practice, this would mean $C_{max} = R_{max} + G_{max}$ and $x = (q_c C_{max}) / (s_c G_{max}) = x_p$ would be obtained.

Calculate the required maximum extension green time from:

$$G_{emax} = G_{max} - G_{min} \quad \text{(B.12)}$$
$$\textit{subject to a } G_{emax} \geq G_{emax\_min}$$

An appropriate minimum value for the maximum extension green time (G_{emax_min}) could be in the range 10 to 20 seconds. This helps to avoid low G_{emax} values that can be obtained with low flow rates (small y values). In this case, the smallest value of the maximum green time would be $G_{max\_min} = G_{min} + G_{emax\_min}$.

The value of the *maximum green setting* to be used in practice needs to be chosen with due consideration to traffic flows at different times (morning peak, evening peak, day off-peak, night off-peak, weekend and shopping periods) and to the peaking characteristics of traffic. This relates to the choice of the *design period* as a basis of calculating the maximum green setting. The objective should be to obtain green times which are not too restrictive for maximum possible flow rates (e.g. during a peak 15-minute period). On the other hand, long maximum settings coupled with a bad choice of the gap and other controller settings can lead to unduly long green and cycle times resulting in inefficient operation during a larger proportion of the time.

The typical values of maximum extension green settings used in Australian practice are in the range G_{emax} = 5.0 to 60.0 s (see *Tables C.3 and C.4* in *Appendix C*).

Example

Given: Total flow of q_{at} = 2160 veh/h in 3 lanes, movement saturation flow s_t = 4320 veh/h, critical lane saturation flow s_c = 1800 veh/h, and G_{min} = 10 s. Find the appropriate maximum extension green setting (use R_{max} = 60 s, x_p = 0.95).

Movement flow ratio can be found as y = 2160 / 4320 = 0.500. Alternatively, from *Equation (B.1)*, the proportion of flow in the critical lane is $p_c = s_c / s_t$ = 1800 / 4320 = 0.417, and the flow in the critical lane is $q_c = p_c\, q_{at}$ = 0.417 x 2160 = 900 veh/h. Therefore, y_c = 900/1800 = 0.500 = y.

From *Equation (B.11)*, G_{max} = 0.500 x 60 / (0.95 – 0.50) = 67 s. From *Equation (B.12)*, G_{emax} = 67 – 10 = 57 s.

B.5 Gap Setting

This is the most important setting that determines the duration of the extension green time. Importantly, the meaning of the gap setting specified in the controller should be understood clearly. The possible gap setting definitions are:

(i) as a *space time* value, e_s (for presence detection),

(ii) as a *gap time* value, e_g (for passage detection), and

(iii) as a *headway* value, e_h (for passage or presence detection).

In the analytical formulations given in this report for determining the average extension time, the gap setting is always used as a headway value (e_h). In cases (i) and (ii), the controller gap settings should be converted to the headway value for the purpose of the estimation process:

$$e_h = e_s + t_{ou} \quad \textit{in the case of presence detection}, \text{ or} \qquad (B.13)$$

$$e_h = e_g + t_{vu} \quad \textit{in the case of passage detection}$$

where t_{ou} and t_{vu} are the vehicle occupancy and passage times during the unsaturated part of the green period, respectively.

To ensure that there is no gap change during the saturated part of green period (i.e. during queue clearance), e_h should be larger than the saturation headway ($h_s = 1 / s_c$) for the critical lane. For example, $e_h > 2.0$ s should be used for $s_c = 1800$ veh/h = 0.5 s ($h_s = 2.0$ s). Using $t_{ou} = 0.5$ s and $t_{vu} = 0.2$ s, the corresponding gap settings as space time and gap time values should be $e_s > 1.5$ s and $e_g > 1.8$ s.

The values of gap settings used in Australian practice are in the range $e_s = 2.0$ to 4.0 s as space time, corresponding to approximately $e_h = 2.5$ to 4.5 s as headway.

B.6 Headway and Waste Settings

Headway and waste settings are related to the waste change method used in the Australian VA controllers. This method provides a type of *volume-density* control which aims to terminate the extension green interval before gap change if the headways are too small for a gap change but too large for efficient traffic operation(see *Sections 2 and 4.3*). The efficiency is measured by difference of headways from the headway setting which is related to saturation headway, (or with presence detection, to a saturation space time). The difference is called a waste increment. The phase terminates when the total waste increment before a gap change equals the waste setting. Therefore, appropriate values of the headway and waste settings need to be determined together.

The typical values of headway settings used in Australian practice are in the range t_h = 0.3 to 2.0 s, and the waste settings are in the range w_s = 3 to 20 s (see *Tables C.3 and C.4* in *Appendix C*).

The recommended method for determining the headway and waste settings is described below. More research is needed to determine the optimum values of these settings.

Headway Setting

The method given here is closer to the method described in RTA (1991) than the method given in Section B.10 of AUSTROADS (1993). The latter guide recommends that the occupancy time for use in the calculation of the headway setting for the controller is calculated by assuming that "the detector zone length is taken as three times the detector length, i.e. 3 x 4.5 = 13.5 m, then occupancy time =13.5 / speed (m/sec)". This is not consistent with the basic relationships described in *Appendix A* of this report, and is not confirmed by any equation given in Section A.7 of the AUSTROADS guide.

With presence detection, the *headway setting* (t_h) as a *space time* value can be calculated from:

$$t_h = p_c f_s h_s - t_{ou} \qquad \text{(B.14)}$$

subject to $t_h > 1.0$ s for single-lane case, and
$t_h > 0.3$ s for the multi-lane case

where

p_c = proportion of total flow in the critical lane ($p_c = q_{ac} / q_{at}$ where q_{ac} = arrival flow in the critical lane, q_{at} = total arrival flow in all lanes),

h_s = saturation headway (seconds) in the critical lane ($h_s = 1 / s_c$ where s_c = saturation flow in the critical lane in veh/second),

f_s = a factor to increase the saturation headway (or decrease the saturation flow), and

t_{ou} = occupancy time during the unsaturated part of the green period (seconds) as relevant to the analysis of waste change before gap change after queue clearance: $t_{ou} = (L_p + L_v) / v_u$ where v_u is the average speed during the unsaturated part of the green period (see *Appendix A*).

In current Australian guides, f_s = 1.25 is used "to ensure that the headway setting will be appropriate for flows in the range 0.8 to 1.25 times the saturation flow". For reasons explained below, the following values of f_s are recommended:

$$f_s = \begin{cases} 1.15 & \textit{for single-lane case} \\ 1.50 & \textit{for multi-lane case} \end{cases} \qquad \text{(B.14a)}$$

Further investigation to establish appropriate values of this factor is necessary.

Since $p_c = s_c / s_t$ from *Equation (B.1)*, $p_c h_s = 1 / s_t = h_t$ = saturation headway for all lanes of the movement allowing for different lane saturation flows and unequal lane utilisation (see *Section B.2*):

$$t_h = \frac{f_s}{s_t} - t_{ou} = f_s h_t - t_{ou} \qquad \text{(B.15)}$$

subject to $t_h > 1.0$ s for single-lane case, and $t_h > 0.3$ s for the multi-lane case

Note that the occupancy value in this formula is calculated as a single lane value (this differs from the current guides).

For a single lane, $s_t = s_c$ and $p_c = 1.0$, therefore $t_h = (f_s / s_c) - t_{ou}$. For the multi-lane case with equal lane utilisation and equal lane saturation flows, $p_c = 1 / n_l$ where n_l is the number of lanes ($s_t = n_l s_c$).

For a single-lane, a smaller value of factor f_s is recommended in order to obtain a less *slack* operation (e.g. 1.15 rather than 1.25). This could be compensated for by increasing the waste setting (see *Equation B.14a*).

If a headway setting derived for the multi-lane case is used for a single-lane movement, waste increments would be accumulated during queue clearance, and therefore a waste change would be very likely to occur before queue clearance unless the waste setting is large (see *Section 4.3*). Therefore, larger values of the headway setting than used in the current practice are advisable for single lane movements (see the example below).

Considering detector actuations across all lanes in the case of a multi-lane movement, it is possible for the space times to exceed the space time based on the average saturation headway for all lanes, depending on the way vehicles in individual lanes are spaced relative to each other. This could result in waste accumulation during queue clearance i.e. when traffic operates in an efficient (saturated) way. For this reason, the use of a larger value of the f_s factor would be useful, and $f_s = 1.50$ is recommended (see *Equation B.14a*).

In the formulae to determine the extension time by waste change given in the main text of this report, the headway setting is used as a headway value rather than a space time value, $h_o = t_h + t_{ou}$ (however, the space time during the saturated part of the green period is found as $t_{ss} = h_s - t_{os}$ where t_{os} is the occupancy time calculated using the saturation speed, v_s). The formulae require that h_o should be greater than the minimum headway for the arrival distribution ($h_o > \Delta$). This means that $t_h > \Delta - t_{ou}$. For the single-lane case, $\Delta = 1.5$, and using $t_{ou} = 0.5$ s, $t_h > 1.0$ is required. For the multi-lane case, $\Delta = 0.5$, and using $t_{ou} = 0.5$ s, $t_h > 0.0$ would not cause a problem, but a minimum value of 0.3 s has been nominated. These are indicated in *Equations (B. 14) and (B.15)*. However, this consideration should be applied to special circumstances with care using appropriate occupancy time values depending on traffic speed, detection zone length and vehicle length.

Example

For a single-lane movement with s_c = 1800 veh/h = 0.5 veh/s (therefore the saturation headway h_s = 1 / 0.5 = 2.0 s), L_p = 4.5 m, L_v = 4 m and v_u = 60 km/h = 16.7 m/s during queue discharge, the occupancy time is t_{ou} = (4.5 + 4) / 16.7 ≈ 0.5 s, and the headway setting from *Equation (B.14)* is t_h = 1.15 x 2.0 – 0.5 = 1.8 s.

For a multi-lane case with n_l = 3, equal lane flows and a total saturation flow of s_t = 3 x 1800 = 5400 veh/h = 1.50 veh/s (h_t = 1 / 1.50 = 0.67, p_c = 1/3), *Equation (B.15)* will give t_h = 1.50 x 0.67 – 0.5 ≈ 0.5 s. If the saturation flow is s = 4000 veh/h = 1.1 veh/s allowing for unequal saturation flows, h_t = 1 / 1.1 = 0.9 s and t_h = 1.50 x 0.9 – 0.5 ≈ 0.9 s is found.

Waste Setting

Both AUSTROADS (1993) and RTA (1991) recommend that waste setting is determined as:

$$w_s = 0.1\, G_{max} \quad \textit{subject to } w_{smin} \le ws \le w_{smax} \tag{B.16}$$

where w_{smin} = 4 seconds (a minimum value) and w_{smax} = 10 seconds (a maximum value).

B.7 Pedestrian Settings

There are various methods in use for determining minimum walk and clearance times for pedestrian movements. The method used in the SIDRA package (Akçelik 1990) is given here. This is similar to the method described in ARR No. 123, Appendix B (Akçelik 1981), and is consistent with the method described in AUSTROADS (1993) and RTA (1991).

For better estimates of pedestrian performance characteristics (delay, etc.), the pedestrian clearance time (Flashing Don't Walk) is specified as a *negative end gain* (i.e. effective loss). This results in an effective green time for the pedestrian movement which is approximately equal to the walk time. Some usage of the clearance period by pedestrians is assumed in this process.

The relationship between the walk, clearance and green times for a pedestrian movement and the phase green time for a concurrent vehicle movement are shown in *Figure B.1*.

The formulae for calculating the clearance time, minimum pedestrian movement time and effective green time for a pedestrian movement are given below. Recommended values of the parameters for use in the formulae (as used in the SIDRA package) are summarised in *Table B.1*.

