Sapienza Università di Roma

Signal Setting Design and Synchronization in Via Prenestina

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1. Introduction

The report is related to Signal Setting Design and Synchronization in Via Prenestina given in the guide of the project. The intersections are examined under the Methodology based on the High-Capacity Manuel (HCM). The project includes several parts which is the introduction, methodology how to determine the level of services in the current situation, the solutions to reduce the delays that users experience during travels in the intersections. The object of the project is to solve and minimize the density problem in the intersections given, analyzing to optimize cycles, managing green times and offset with available traffic signal control.

First of all, the capacity and level of service of signalized intersections are defined. Level of service directly related to the control delay value. The project was analyzed by taking into account the input parameters required to conduct an operational analysis for signalized intersections which are divided into three main categories namely geometric, traffic, and signalization.

The determination of flow rate is obtained by considering the lane grouping, adjustment factors and basic equations. Capacity is evaluated in terms of the ratio of demand flow rate to capacity (v/c ratio), while the level of service is evaluated based on control delay per vehicle (in seconds per vehicle). After definition of the capacity and level of service, performance measures which is the delay are evaluated for each lane group by analyzing separately. Control delay includes initial deceleration delay, queue move-up time, stopped delay, and final acceleration delay. This report gives information about optimization in the intersection that minimize the delays.

As a result, the report gives opportunity the cooperation between the results of the current situation of the intersections and better alternatives that are able to implemented in the intersections to obtain better performance.

2. Study Area

The road artery chosen for this project is Via Prenestina. The study area is composed by 4 intersections from Via Dignano d'Istria, with a total length of 812 m.

Table 1 - The List of Study Area

Intersection Number	**Approaches**				**Number of Phases**
	East Bound	**West Bound**	**North Bound**	**South Bound**	
1	**Via Prenestina**	**Via Prenestina**	Largo Irpinia	**Via Dignano d'Istria**	2
2	**Via Prenestina**	**Via Prenestina**	**Via Olevano Romano**	-	2
3	**Via Prenestina**	**Via Prenestina**	**Via Tor de'Schiavi**	Viale della Serenissima	3
4	**Via Prenestina**	**Via prenestina**	**Via Giacomo Bresadola**		2

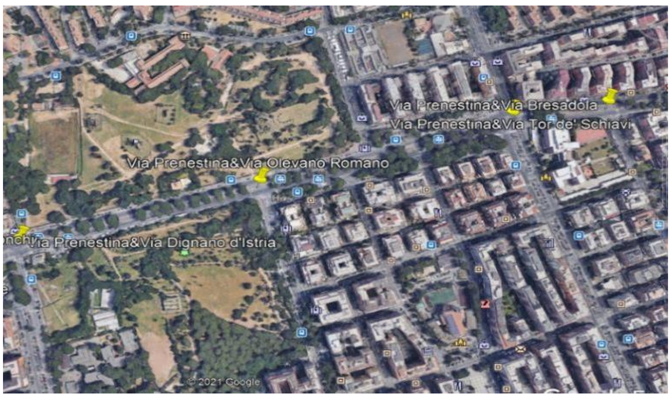


Figure 1-Study Area

3. Highway Capacity Manual

The High way Capacity Manual (HCM) is a publication of the Transportation Research Board of the National Academies of Science in the United States. It contains concepts, guidelines, and computational procedures for computing the capacity and quality of service of various highway facilities, including freeways, highways, arterial roads, roundabouts, signalized and unsignalized intersections, rural highways, and the effects of mass transit, pedestrians, and bicycles on the performance of these systems. The inputs of the procedure and which are the out puts are shown below;

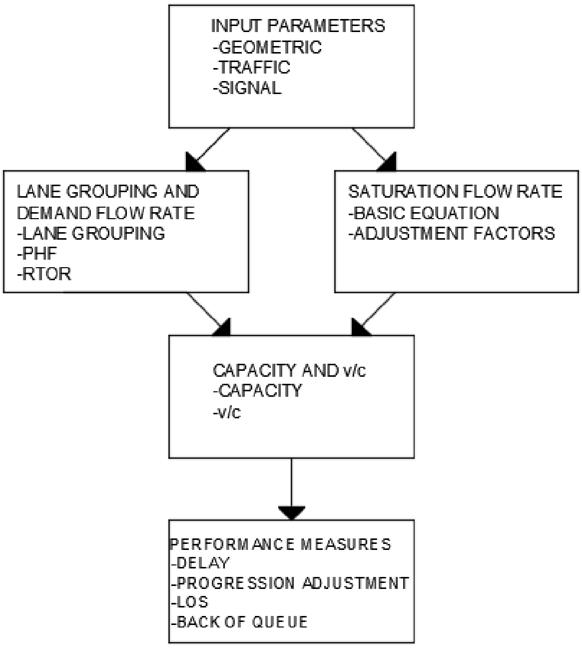


Figure 2 - Signalized Methodology

3.1. Input Parameters

The outputs are found with the help of the inputs as geometric, traffic and signalization by following process. There are the details on the below.