The *total clearance time* (duration of the Flashing Don't Walk interval), t_{pc} can be calculated from:

$$t_{pc} = max\left(\frac{L_w}{v_p}, t_{pcm}\right) \tag{B.17}$$

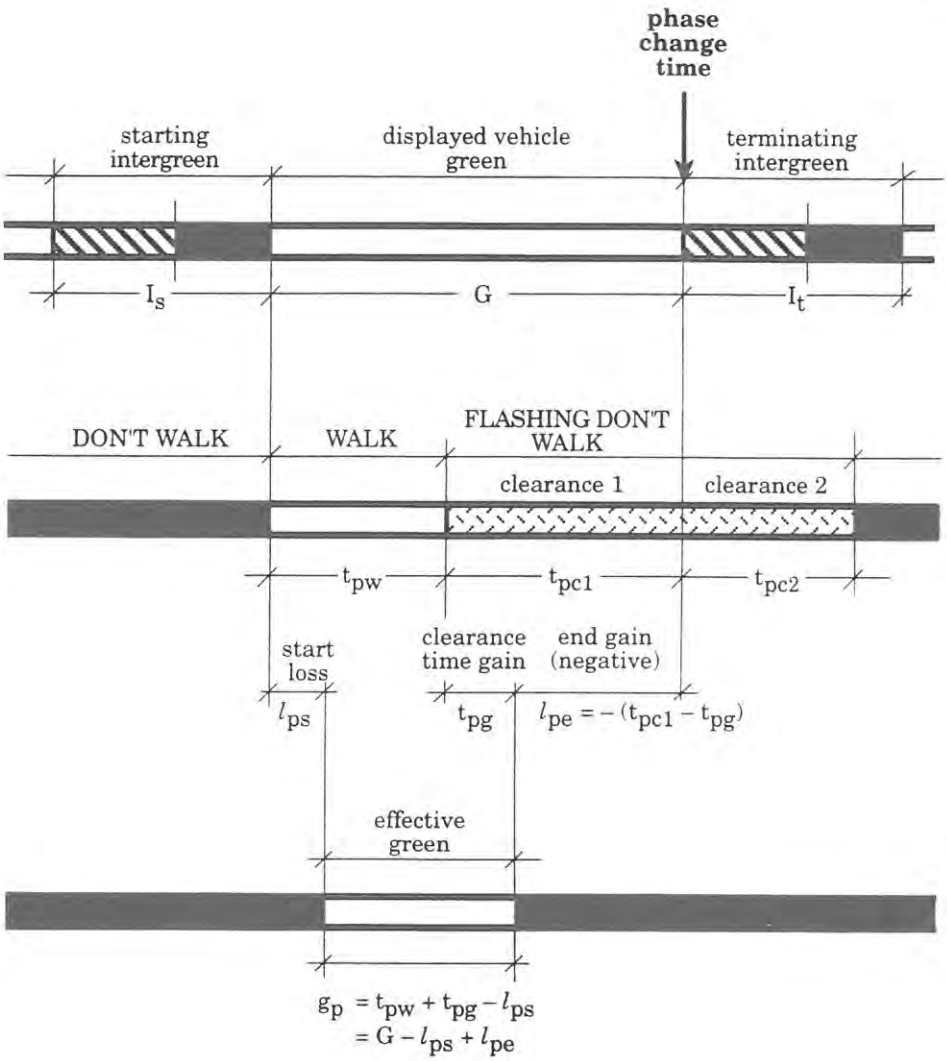


Fig. B.1 – Walk, clearance and effective green times for pedestrian movements

Table B.1

Recommended parameter values for calculating pedestrian timing data

	SIDRA standard	US HCM standard
Minimum clearance time, t_{pcm}	5 s	5 s
Clearance (walking) speed, v_p	1.2 m/s	4.0 ft/s
Minimum walk time, t_{pwm}	6 s	7 s
Clearance 2 time, t_{pc2}	2 s	3 s
Clearance time gain, t_{pg}	3 s	3 s
Start loss, l_{ps}	1 s	1 s

where

t_{pc} = total clearance time (in seconds),

t_{pcm} = minimum clearance time (in seconds),

L_w = clearance distance (in metres), i.e. an appropriate pedestrian crossing distance determined according to full crossing or staged crossing), and

v_p = clearance speed (in metres per second).

The total clearance time consists of clearance intervals 1 and 2 ($t_{pc} = t_{pc1} + t_{pc2}$). Clearance 1 time which excludes any overlap with the terminating intergreen time can therefore be calculated from:

$$t_{pc1} = t_{pc} - t_{pc2} \quad \text{(B.18)}$$

Clearance 2 time should not exceed the terminating intergreen time (plus early cut-off time where applicable).

The *effective green time* for a pedestrian movement for use in performance calculations can be calculated from:

$$g_p = t_{pw} + t_{pg} - l_{ps} \quad \text{(B.19)}$$
$$= G - l_{ps} + l_{pe}$$

where

t_{pw} = pedestrian walk time (sum of the durations of walk 1 and walk 2 intervals) (in seconds),

l_{ps} = pedestrian start loss, i.e. the unused initial part of pedestrian walk interval (seconds),

G = phase green time for the concurrent vehicle movement (seconds), and

l_{pe} = the negative end gain for pedestrian movement, i.e. the unused part of clearance 1 interval (seconds) calculated from

$$l_{pe} = -(t_{pc1} - t_{pg}) \quad \text{(B.20)}$$

where

t_{pc1} = clearance 1 time as calculated above, and

t_{pg} = clearance time gain (the first part of the clearance 1 interval which is used by pedestrians).

If the pedestrian walk time is extended beyond the minimum value to match the concurrent vehicle phase green time (G), the walk time would be equal to:

$$t_{pw} = G - t_{pc1} \quad \text{(B.21)}$$

In this case, walk 2 time is given by:

$$t_{pw2} = t_{pw} - t_{pwm} \quad \text{(B.22)}$$

where walk 1 time equals the minimum walk time, $t_{pw1} = t_{pwm}$.

The concept of *pedestrian minimum time*, G_{pmin}, is useful in terms of the minimum green time imposed on concurrent vehicle movements. This is given by:

$$G_{pmin} = t_{pwm} + t_{pc1} \quad \text{(B.23)}$$

where t_{pwm} is the minimum walk time, i.e. duration of walk 1 interval (seconds), and t_{pc1} is clearance 1 time (seconds).

Example

Calculate the pedestrian minimum green time and end gain values for a crossing distance of L_w = 17 m using the recommended parameter values (SIDRA standard) given in *Table B.1*. If the phase green time is G = 40 seconds, what are the values of walk time and effective green for the pedestrian movement?

The total clearance time is $t_{pc} = max\ (17/1.2, 5) = 14.2$ s. Clearance 1 time is $t_{pc1} = 14.2 - 2 = 12.2$. The pedestrian minimum time is $G_{pmin} = 6 + 12.2 = 18.2$ s.

The negative end gain is $l_{pe} = -(12.2 - 3) = -9.2$ s. The effective green time for the pedestrian movement is $g_p = G - l_{ps} + l_{pe} = 40 - 1 - 9.2 = 29.8$ s. The corresponding pedestrian walk time is $t_{pw} = 40 - 12.2 = 27.8$, and using minimum pedestrian walk time $t_{pwm} = t_{pw1} = 6$ s, walk 2 time is $t_{pw2} = 27.8 - 6 = 21.8$ s.

APPENDIX C

Survey of Australian Vehicle-Actuated Control Practice

A survey of the Australian vehicle-actuated control practice was carried out as part of this research in order to establish the range of operational configurations in use, to identify the timing design and analysis procedures that are commonly employed in practice, and to determine the issues related to VA control that should be considered in the current and future research. Some data compiled through this survey are presented in various tables. A copy of the survey form is also given.

Notes

1. The following abbreviations are used in the tables:

RTA NSW	Roads and Traffic Authority of New South Wales
VIC Roads	Roads Corporation of Victoria
QLD Transp	Queensland Department of Transport
Brisbane City	Brisbane City, Queensland
MRWA	Main Roads Western Australia
DoT SA	Department of Transport South Australia
Adelaide City	City of Adelaide, South Australia
DTW TAS	Department of Transport & Works, Tasmania
ACT DUS	Australian Capital Territory, Department of Urban Services
DTW NT	Department of Transport & Works, Northern Territory

   These authorities are not necessarily responsible for all the signals reported under their name since there are many different local authorities who own the signals in their areas.

   The results for Queensland Transport are based on responses received from the Metropolitan Traffic Branch, Traffic Signals and Electrical Section, and the South East-Hinterland District.

2. Huddart (1980) reported that there were 3160 signalised intersections in Australia in 1978. As seen in *Table C.1*, the total number of signalised intersections in Australia is now about 7110, indicating an increase of 125 per cent during the last seventeen years.

3. In *Table C.1*, *SCATS Master Isolated* is a special mode of VA operation that uses ***SCATS*** algorithms, and is different from traditional isolated VA operation discussed in this report. In practice, the term *SCATS Isolated* is used for sites linked to a SCATS system but running as traditional isolated VA. Such sites are included in the *Isolated VA* category in *Table C.1*. RTA NSW could not identify the number of sites running in *SCATS Master Isolated* mode due to the flexibility of arrangements regarding this.

3. In *Table C.1, Other Coordinated* includes the following:

   RTA NSW: 7 sites under cableless link, and 10 sites under sister link system.

   VIC Roads: 18 sites under flexilink system.

   QLD Transp: 535 sites in Queensland run in coordinated mode under the TRACS control system; there are also 10 sites under flexilink system (6 of these are included in the isolated site count).

   Brisbane City : 626 sites are under the BLISS coordinated control system.

   Adelaide City : 68 sites under flexilink system.

   DTW TAS: 6 sites under flexilink system.

   TRACS and ***BLISS*** systems used in Queensland control about a quarter of coordinated intersections in Australia while about three quarters of coordinated intersections are under ***SCATS*** control.

4. In *Table C.1, Other Isolated* includes 6 sites under Queensland Transport which are the only fixed-time isolated signalised intersections reported.

5. Other facilities not included in *Table C.1*:

   DoT SA: 5 sites (emergency signals), 44 sites part-time school crossings (flashing yellow signals only).

   DTW TAS: 2 sites ("Wig Wag") emergency signals and 11 slip lane crossings.

   Not all emergency signals in use by other States have been reported.

Table C.1 – Number of Signalised Intersections, Signalised Pedestrian Crossings and Roundabouts with Signals

	RTA NSW	VIC Roads	QLD Transp	Brisbane City	MRWA	DoT SA	Adelaide City	DTW TAS	ACT DUS	DTW NT	TOTAL
Signalised Intersections											
Total	2363	1758	854	650	606	379	105	205	149	28*	7097
Isolated VA	566	462	309	24	423	30	15	122	74	5	2030
SCATS Master Isolated		200			2	9	2		5	1	219 + RTA
Other Isolated			6								6
SCATS Coordinated	1780	1078			181	340	20	77	70	22	3568
Other Coordinated	17	18	539	626			68	6			1274
Signalised Pedestrian Crossings (categories overlap)											
Total	505	925	117	106	23	201	7	39	24	7	1954
Pedestrian actuated	314	925	115	106	23	201	7	28	22	7	1748
Vehicle actuated:		900	87	106	23			25	17	7	1165
Coordinated	166	384	46	90	5	150	4	3	2		850
Pelican	25	2			23						50
Roundabouts with part-time or full-time signals on one or more legs											
	4	8	3		1	1		1			18

* In Northern Territory, there are 10 more installations under the Darwin and Alice Springs City Councils.

Table C.2 – Controller Types and Detection

	RTA NSW	VIC Roads	QLD Transp	Brisbane City	MRWA	DoT SA	Adelaide City	DTW TAS	ACT DUS	DTW NT
Most common make and type of controller	Philips PSC	Philips PSC Philips PTF AWA Delta 3	Philips PTF Philips PSC	Philips PSC	Philips PTF Philips PSC	Philips PTF Philips PSC	AWA Delta 3	Philips PTF Philips PSC	AWA Delta 3	Philips PTF
Most common type of detection	Presence	Presence	Presence	Presence	Presence	Presence	Presence	Presence	Presence	Presence
Typical detector loop length	4.5 m	4.5 m	1.8 - 3.0 m	4.5 m	4.5 m	4.0 m	4.0 m	4.0 m	4.5 m	4.5 m
Detector location (setback distance)	1.5 m	1.5 m	30-36 m on major rd 6 m or stop-line on minor rd	1.5 m	1.5 m	0.3 m	0.3 m	Stopline and advance *	1.5 m	1.50m
Slip lane detection	For traffic counting only	Very rare	For traffic counting only	For traffic counting only	No	For traffic counting only	No	For traffic counting only	For traffic counting only	For traffic counting only

* Stopline detectors in SCATS areas, and advance detectors for isolated sites (35 m in urban areas, 60-100 m on highways)

Table C.3 – Typical Controller Settings for Through Vehicle Phases

	RTA NSW	VIC Roads	QLD Transp	Brisbane City	MRWA	DoT SA	Adelaide City	DTW TAS	ACT DUS
Gap setting	3-4 (10)	2.5 (4.0)	3-4 (8)	3 (5)	3 (4)	3	2.5-3.0 (4.0)	3.5 (6.0)	4 (5)
Headway setting	0.8-1.2 (5.0)	0.3-1.2 (1.6)	1-2 (5)	1 (2)	1 (1.5)	1	0.8-1.0 (4.0)	1.0 (2.0)	1.0-1.5 (1.5)
Waste setting	4-10 (50)	7.0 (15.0)	10-20 (25)	0.1 Gmax (20)	4 (10)	11 (20 - 50)	10-11 (30)	6 (12)	5 (10)
Minimum (initial) green	4-5 (20)	8-10 (12)	6-10 (20)	6 (12)	6-8 (10)	5	6-10 (15)	8 (14)	8 (8)
Vehicle increment (variable initial green)	2.0 (5.0)		2-4 (4)	0.5 (1)			0.8-1.0 (2.0)	1 (1.5)	
Maximum initial green	0 (40)		12-14 (20)	depends on det. setback				8 (16)	
Maximum extension green time		10-60 (80)	30-60 (80)		5-20 (100)			30 (70)	30 (50)
Maximum green time				60 (120)		20-90 (90)	30-50 (90)		
Presence time	2-3 (5)		3	3 (5)		0	0 (5)		
Late start	2-5 (20)	0	0-6 (12)	6 (8)		0	0 (0)	2 (3)	4 (8)
Early cut-off green	0 (20)	0	0-6 (6)	2 (6)	5-8	0			3 (4)
Yellow	4 (6.4)	4.0-5.0 (5.0)	4-5 (5)	4 (2)	4 (5)	4-6 (6)	3-4 (4)	3 (5)	4 (5)
All-red	1-2 (15)	0.5-3.0 (5.0)	1-5 (5)	2 (5)	1-2 (5)	2 (3)	2 (4)	1 (2)	2 (4)

Values are in seconds (maximum values in brackets).

Table C.4 – Typical Controller Settings for Right-Turn Vehicle Phases

	RTA NSW	VIC Roads	QLD Transp	Brisbane City	MRWA	DoT SA	Adelaide City	DTW TAS	ACT DUS
Gap setting	2-3	2.5 (4.0)	2.5-4 (4)	3 (3)		3		2 (2.5)	3 (4)
Headway setting	1.0-1.2	0.6-1.2 (1.2)	1 (5)	1 (2)	0.2*	1			1.0 (1.2)
Waste setting	3-4	7.0 (15.0)	1-15 (15)	0.1 Gmax (20)	2-3 (4)	8 (11)			4 (6)
Minimum (initial) green	4	6 (12)	6-8 (10)	6 (8)		5			6 (8)
Vehicle increment (variable initial green)				0.5 (1)					
Maximum initial green	0			depends on det. setback					
Maximum extension green time		5-25 (40)	20-40 (60)					15 (20)	12 (20)
Maximum green time				30 (60)		30			
Presence time		3.0-5.0 (5.0)	0-3 (3)	3 (5)	3 (5)	3		2 (3)	2 (3)
Late start		0		6 (8)	1-4 (4)	2			
Early cut-off green		0		2 (6)		0			
Yellow	3-4	3.0 (3.5)	4-5 (5)	4 (2)		3-6		3 (3)	4 (4)
All-red		0.5-3.0 (4.0)	1-5 (5)	2 (5)				0.5 (1.0)	2 (4)

Values are in seconds
(maximum values in brackets).
* Headway time for 11.0 m loop for shared lanes

Table C.5 – Typical Controller Settings for Pedestrian Movements

	RTA NSW	VIC Roads	QLD Transp	Brisbane City	MRWA	DoT SA	Adelaide City	DTW TAS	ACT DUS
Walk	6 (40)	8-16 (16)	6-14 (17)	8	6 (10)	5	6-8 (21)	8 (10)	6-12 (14)
Flashing Don't Walk	14-20 (50)	6-16 (20)	10-20 (35)	$\frac{\text{Distance}}{1.2\ \text{m/s}}$	16-20 (30)	$\frac{\text{Distance}}{1.2\ \text{m/s}}$	12-16 (24)		6-8 (14)
Clearance 1	8-14 (40)		6-16 (31)	$\frac{\text{Distance}}{1.2\ \text{m/s}}$	13-16 (26)	FDW – C2	2-16 (24)	8 (16)	4 (6)
Clearance 2	6 (10)	Rarely used	4-5 (5)	4 (5)	3-4	Yellow + All-red	0-3 (5)	10 (10)	

Values are in seconds (maximum values in brackets).

Table C.6 – Methods Used for Determining VA Controller Settings and for Capacity and Performance Analysis

	RTA NSW	VIC Roads	QLD Transp	Brisbane City	MRWA	DoT SA	Adelaide City	DTW TAS	ACT DUS	DTW NT
VA controller settings	RTA Manual RTA-TC-106, experience	Departmental rules and engineering judgement	AUSTROADS TEP Part 7, Departmental practices, local experience and on-site tuning	AUSTROADS TEP Part 7	AUSTROADS TEP Part 7 and on-site tuning	AUSTROADS TEP Part 7	AUSTROADS TEP Part 7	Departmental guidelines based on AUSTROADS and ARR 123, and on-site tuning	RTA NSW, AUSTROADS and ACT practice *	RTA standards and SIDRA
Capacity and performance analysis:										
ISOLATED	SIMSET or INTANAL	SIDRA and manual	SIDRA and manual	SIDRA	SIDRA and manual	SIDRA and SCATES	SIDRA and manual	SIDRA and on-site checks	SIDRA and manual	SIDRA
COORDINATED	None	SCATES and manual	TRANSYT and manual	TRANSYT	SCATES	SCATES	SCATES and manual	SCATES	TRANSYT and manual	SCATES

* Manual and computer calculations for maximum timings.

ARRB Project TE 074
Performance of Intersections Controlled by Vehicle-Actuated Signals

Survey of Vehicle-Actuated Control Practice

(approximate numbers are sufficient where numerical values are requested)

Reporting Agency: .....
Contact Person: .....
Title: .....
Address: .....
..... Post code: .....
Phone: ..... Fax: .....

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1. *How many* signal controllers do you have under your authority's responsibility?

Where applicable, enter **NA** = information not available, **None** = facility not in use.

1a. Signalised intersections

Total number: .....
Isolated Vehicle-Actuated (VA): .....
SCATS Master Isolated: .....
Fixed-Time Isolated: .....
Other: ..... Specify type: .....
SCATS Coordinated (local VA): .....
Other Coordinated:..... Specify type *: .....
.....

* Name and the type of the control system (plan selection method, etc.), whether local traffic actuation is allowed, and so on.

1b. Pedestrian Crossings (the categories may overlap)

Total number: .....
Pedestrian actuated: ..... Vehicle actuated: .....
Coordinated:.....
Pelican:..... Zebra crossing: .....
Other: ..... Specify type: .....

1c. Roundabouts with Signals

Number of roundabouts with part-time or full-time signals:
Signals on one leg only: ..... on more than one leg: .....

Comments or explanations related to answers 1a to 1c : .....
.....
.....
.....

Continued >>>

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2. What is the *most common make and type of controller* used in the area under your responsibility, and what percentage of controllers does it represent?

.....
.....

3. What are the most common type, size and location of *detectors*?
In addition to *typical* detection information, you may wish to include information about some unusual arrangements, e.g. queue detection in turn slots.

The type of detection (presence or passage): .....
.....

The typical location of detectors (distance from stop line): .....
.....

The typical size of detectors: .....
.....

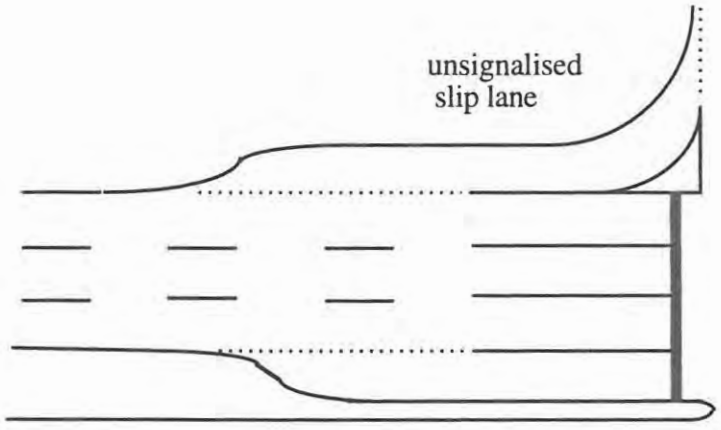
Effective detection zone length for timing purposes: .....
.....

Any special features of right-turn movement detection: .....
.....

Any special features of signalised left-turn movement detection: .....
.....

Detection of unsignalised left-turn movements in slip lanes: .....
.....

Other remarks: .....
.....
.....



Please show the location (distance from stop line) of all detectors (typical setup)

Continued >>>

4. What are the *typical settings* you use for the *traditional VA* controller for the most common type of controller as stated in question 2 above?

These will be an important source of information for determining the range of operating conditions to be covered by our current research. Indicate a typical range rather than a single value where necessary. For right-turn phases, give only values for features that are different from through phases

Typical values of controller settings	Through VEH. phases		Right-Turn VEH. Phases	
	Typical value(s)	Maximum value(s)	Typical value(s)	Maximum value(s)
Gap time*				
Headway time*				
Waste time*				
Minimum (initial) green				
Increment (for variable initial green)				
Maximum initial green				
Maximum extension time ☐ or Maximum green time ☐ (tick)				
Presence time				
Late start				
Early cut-off green				
Yellow				
All-red				

PEDESTRIANS		
	Typical value(s)	Maximum value(s)
Walk		
Flashing Don't Walk		
Clearance 1		
Clearance 2		

* Gap, headway and waste time settings are based on *space time* (headway less occupancy time) if presence detector is used.

Additional information:

Recall facility, coordinated VA operations, etc.?: .....

.....

Special/other features?: .....

.....

The information given in this page represent approximately ..... % of your intersections.

Continued >>>

5. What *method* do you use for determining appropriate values of the VA *controller settings* (as specified in question 4 above) for *traditional VA* controllers?

A set of rules developed by your Department, AUSTROADS Traffic Engineering Practice Part 7 (App. B), a spreadsheet or computer program, and so on.

.....
.....
.....
.....

6. What *method (s) and procedure(s)* do you use for the *capacity and performance* evaluation of intersections under VA control?

Isolated intersections: .....
(Manual, SIDRA, SIMSET, etc.)

Coordinated intersections: .....
(Manual, SCATES, TRANSYT, etc.)

Comments: .....
.....
.....
.....

7. In your opinion, what are the *issues* related to VA control that should be considered in the current and future research?