Type of Condition	Parameter
Geometric conditions	Area type Number of lanes, N Average lane width, W (m) Grade, G (%) Existence of exclusive LT or RT lanes Length of storage bay, LT or RT lane, L_s (m) Parking
Traffic conditions	Demand volume by movement, V (veh/h) Base saturation flow rate, s_0 (pc/h/ln) Peak-hour factor, PHF Percent heavy vehicles, HV (%) Approach pedestrian flow rate, v_{ped} (p/h) Local buses stopping at intersection, N_B (buses/h) Parking activity, N_m (maneuvers/h) Arrival type, AT Proportion of vehicles arriving on green, P Approach speed, S_A (km/h)
Signalization conditions	Cycle length, C (s) Green time, G (s) Yellow-plus-all-red change-and-clearance interval (intergreen), Y (s) Actuated or pretimed operation Pedestrian push-button Minimum pedestrian green, G_p (s) Phase plan Analysis period, T (h)

Figure 3 - Input Data's

3.2. Lane Grouping

One of the main subjects is lane grouping. The type of lane should be determined based on the movements (Right and Left Turn) and approaches (South Bound/North Bound/West Bound/East Bound) and typology.

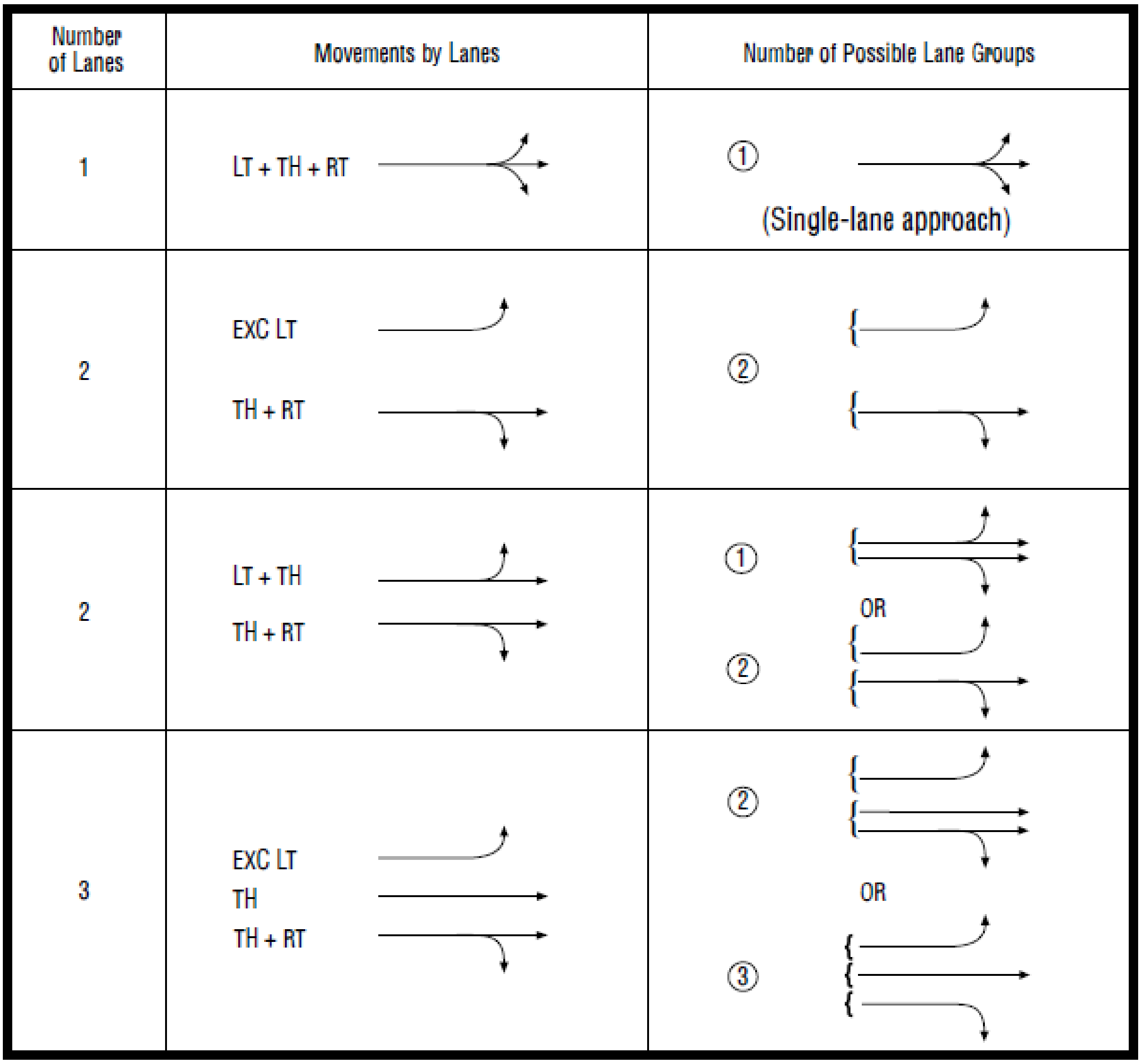


Figure 4 - Analysis of lane groups

3.3. Demand Flow Rate

There are three approaches to determine the flow rate. If necessary, volume of movements is adjusted according to flow rates for each desired period of analysis. The Approach A is used and calculated during 15 min period of peak hours in this project. Time and date information was givenon below.