We would welcome your comments on what features and capabilities are most important in a traffic-actuated control analysis method, what are the more general related research issues, and so on.

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Thank you for your help.

Please forward the completed form to:

Rahmi Akçelik
Chief Scientist
Australian Road Research Board Ltd
500 Burwood Highway
VERMONT SOUTH VIC 3133

APPENDIX D

Calibration of the Bunched Exponential Distribution of Arrival Headways

This appendix presents a reprint of the article published in the March 1994 issue of ***the Road & Transport Research*** (Akçelik and Chung 1994a). The methods for calculating gap acceptance capacities and delays described in the paper have now been superseded by new methods adopted in SIDRA 4.1 (Akçelik and Chung 1994b).

Calibration of the bunched exponential distribution of arrival headways

R. Akçelik and E. Chung

ABSTRACT

The estimation of arrival headways is fundamental to the modelling of gap acceptance processes for estimating capacities of sign-controlled traffic streams, roundabout entry streams and filter turns at signalised intersections. It is also essential in modelling both vehicle-actuated signal timings and queueing at all types of intersections for performance prediction.

This paper considers a class of arrival headway distributions known as negative exponential, shifted negative exponential, and bunched exponential. A description of the bunched exponential arrival headway distribution is presented, and the results of its calibration using real-life data for single-lane traffic streams and simulation data for multi-lane streams are given. Examples of gap-acceptance capacities and delays predicted by different exponential headway distributions are also presented.

Although the bunched exponential distribution is relatively new and its use is less common, it has been found to be more realistic than the negative exponential and shifted negative exponential distributions. Its general use in traffic modelling is recommended.

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The estimation of arrival headways is fundamental to the modelling of all aspects of traffic. Important applications include modelling of:

(a) gap acceptance processes for estimating capacities of sign-controlled traffic streams, roundabout entry streams and filter turns at signalised intersections (e.g. Akçelik 1981, 1990; Akçelik and Troutbeck 1991; Troutbeck 1986, 1989, 1991);

(b) queueing at all types of intersections for predicting delays, queue lengths and stop rates (e.g. Akçelik 1981, 1990; Akçelik and Chung 1994);

(c) extension times for estimating green times and cycle time at vehicle-actuated signals (Akçelik 1994a *b*; Lin 1982*a* and 1982*b*); and

(d) uninterrupted traffic flows on freeways and rural roads — see May (1990) for a brief discussion.

This paper considers the following class of *exponential* arrival headway distribution models:

- Model 1 (M1): negative exponential;
- Model 2 (M2): *shifted* negative exponential; and
- Model 3 (M3): *bunched* exponential.

The simple negative and shifted negative exponential distributions are extensively discussed and used in the literature. On the other hand, the bunched exponential distribution is relatively new, and, while more realistic, its use is less common. In particular, the bunched exponential distribution offers improved accuracy in the prediction of small arrival headways (up to about 12 seconds), which is important for most urban traffic analysis applications.

A description of the bunched exponential arrival headway distribution is presented, as well as results of its calibration using real-life data for single-lane traffic streams and simulation data for multi-lane streams. Examples of gap-acceptance capacities and delays predicted by different exponential headway distributions are also presented.

BUNCHED EXPONENTIAL DISTRIBUTION

The bunched exponential distribution of arrival headways (M3) was proposed by Cowan (1975). It was used extensively by Troutbeck (e.g. 1986, 1989, 1991) for estimating capacity and performance of roundabouts and other unsignalised intersections. A special case of the model was previously used by Tanner (1962, 1967) for unsignalised intersection analysis. The M1 and M2 models can be derived as special cases of the M3 model through simplifying assumptions about the bunching characteristics of the arrival stream.

The M1 and M2 models are more commonly used in the traffic analysis literature as models of *random arrivals*. Alternative headway distribution models such as *Pearson Type III*, *Erlang*, *Gamma*, *lognormal* and various *composite* models (*exponential*, *hyperlang)* have been considered in the literature, to overcome the shortcomings of

models M1 and M2 in predicting headway probabilities for small headways and for high arrival flow rates. Useful references on arrival headway distributions for road traffic analysis include Ashton (1966), Drew (1968), Gerlough and Huber (1975), Greenshields and Weida (1978), Haight (1963) and May (1990).

The M3 model overcomes the shortcomings of the M1 and M2 models in representing real-life traffic characteristics, yet retains the simplicity of those models. In recent work, the M3 model was used successfully for deriving various formulae for the analysis of actuated signal operations (Akçelik 1993 and 1994). The model can be used consistently for all urban traffic analyses (gap acceptance modelling at signalised and unsignalised traffic facilities, modelling of traffic performance, etc.) to replace the M1 and M2 models.

The cumulative distribution function, F(t), for the bunched exponential distribution of arrival headways, representing the probability of a headway less than t seconds in a single-lane or multi-lane traffic stream, is given by:

$$F(t) = 1 - \varphi e^{-\lambda (t - \Delta)} \quad \text{for } t \geq \Delta \qquad (1)$$
$$= 0 \quad \text{for } t < \Delta$$

where:

Δ = minimum arrival (intra-bunch) headway (in secs);

φ = proportion of free (unbunched) vehicles, and

λ = a model parameter given by:

$$\lambda = \varphi q_t / (1 - \Delta q_t) \qquad (1a)$$

where q_t is the total arrival flow in all lanes of the traffic stream (in veh/s).

According to the model, the traffic stream consists of:

(a) *bunched* vehicles with all intra-bunch headways equal to the minimum arrival headway, Δ (the proportion of bunched vehicles = $1 - \varphi$); and

(b) *free* vehicles with headway greater than the minimum arrival headway, Δ (thus, the proportion of free vehicles, φ, represents the unbunched vehicles with randomly distributed headways).

It is emphasised that:

(i) Δ is a minimum *arrival* headway, and is not necessarily the same as h_s, the *saturation* headway ($h_s = 1/s$, where s = saturation flow or queue discharge rate); and

(ii) the bunched exponential model implies a geometric distribution of bunch sizes and an exponential distribution of gaps between bunches (Cowan 1975), and therefore does not represent platoons (bunches) of traffic formed at traffic signals.

The complementary function, $1 - F(t)$, representing the probability of a headway greater than or equal to t seconds, is given by:

$$1 - F(t) = \varphi e^{-\lambda (t - \Delta)} \quad \text{for } t \geq \Delta \qquad (2)$$
$$= 1 \quad \text{for } t < \Delta$$

The probability density function of arrival headways for the bunched exponential model, representing the probability of a headway of t seconds, is:

$$f(t) = \varphi \lambda e^{-\lambda (t - \Delta)} \quad \text{for } t > \Delta \qquad (3)$$
$$= 1 - \varphi \quad \text{for } t = \Delta$$
$$= 0 \quad \text{for } t < \Delta$$

M1 and M2 models

The M1 and M2 models can be derived from the M3 model by setting the bunching parameters as follows.

Negative exponential model (M1)

$$\Delta = 0 \text{ and } \varphi = 1 \text{ (therefore } \lambda = q_t \text{)} \qquad (4)$$

Shifted negative exponential model (M2)

$$\varphi = 1 \qquad (5)$$

Discussion

Thus, models M1 and M2 assume no bunching for all levels of arrival flows. On the other hand, model M3 can be used either with a known (measured) value of φ, or more generally, with a value of φ estimated as a function of the arrival flow rate.

M3A and M3T models

The following relationship was derived as a general formula for estimating the proportion of free vehicles in the traffic stream (φ) by generalising the bunching *implied* by the negative exponential model (Akçelik 1994a,b):

$$\varphi = e^{-b \Delta q} \qquad (6)$$

where b is a bunching factor, Δ is the minimum arrival (intra-bunch) headway, and q is the arrival flow rate (in veh/s). An empirical relationship of a similar form ($\varphi = \exp(-b'q)$ where $b' = 6$ to 9) was previously used by Brilon (1988) based on the earlier work by Jacobs (1979). The same empirical relationship has been used by Sullivan and Troutbeck (1993). Neither reference was sighted at the time of writing this paper.

The M3 model with estimates of φ obtained from Eqn (6) will be referred to as the **M3A** model.

The following linear model for the proportion of free vehicles was used by Tanner (1962 and 1967):

$$\varphi = 1 - \Delta q \qquad \text{for } q < 1/\Delta \qquad (7)$$

The M3 model with estimates of φ obtained from Eqn (7) will be referred to as the **M3T** model.

More general forms of the linear $\varphi - q$ model can be considered for calibrating real-life data. The AUSTROADS (1993) roundabout guide uses a linear model that has been generalised in SIDRA 4.07 (Akçelik 1991; Akçelik and Besley 1992) as $\varphi = a(1 - \Delta q)$ where a is a constant (see Eqn 9).

Discussion

Both the M3A and M3T models assume that the proportion of free vehicles decreases, i.e. the proportion of bunched vehicles increases, with increasing arrival flow rate. They predict zero bunching ($\varphi = 1.0$) at very low flows. While the M3T model assumes $\varphi = 0$ at $q = 1/\Delta$, the M3A model yields non-zero values of φ at high flows. This is discussed in the following section.

For the work reported in Akçelik (1994a), the M3A model was used with the parameter values of $\Delta = 2.0$ s and $b = 1.5$ for the single-lane case, $\Delta = 1.0$ s and $b = 1.0$ for the two-lane case, and $\Delta = 0.5$ s and $b = 1.0$ for cases with more than two lanes. The bunching factor of $b = 1.5$ for the single-lane case was derived as an approximation to the values predicted by the Tanner model (see Eqn 7).

The bunching factors for multi-lane cases were derived through comparison of the lane-by-lane treatment with the treatment of arrival flows in all lanes as a *single stream*.

CALIBRATION

Further research was carried out for better calibration of the M3A model using real-life and simulation data. Calibration of the M3A model involved determination of a minimum arrival headway (Δ) and a bunching factor (b) in Eqn (6) that give:

(a) a reasonable fit for the proportion of bunched vehicles (Eqn 6); and, more importantly,

(b) the best fit for the resulting headway distribution (Eqn 1).

The calibration process was carried out using real-life data for the single-lane case and simulated data for the multi-lane case. Note that the choice of Δ affects the proportion of unbunched vehicles (φ) used in the process of calibration.

Single-lane traffic surveys

The calibration of the model for single-lane traffic was based on field data collected using ARRB's VDAS vehicle detection system with treadle detectors (Leschinski and Roper 1993) at the following sites in Melbourne that represent a variety of traffic conditions. The VDAS system was used in the *individual vehicle* mode giving headway, speed and vehicle type data (a total of about 29,000 headways).

Site 1 (Ferntree Gully Road in Scoresby): Uninterrupted flow conditions at a mid-block location Eastbound between Jells Road and Nyadale Drive; two-lane traffic; headways in the shoulder-side lane recorded; distance to the nearest major upstream intersection (Jells Road; signalised) approximately 2.1 km; distance to the nearest major downstream intersection (Nyadale Drive; signalised) approximately 1.1 km; speed limit 80 km/h; 15-min flow rates in the range 456 to 1264 veh/h; heavy vehicle percentages in the range 5 to 20 per cent; total of 14,087 headways.

Site 2 (Fitzsimons Lane in Templestowe): Uninterrupted flow conditions at a mid-block location Southbound between Porter Street and Main Road; two-lane traffic; headways in the

shoulder-side lane recorded; distance to the nearest major upstream intersection (Main Road; roundabout) approximately 1 km; distance to the nearest major downstream intersection (Porter Street; roundabout) approximately 1.4 km; speed limit 80 km/h; 15-min flow rates in the range 8 to 1208 veh/h; heavy vehicle percentages in the range 8 to 13 per cent; total of 12,235 headways.

Site 3 (Clarinda Road in Clayton South): Uninterrupted flow conditions on the Southbound approach to the Clarinda Road–Bourke Road roundabout; single-lane traffic; distance to the give-way line approximately 180 m; distance to the nearest major upstream intersection (Bunney Road; roundabout) approximately 0.6 km; speed limit 60 km/h; 15-min flow rates in the range 352 to 584 veh/h; about 10 per cent heavy vehicles; total of 1052 headways.

Site 4 (Bourke Road in Clayton South): Uninterrupted flow conditions on the Eastbound approach to the Clarinda Road – Bourke Road roundabout; single-lane traffic; distance to the give-way line approximately 180 m; distance to the nearest major upstream intersection (Old Dandenong Road; sign-controlled) approximately 0.4 km; speed limit 60 km/h; 15-min flow rates in the range 348 to 556 veh/h; about 9 per cent heavy vehicles; total of 1016 headways.

Site 5 (Springvale Road in Glen Waverley): Headways of unqueued vehicles measured just past the stop line Southbound at the signalised intersection with Waverley Road; three lanes of through traffic; headways in the middle lane measured; distance to the nearest major upstream intersection (Kingsway; signalised) approximately 0.6 km; distance to the nearest major downstream intersection (Ferntree Gully Road; signalised) approximately 1.6 km; speed limit 70 km/h; 5-min flow rates in the range 816 to 1008 veh/h; about 8 per cent heavy vehicles; total of 274 headways.

Site 6 (Stephensons Road in Mount Waverley): Headways of unqueued vehicles measured just past the stop line Northbound at the signalised intersection with Waverley Road; two lanes of through traffic; headways in the centre lane measured; distance to the nearest major upstream intersection (Ferntree Gully Road; signalised) approximately 1.6 km; distance to the nearest major downstream intersection (Virginia Street; signalised) approximately 0.7 km; speed limit 60 km/h; 5-min flow rates in the range 732 to 888 veh/h; about 2 per cent heavy vehicles; total of 220 headways.

Note: Five-minute flow periods were used at Sites 5 and 6 due to the small amount of data available at these sites.

Multi-lane traffic simulations

Data for multi-lane cases were generated by simulating headways in individual lanes and measuring the headways of vehicles (cars only) across all lanes. In simulation, the headways in individual lanes were generated using the calibrated M3A model for single lanes — see Eqn (8a) below. Both equal and unequal lane utilisation cases were considered. The following conditions were simulated (total of about 367,500 headways were generated):

Two lanes:
Equal lane flows with total flow rates of 450, 900 and 1350 veh/h per lane; unequal flow cases with total flow rates of 900 veh/h (lane flow split: 33%, 67%), 1800 (lane flow split: 25%, 75%), and 2700 (lane flow split: 39%, 61%);

Three lanes:
Equal lane flows with total flow rates of 450, 900 and 1350 veh/h per lane; unequal flow cases with total flow rates of 1350 veh/h (lane flow split: 22%, 22%, 56%), 2700 (lane flow split: 17%, 33%, 50%), and 4050 (lane flow split: 18%, 41%, 41%);

Four lanes:
Equal lane flows with total flow rates of 450, 900 and 1350 veh/h per lane; unequal flow cases with total flow rates of 1800 veh/h (lane flow split: 17%, 17%, 33%, 33%), and 3600 (lane flow split: 13%, 25%, 25%, 37%), 5400 (lane flow split: 19%, 19%, 31%, 31%);

Calibration results

The following parameter values were found for the calibrated M3A model:

Single-lane case:

$$\Delta = 1.5 \text{ s and } b = 0.6 \qquad (8a)$$

Multi-lane cases:

$$\Delta = 0.5 \text{ s and } b = 0.5 \quad \text{for 2 lanes} \qquad (8b)$$
$$\Delta = 0.5 \text{ s and } b = 0.8 \quad \text{for > 2 lanes} \qquad (8c)$$

Headway distributions for equal and unequal lane utilisation in multi-lane cases did not indicate any significant differences. Therefore, the simple method of treating all lanes as a single traffic stream (using the total flow in equations) appears

to be a satisfactory method for the prediction of arrival headways for multi-lane cases.

The parameters for models M1, M2 and M3A given in Eqns (4), (5) and (8a) to (8c) are summarised in *Table I*. The proportions of free (unbunched) vehicles predicted using Eqns (6) to (8c) for single-lane and multi-lane flows are shown in *Figs 1a to 1c* together with the corresponding observed (field and simulation) data.

Discussion

It is seen that Eqns (8a) to (8c) indicate lower levels of bunching than those predicted by Tanner's linear model (Eqn 7), and those used previously (Akçelik 1994a). It is also seen that it would be possible to use more general linear models to represent real-life data more effectively than Tanner's linear model.

The calibrated model predicts rather high values of φ at high flows. The real-life data for the single-lane case did not include very high arrival flow rates since it was difficult to find sites that operate under such conditions (maximum average 15-min flow rate observed was 1264 veh/h). Since, by definition, all vehicles are expected to be bunched when the arrival flow rate equals $1/\Delta$, some improvement to the model for near-capacity conditions should be considered in future research. A two-regime model, a sudden change in the nature of the relationship near capacity (similar to speed-flow models) and even a change in the value of Δ as the flows approach uninterrupted flow capacity are the possibilities to explore for this purpose. In this context, it should be noted that $1/\Delta$ using the minimum headway value (Δ) determined for the uncongested flow conditions does not necessarily represent the uninterrupted flow capacity.

Flow conditions approaching uninterrupted flow capacities correspond to very poor levels of service for most urban traffic analyses, and may not be easy to find in practice due to other constraints on flows. However, research on this topic would be useful in understanding some fundamental properties of traffic flows.

An interesting observation from the field surveys was that the arrival headway distributions at Sites 1 and 2 (mid-block free flow sites) and at Site 5 (the stop line of a busy signalised intersection) were virtually identical (with similar speed limits and same average arrival flow rate).

Roundabout circulating streams

In SIDRA 4.07 and earlier versions, as in the new AUSTROADS (1993) roundabout guide, all lanes of the circulating flow are treated as a single stream, and the following linear model is used for estimating the proportion of free vehicles in the circulating stream:

$$\varphi_c = 0.75\,(1 - \Delta\, q_c) \quad \text{for } q_c < 1/\Delta \qquad (9)$$

where q_c is the circulating flow rate (veh/s). Intra-bunch headways of $\Delta = 2$ s for single-lane circulating roads and $\Delta = 1$ s for multi-lane circulating roads are used.

TABLE I

Summary of parameter values for various headway distribution models

Number of lanes in traffic stream	Negative exponential model (M1)		Shifted negative exponential model (M2)		Bunched exponential model (M3A)		
	Δ	φ	Δ	φ	Δ	b	φ
1	0	1.0	1.5	1.0	1.5	0.6	$e^{-0.9\,q}$
2	0	1.0	0.5	1.0	0.5	0.5	$e^{-0.25\,q}$
>2	0	1.0	0.5	1.0	0.5	0.8	$e^{-0.4\,q}$

- For the M3T model, use $\varphi = 1 - \Delta q$ with same Δ values as model M3A.
- Normally, the shifted negative exponential model (M2) is used for single-lane traffic only.

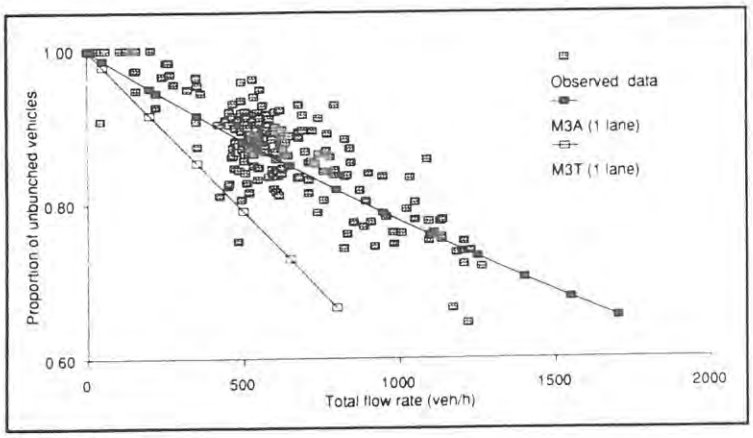


Figure 1a

Proportion of unbunched (free) vehicles (φ) for single-lane uninterrupted traffic streams as a function of the total flow (q_f): field data and predictions for M3A and M3T models

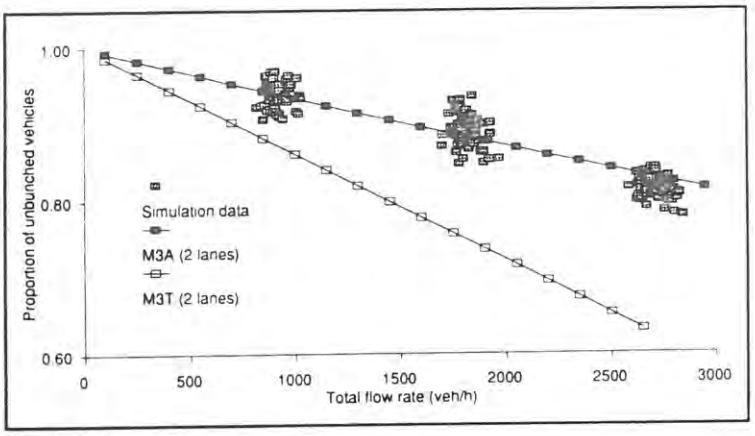


Figure 1b

Proportion of unbunched (free) vehicles (φ) for two-lane uninterrupted traffic streams as a function of the total flow in all lanes (q_f): simulation data and predictions for M3A and M3T models

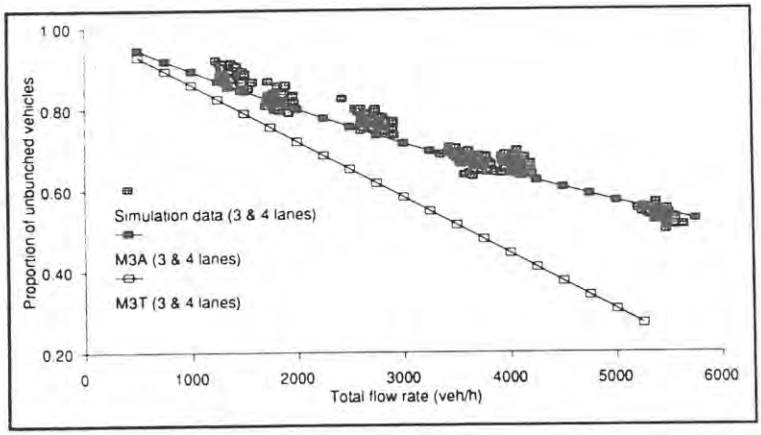


Figure 1c

Proportion of unbunched (free) vehicles (φ) for three-lane and four-lane uninterrupted traffic streams as a function of the total flow in all lanes (q_f): simulation data and predictions for M3A and M3T models

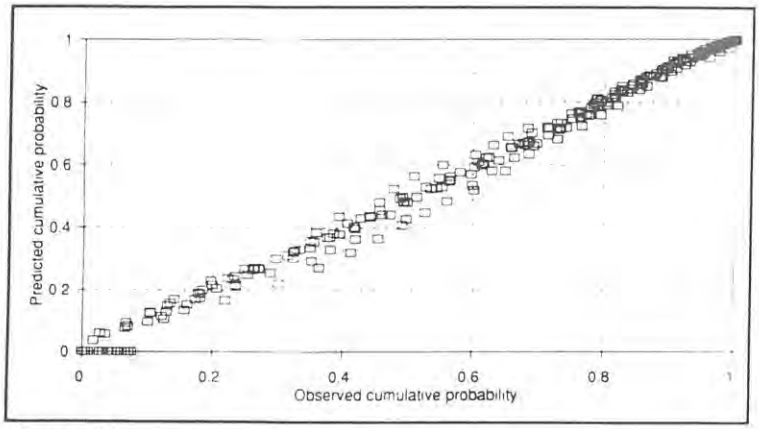


Figure 1d

Predicted vs measured cumulative arrival headway probabilities, F(t), for single-lane traffic streams

The data used for the derivation of Eqn (9) is listed in ARRB Special Report SR 45 (Troutbeck 1989). Using this data, b = 2.5 was chosen for both both single-lane and multi-lane circulating flows since the differences between b values for single-lane and multi-lane cases were found to be small — as in Eqn (9). Note that this calibration was based on the use of the same intra-bunch headways as for Eqn (9) since the headway data were not available for full calibration of the model. Thus, for roundabout circulating streams:

$$\varphi_c = e^{-2.5 \Delta q_c} \qquad (10)$$

where Δ = 2 s for single-lane and Δ = 1 s for multi-lane cases.