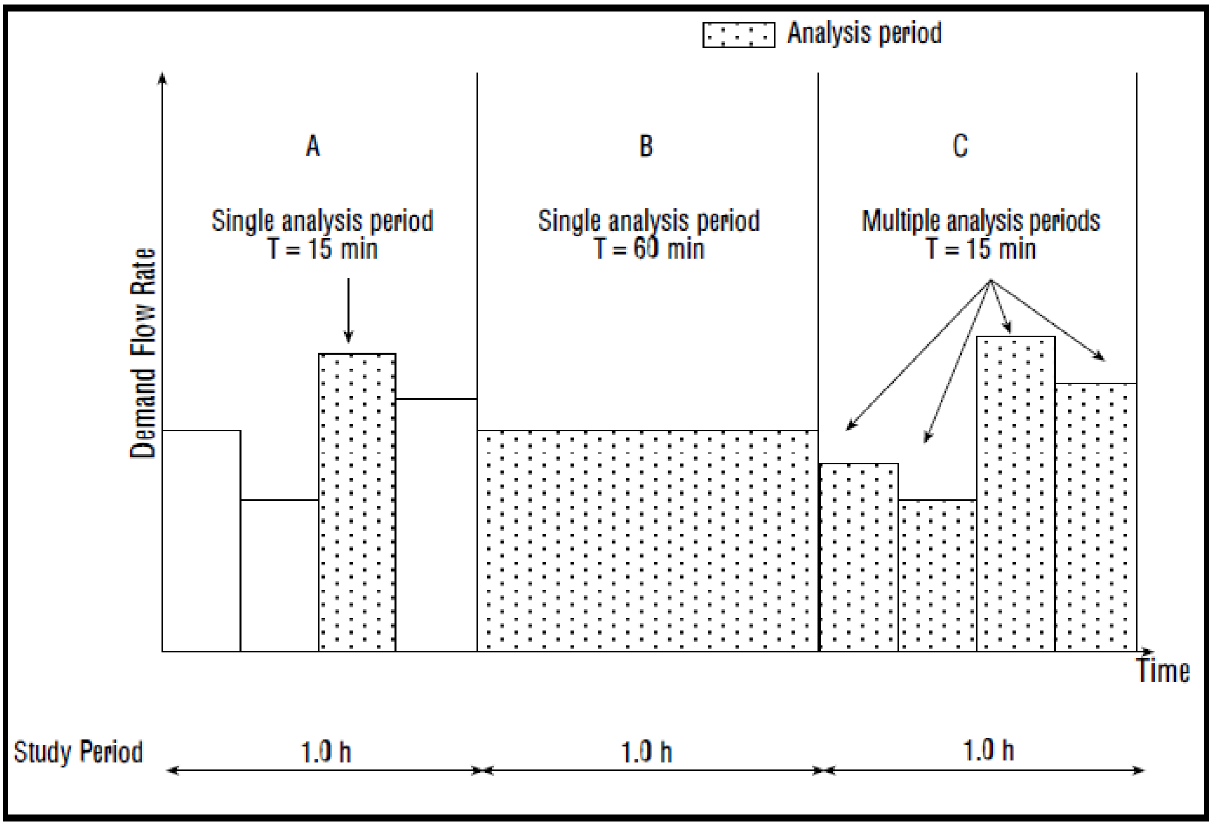


Figure 5 - Three Alternative Flow Rate Approaches

The approach A is the one adopted in this paper, and it's also the one that has traditionally been used in the HCM. The length of the period being analyzed is only 15 min, and the analysis period (T), therefore, is 15 min or 0.25 h. In this case, either a peak 15-min volume is available or, because the 15 min flow is not known, flow can be estimated using hourly volume and the Peak Hour Factor (PHF). A peak 15-min flow rate is derived from an hourly volume by dividing the movement volumes by an appropriate PHF, which may be defined for the intersection as a whole, for each approach, or for each movement.

$$v_p = \frac{V}{PHF}$$

-V_p = flow rate during peak 15 mn period (veh/h)
-V = Hourly volume (veh/h)
-PHF = peak-hour factor

PHF was set at 0.9, assuming that the volume of traffic doesn't vary substantially during the peak hours. If v/c exceeds 1.0 during the analysis period, the length of the analysis period should be extended to cover the period of oversaturation in the same trend, as long as the average flow during that period is relatively constant.

3.4. Saturation Flow Rate

Normally the saturation flow rate is equal to 1 for theoretical approach but, it never become 1. Because some factors are affecting the value of it like number of parking cars, range of heavy vehicles etc. The saturation flow rate is the flow in vehicles per hour that can be accommodated by the lane group assuming that green phase was displayed 100 percent of the time (i.e. g/C=1.0). The saturation flow rate for each lane group is computed according to this equation:

$$s = s_o\, N\, f_w\, f_{HV}\, f_g\, f_p\, f_{bb}\, f_a\, f_{LU}\, f_{LT}\, f_{RT}\, f_{Lpb}\, f_{Rpb}$$

Where;

s = saturation flow rate for subject lane group, expressed as a total for all lanes in lane group (veh/h);
s_o = base saturation flow rate per lane (pc/h/ln);