Figs 2a and 2b show the proportions of free (unbunched) vehicles predicted using Eqn (10) for single-lane and multi-lane circulating flows at roundabouts together with the corresponding observed (SR 45) data and the estimates from the linear model (Eqn 9).

Eqns (9) and (10) predict higher levels of bunching compared with those predicted by Eqns (8a) to (8c). The difference can be explained by vehicles entering the circulating stream from queues at roundabout approaches.

Note: For predicting delays, queue length and stop rates for roundabout entry streams, arrival headway distributions based on the parameter values given in Eqn (8a) or *Table I* (single-lane cases) are used.

Examples

Figure 3 shows the *observed* cumulative headway probabilities for a single-lane case with an average arrival flow rate of 1150 veh/h (field data from Sites 1 and 2), as well as the corresponding *predictions* by the arrival headway models M1 (Δ = 0), M2 (Δ = 1.5 s), M3A (Δ = 1.5 s, b = 0.6) and M3T (Δ = 1.5 s) calculated from Eqns (1) to (8).

Similarly, *Fig. 4* shows observed cumulative headway probabilities (data from simulation) for a four-lane case with a total arrival flow rate of 3600 veh/h, as well as the corresponding predictions by the arrival headway models M1 (Δ = 0), M2 (Δ = 0.5 s), M3A (Δ = 0.5 s, b = 0.8) and M3T (Δ = 0.5 s).

Figures 5a and *5b* show examples of cumulative distribution and probability density functions for a single-lane case (arrival flow rate = 1200 veh/h) for the arrival headway models M1 (Δ = 0), M2 (Δ = 1.5 s), and M3A (Δ = 1.5 s, b = 0.6). For the same single-lane case, *Figs 6a* and *6b* show examples of cumulative distribution and probability density functions for model M3 with specified φ values of 0.4 for high bunching and 0.9 for low bunching and for model M3A (Δ = 1.5 s, b = 0.6). (Note: in the figures, read *phi* as φ.)

Discussion

It is seen that there are significant differences in the predictions of arrival headways by different models, especially for small headways. Model M3A is seen to provide good estimates of arrival

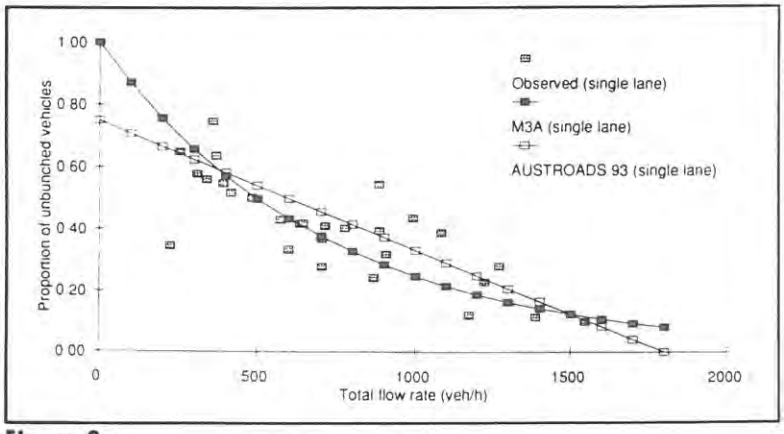


Figure 2a

Proportion of unbunched (free) vehicles (φ_c) for single-lane circulating streams at roundabouts as a function of the total circulating flow (q_c): observed (SR 45) data and predictions for M3A and AUSTROADS (1993) models

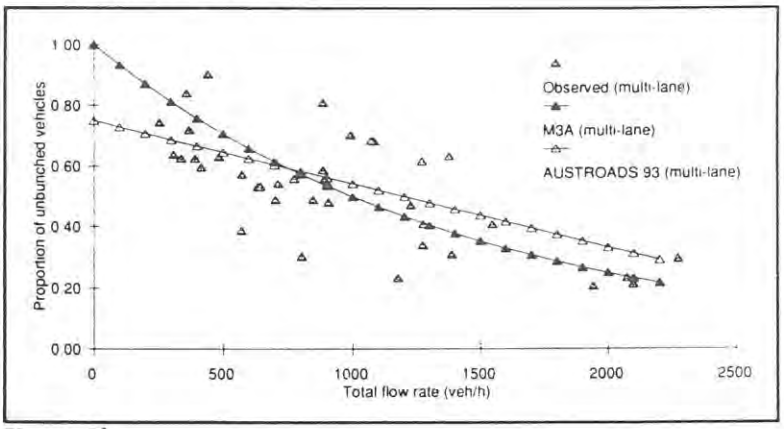


Figure 2b

Proportion of unbunched (free) vehicles (φ_c) for multi-lane circulating streams at roundabouts as a function of the total circulating flow (q_c): observed (SR 45) data and predictions for M3A and AUSTROADS (1993) models

headways. Generally, the shifted negative exponential model (M2) does not appear to be satisfactory. It is also seen that the amount of bunching as represented by parameter φ in model M3, has a significant effect on the prediction of arrival headways.

GAP-ACCEPTANCE CAPACITIES AND DELAYS USING DIFFERENT EXPONENTIAL HEADWAY DISTRIBUTIONS

Formulae for estimating the average number of arrivals and average headway before a gap change, and the corresponding average green extension time at actuated signals using the bunched exponential headway distribution are given in Akçelik (1994a,b). Significant differences were found in average green times and cycle times predicted using different arrival headway distributions (models M1, M2, and M3A) and different bunching levels in model M3. In this paper, another area of application, namely the prediction of gap-acceptance capacities and delays will be discussed as an example to indicate the impact of different arrival headway distributions. In this example, capacities and delays for a left-turning vehicle stream (*minor* or *opposed* movement) which gives way to through traffic (*major* or *opposing* movement) at an intersection

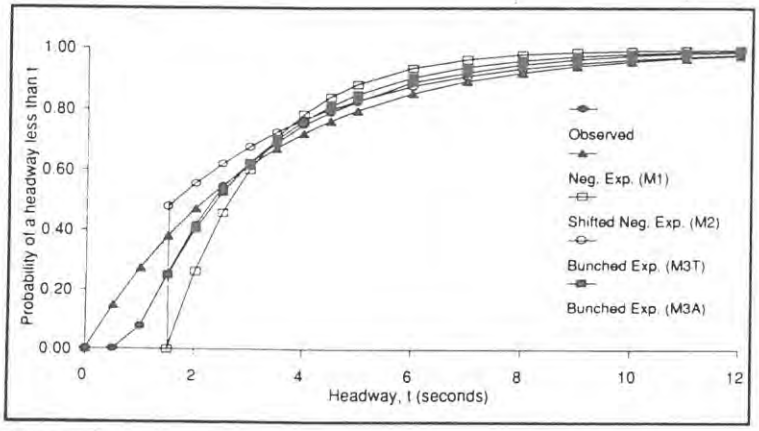


Figure 3

Cumulative arrival headway distribution: observed (field) data and predictions by models M1 (Δ = 0), M2 (Δ = 1.5), M3A (Δ = 1.5, b = 0.6) and M3T (Δ = 1.5): single-lane case, average arrival flow rate = 1150 veh / h

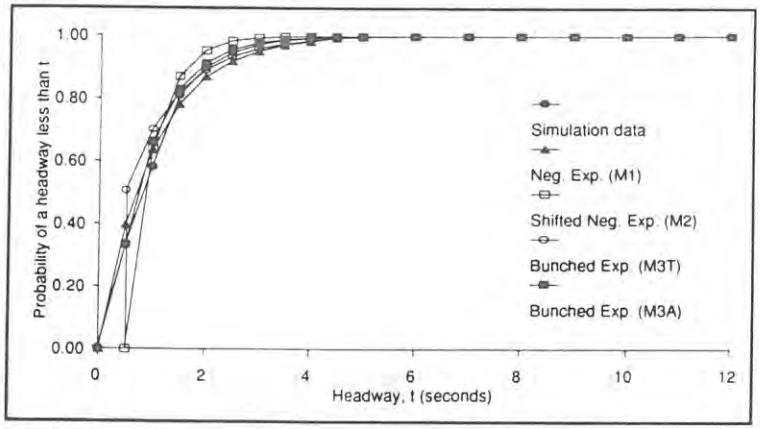


Figure 4

Cumulative arrival headway distribution: observed (simulation) data and predictions by models M1 (Δ = 0), M2 (Δ = 0.5), M3A (Δ = 0.5, b = 0.8) and M3T (Δ = 0.5): four-lane case, average arrival flow rate = 3600 veh / h

controlled by a give-way sign will be considered. All capacity and delay calculations are carried out for individual lanes of minor movements.

Gap-acceptance capacity

The general capacity model for gap-acceptance processes when each lane of the major (opposing) movement is treated separately (Tanner 1967; Troutbeck 1986, 1989, 1991) was adopted in SIDRA 4.07 in the form of the following formula with the introduction of a minimum capacity concept (Akçelik 1991):

$$Q = max\,(Q_g, Q_{min}) \qquad (11)$$

$$Q_g = \frac{3600\,\lambda\theta\,e^{-\lambda(\alpha-\Delta)}}{1-e^{-\lambda\beta}} \quad for\ q_t > 0 \qquad (11a)$$

$$= 3600/\beta_o \quad for\ q_t = 0$$

$$Q_{min} = min\,(q_e, 60\,n_m) \qquad (11b)$$

where:

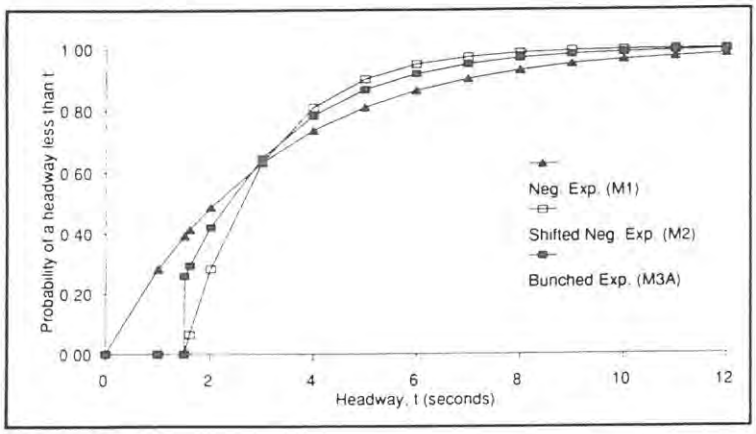


Figure 5a

Cumulative headway probabilities predicted by models M1 (Δ = 0), M2 (Δ = 1.5), M3A (Δ = 1.5, b = 0.6): single-lane case, arrival flow rate = 1200 veh / h

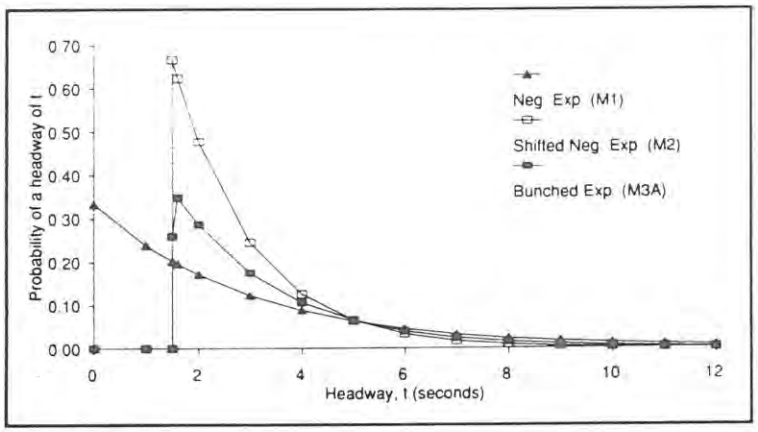


Figure 5b

Arrival headway probabilities predicted by models M1 (Δ = 0), M2 (Δ = 1.5), M3A (Δ = 1.5, b = 0.6): single-lane case, arrival flow rate = 1200 veh / h (corresponding to cumulative probabilities in Figure 5a)

Q = entry capacity of the minor movement (in veh/h);

Q_g = capacity estimate using the gap acceptance technique (in veh/h);

Q_{min} = minimum capacity (in veh/h);

q_e = minor movement arrival flow rate (in veh/h);

n_m = minimum number of vehicles per minute that can enter the major traffic stream under heavy opposing flow conditions;

q_t = total arrival flow in all lanes of the major movement (in veh/s);

α, β = mean critical gap and follow-up headway for the minor movement (entry stream) (in secs);

β_0 = follow-up headway when the major movement flow rate is zero ($\beta_0 = \beta$ for constant follow-up headways);

Δ = intra-bunch (minimum) headway in a major movement *lane* (in sec); and

λ, θ = parameters for the major movement calculated from:

$$\lambda = \Sigma \lambda_i \qquad (11c)$$

$$\lambda_i = \varphi_i \, q_i / (1 - \Delta q_i) \qquad (11d)$$

$$\theta = \prod (1 - \Delta q_i) \qquad (11e)$$

subject to $q_i \leq 0.98/\Delta$

The summation and multiplication are for the major movement lanes i = 1 to n (n = number of major movement lanes), φ_i = proportion of free (unbunched) vehicles in *ith* major movement lane, and q_i = flow rate in *ith* major movement lane (*veh/s*); the total opposing flow, $q_t = \Sigma q_i$. The upper limit of $q_i = 0.98/\Delta$ was introduced in SIDRA to enable the calculation of non-zero values of λ and θ when the major movement flow rate approaches $1/\Delta$.

The general capacity model expressed by Eqn (11) is also applicable to the case of *several opposing movements*. In this case, Eqns (11c) to (11e) are applied for each lane of all opposing movements consecutively.

The gap-acceptance capacity can be expressed as follows when traffic in all lanes of the major movement is treated together as one stream, i.e. not lane by lane, as in the case of current AUSTROADS and SIDRA 4.07 models for roundabouts:

$$Q_g = \frac{3600\ \varphi\ q_t\ e^{-\lambda(\alpha-\Delta)}}{1-e^{-\lambda\beta}} \qquad \text{for } q_t > 0 \quad (12)$$

$$= 3600/\beta_0 \qquad \text{for } q_t = 0$$

where:

$$\lambda = \varphi\ q_t / (1 - \Delta q_t) \quad (12a)$$

subject to $q_t \le 0.98/\Delta$

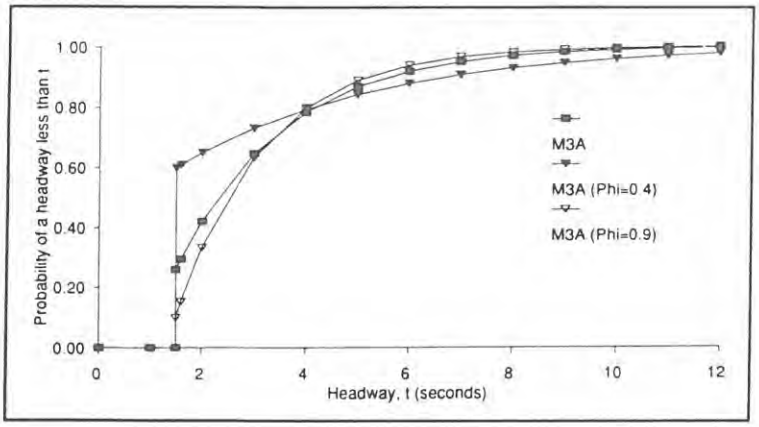


Figure 6a

Cumulative headway probabilities predicted by model M3 with specified proportions of free vehicles (φ = 0.4 for high bunching, and φ = 0.9 for low bunching) and by model M3A (Δ = 1.5, b = 0.6): single-lane case with arrival flow rate = 1200 veh / h

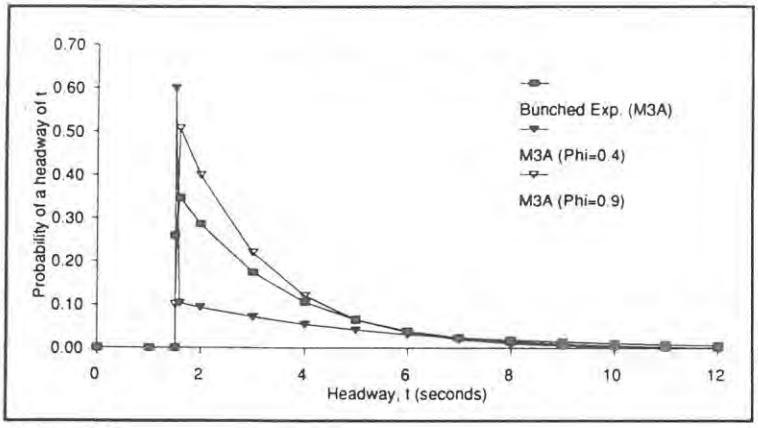


Figure 6b

Arrival headway probabilities predicted by model M3 with specified proportions of free vehicles (φ = 0.4 for high bunching, and φ = 0.9 for low bunching) and by model M3A (Δ = 1.5, b = 0.6): single-lane case with arrival flow rate = 1200 veh / h (corresponding to cumulative probabilities in Figure 6a)

For a single-lane major movement, Eqns (11) and (12) give the same result. For multi-lane cases, the intra-bunch headway (Δ) and proportion of free vehicles (φ) for use in Eqn (11) are for the single lane case — see Eqn (8a). However, Δ and φ for use in Eqn (12) are for the multi-lane case — see Eqns (8b) and (8c). For model M1, there is no difference between Eqns (11) and (12) since $\Delta = 0$, $\varphi = 1$ and $\lambda = q_t$.

The following should be noted about the formulae used in Chapter 8 of the current AUSTROADS (1988) *capacity guide* :

(a) The method used for calculating unsignalised (sign-controlled) intersection capacities is equivalent to the M1 model ($\Delta = 0$, $\varphi = 1$ and $\lambda = q_t$).

(b) For roundabouts, the same method as in the NAASRA (1986) *roundabout guide* is described. This method uses the M1 model ($\Delta = 0, \varphi = 1$ and $\lambda = q_t$) for multi-lane circulating streams and the M3T model (Eqn 7) for φ with Δ = 2 s) for single-lane circulating streams.

The new roundabout guide (AUSTROADS 1993) uses a capacity formula equivalent to Eqn (12), using the M3 model with φ predicted using Eqn (9).

Examples

Figure 7a shows entry capacities as a function of the arrival flow rate in the case of a single-lane major movement predicted from Eqn (12) using α = 4 s and β = 2 s for arrival headway models M1 ($\Delta = 0$), M2 (Δ = 1.5 s), M3A (Δ = 1.5 s, b = 0.6) and M3T (Δ = 1.5 s).

Figure 7b shows entry capacities as a function of the arrival flow rate in the case of a two-lane major movement (equal lane flows) predicted from Eqn (12) using α = 4 s and β = 2 s for arrival headway models M1 ($\Delta = 0$), M2 (Δ = 0.5 s), M3A (Δ = 0.5 s, b = 0.5) and M3T (Δ = 0.5 s).

Figure 8 presents a comparison of the minor movement capacities for the case of a two-lane major movement (equal lane flows) predicted using the M3A model with α = 4 s and β = 2 s and treating the major movement (a) lane-by-lane: Δ = 1.5 s, b = 0.6 for each lane in Eqn (11), and (b) as a single stream: Δ = 0.5 s, b = 0.5 for both lanes together in (Eqn 12).

Discussion

It is seen that there are significant differences in capacities predicted using different exponential distributions of arrival headways, especially in the case of single-lane major movements and when the major flow arrival rates are high. Differences between capacities predicted by treating the major movement lane-by-lane and as a single stream are also significant for high major movement flow rates, but not very large.

Gap-acceptance delay

The following time-dependent delay formula is used in SIDRA 4.07 for estimating delays to entry streams at roundabouts and minor movements at unsignalised intersections (Akçelik 1991; Akçelik and Troutbeck 1991). The formula is based on the steady-state model given by Troutbeck (1986, 1989, 1991):

$$d = d_m + 900\,T\left[(x-1) + \sqrt{(x-1)^2 + \frac{8k\,x}{Q\,T}}\right] \quad (13)$$

$$d_m = \frac{e^{\lambda(\alpha-\Delta)}}{\lambda\,\theta} - \alpha - \frac{1}{\lambda} + \frac{\lambda\Delta^2 - 2\Delta + 2\Delta\varphi}{2\,(\lambda\Delta + \varphi)} \quad (13a)$$

$$k = d_m\,Q / 3600 \quad (13b)$$

where:

d = average delay per vehicle (in s);

d_m = minimum delay experienced by the minor movement (in s);

T = duration of the flow (design or analysis) period (in hours);

x = degree of saturation (flow/capacity ratio) in the specified flow period;

k = delay parameter;

Q = minor movement capacity (in veh/h) calculated from Eqn (11) or (12); and

$\alpha, \Delta, \lambda, \theta$ are the gap acceptance parameters defined in Eqn (11) for the lane-by-lane treatment (with $\varphi = \lambda\theta/q_t$) and Eqn (12) for the single-stream treatment.

The AUSTROADS (1993) roundabout guide uses Eqn (13) for the case when the circulating stream

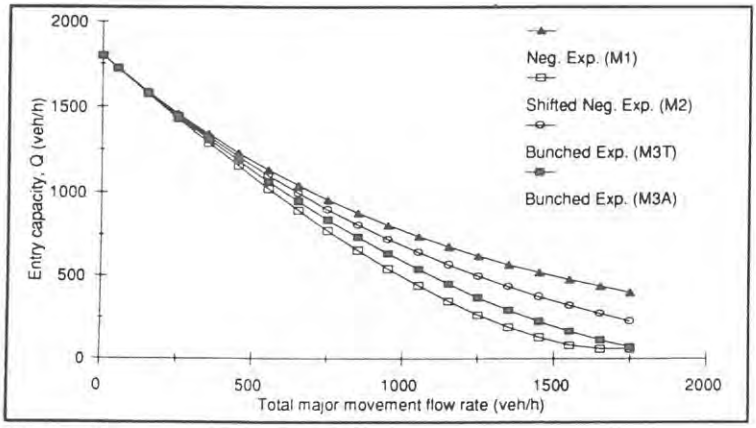


Figure 7a

Minor movement capacities (Q) as a function of the arrival flow of a single-lane major movement predicted using arrival headway models M1 ($\Delta = 0$), M2 ($\Delta = 1.5$ s), M3A ($\Delta = 1.5$ s, $b = 0.6$) and M3T ($\Delta = 1.5$ s): $\alpha = 4$ s and $\beta = 2$ s

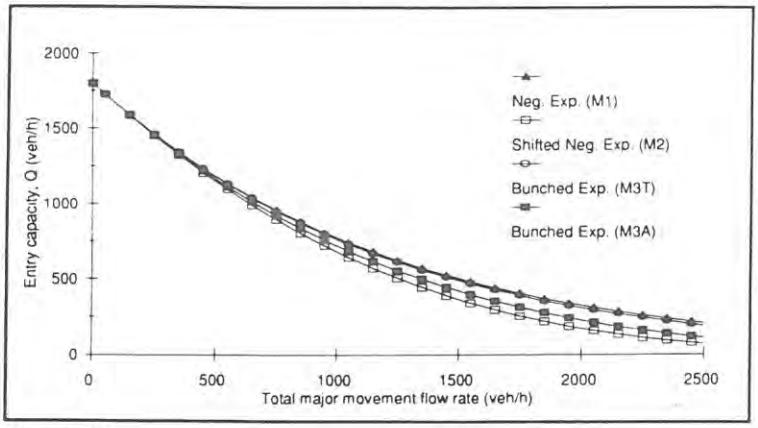


Figure 7b

Minor movement capacities (Q) as a function of the arrival flow of a two-lane major movement (equal lane flows) predicted using arrival headway models M1 ($\Delta = 0$), M2 ($\Delta = 0.5$ s), M3A ($\Delta = 0.5$ s, $b = 0.5$) and M3T ($\Delta = 0.5$ s): $\alpha = 4$ s and $\beta = 2$ s

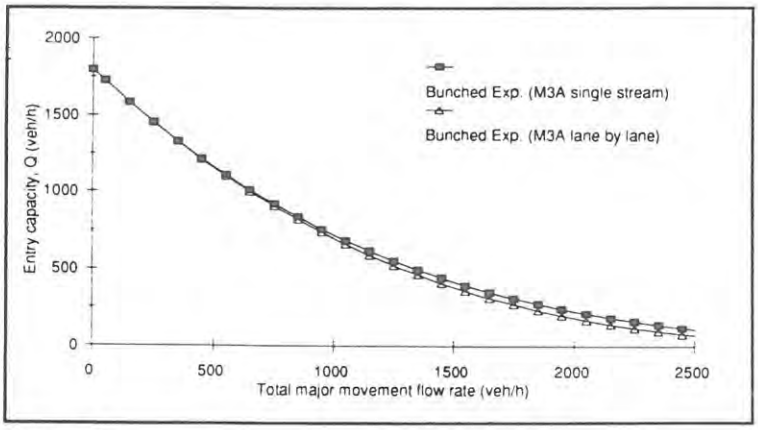


Figure 8

Comparison of minor movement capacities (Q) for a case of two-lane major movement (equal lane flows) predicted using model M3A by treating the major movement lane by lane ($\Delta = 1.5$ s, $b = 0.6$ for each lane) and as a single stream ($\Delta = 0.5$ s, $b = 0.5$): $\alpha = 4$ s and $\beta = 2$ s

free vehicles calculated from Eqn (9) (see also Akçelik and Troutbeck 1991).

Examples

Figure 9a shows delays as a function of the minor movement degree of saturation in the case of a single-lane major movement with an arrival flow rate of 900 veh/h, predicted using $\alpha = 4$ s and $\beta = 2$ s for the arrival headway models M1 ($\Delta = 0$), M2 ($\Delta = 1.5$ s), M3A ($\Delta = 1.5$ s, b = 0.6) and M3T ($\Delta = 1.5$ s) for the major movement.

Figure 9b shows delays as a function of the minor movement degree of saturation in the case of a two-lane major movement with an arrival flow rate of 900 veh/h per lane, predicted using $\alpha = 4$ s and $\beta = 2$ s for the arrival headway models M1 ($\Delta = 0$), M2 ($\Delta = 0.5$ s), M3A ($\Delta = 0.5$ s, b = 0.5) and M3T ($\Delta = 0.5$ s) for the major movement.

Figure 10 presents a comparison of the minor movement delays for the case of a two-lane major movement predicted using the M3A model (same conditions as in *Fig. 9b*) and treating the major movement (a) lane by lane: $\Delta = 1.5$ s, b = 0.6 for each lane; and (b) as a single stream: $\Delta = 0.5$ s, b = 0.5 for both lanes together.

A flow period of T = 0.5 h was used in delay calculations for *Figs 9a, 9b* and *10*.

Discussion

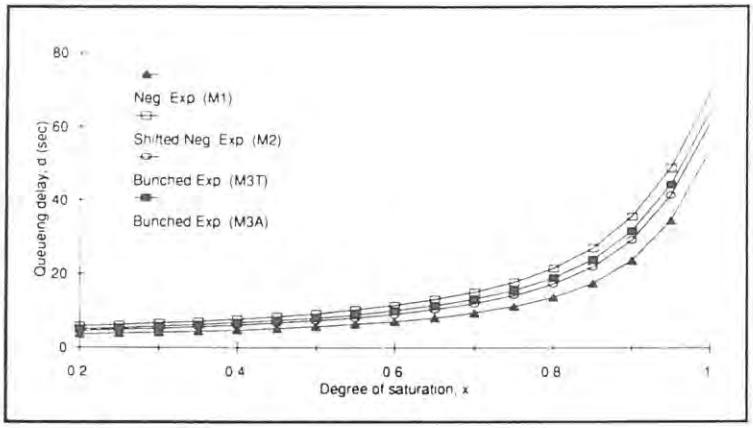


Figure 9a

Minor movement delays (d) as a function of the minor movement degree of saturation (x) in the case of a single-lane major movement (arrival flow rate = 900 veh/h) predicted using the arrival headway models M1 ($\Delta = 0$), M2 ($\Delta = 1.5$ s), M3A ($\Delta = 1.5$ s, b = 0.6) and M3T ($\Delta = 1.5$ s): $\alpha = 4$ s and $\beta = 2$ s, T = 0.5 h

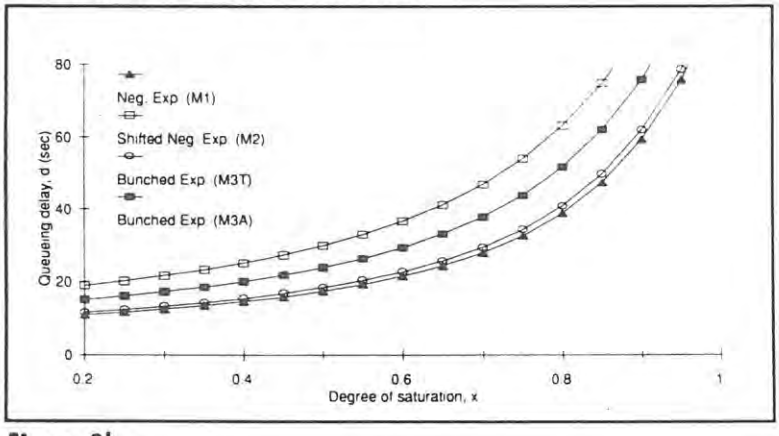


Figure 9b

Minor movement delays (d) as a function of the minor movement degree of saturation (x) in the case of a two-lane major movement (arrival flow rate = 900 veh/h per lane) predicted using the arrival headway models M1 ($\Delta = 0$), M2 ($\Delta = 0.5$ s), M3A ($\Delta = 0.5$ s, b = 0.5) and M3T ($\Delta = 0.5$ s): $\alpha = 4$ s and $\beta = 2$ s, T = 0.5 h

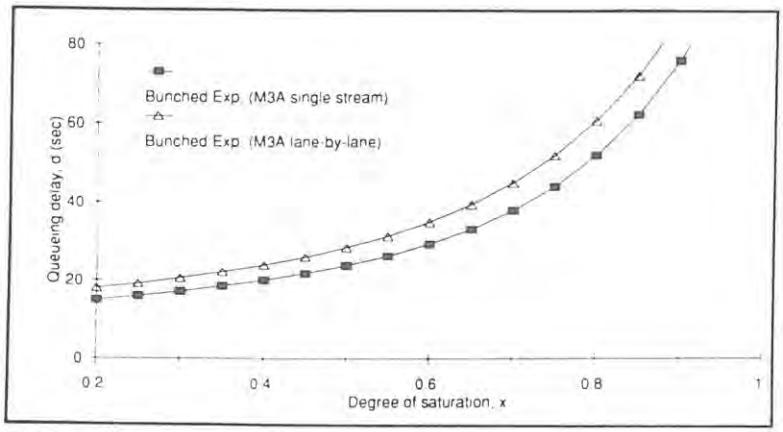


Figure 10

Comparison of minor movement delays (d) for a case of two-lane major movement (arrival flow rate = 900 veh / h per lane) predicted using model M3A by treating the major movement lane-by-lane (Δ = 1.5 s, b = 0.6 for each lane) and as a single stream (Δ = 0.5 s, b = 0.5); α = 4 s and β = 2 s, T = 0.5 h

As in the case of capacity prediction, it is seen that there are significant differences in delays predicted using different exponential distributions of arrival headways. Differences between delays predicted by treating the major movement lane by lane and as a single-stream are small at low flows (as discussed by Troutbeck 1991), but increase with increased flows.

CONCLUSIONS

The calibration of the model for predicting the proportion of free (unbunched) vehicles indicates lower levels of bunching than those predicted by Tanner's linear model. The new model also predicts rather low values of bunching at high flows compared to Tanner's model. The real-life data did not include very high arrival flow rates as it is difficult to find sites that operate under such conditions. Some improvement to the model for high-flow conditions should be considered in future research. A two-regime model, a sudden change in the nature of the relationship near capacity (similar to speed-flow models) and even a change in the value of Δ (for better fit) as the flows approach capacity are possibilities to explore for this purpose.

Significant differences were found in capacities and delays predicted using different exponential distributions of arrival headways. Differences between capacities and delays predicted by treating the major movement lane by lane and as a single stream are also significant, but not large.

These findings are in line with work on actuated signals which indicated that there are significant differences in average actuated signal green times and cycle time predicted using different exponential distributions of arrival headways.

While the bunched exponential distribution of arrival headways is relatively new and its use less common, it has been found to be more realistic. Its use instead of the other two exponential models is strongly recommended. In particular, the more commonly used shifted negative exponential (M2) model is found to give poor predictions for the range of small headways which is of particular interest in the modelling of intersection operations. ▮

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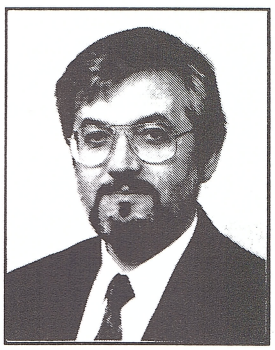
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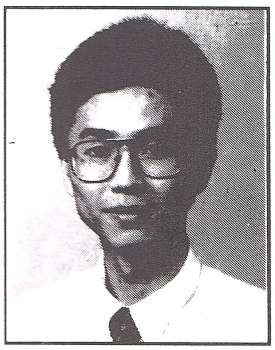
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Rahmi Akçelik

Rahmi Akçelik is a Chief Research Scientist at the Australian Road Research Board Ltd. His current work is on Capacity and Congestion Management under the Transport Efficiency Program of ARRB. Previously, he worked as a traffic engineer-planner with the National Capital Development Commission in Canberra, and as a lecturer in Road and Traffic Engineering at the Black Sea Technical University, Turkey. Rahmi graduated from Istanbul Technical University, Turkey, as a Civil Engineer in 1968, and received a Ph.D. in Transport Engineering from the University of Leeds, England, in 1974. He is a Fellow of the Institution of Engineers, member of the US Transportation Research Board Committees – Highway Capacity and Quality of Service, and Traffic Signal Systems.



Edward Chung

Edward Chung is a research assistant at the Australian Road Research Board Ltd. He graduated from Monash University in 1986 with a Bachelor degree in Civil Engineering. He studied at the University of California, Berkeley in 1989. Edward submitted his Ph.D. thesis "Modelling Single-lane Roundabout Performance" at Monash University in 1993. His current work is on modelling of signals, roundabouts and sign-controlled intersections.

Contact *Dr Rahmi Akçelik*
Australian Road Research Board
PO Box 156
Nunawading Vic 3131