N = number of lanes in lane group;
f_w = adjustment factor for lane width;
f_{HV} = adjustment factor for heavy vehicles in traffic stream;
f_g = adjustment factor for approach grade;
f_p = adjustment factor for existence of a parking lane and parking activity adjacent to lane group;
f_{bb} = adjustment factor for blocking effect of local buses that stop within intersection area;
f_a = adjustment factor for area type;
f_{LU} = adjustment factor for lane utilization;
f_{LT} = adjustment factor for left turns in lane group;
f_{RT} = adjustment factor for right turns in lane group;
f_{Lpb} = pedestrian adjustment factor for left-turn movements; and
f_{Rpb} = pedestrian-bicycle adjustment factor for right-turn movements.

Factor	Formula	Definition of Variables	Notes
Lane width	$f_w = 1 + \frac{(W - 3.6)}{9}$	W = lane width (m)	$W \geq 2.4$ If W > 4.8, a two-lane analysis may be considered
Heavy vehicles	$f_{HV} = \frac{100}{100 + \% HV(E_T - 1)}$	% HV = % heavy vehicles for lane group volume	$E_T = 2.0$ pc/HV
Grade	$f_g = 1 - \frac{\% G}{200}$	% G = % grade on a lane group approach	$-6 \leq \% G \leq +10$ Negative is downhill
Parking	$f_p = \frac{N - 0.1 - \frac{18N_m}{3600}}{N}$	N = number of lanes in lane group N_m = number of parking maneuvers/h	$0 \leq N_m \leq 180$ $f_p \geq 0.050$ $f_p = 1.000$ for no parking
Bus blockage	$f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N}$	N = number of lanes in lane group N_B = number of buses stopping/h	$0 \leq N_B \leq 250$ $f_{bb} \geq 0.050$
Type of area	$f_a = 0.900$ in CBD $f_a = 1.000$ in all other areas		
Lane utilization	$f_{LU} = v_g/(v_{g1}N)$	v_g = unadjusted demand flow rate for the lane group, veh/h v_{g1} = unadjusted demand flow rate on the single lane in the lane group with the highest volume N = number of lanes in the lane group	

Figure 6 - Adjustment Factors For Saturation Flow Rate

Left turns	Protected phasing: Exclusive lane: $f_{LT} = 0.95$ Shared lane: $f_{LT} = \frac{1}{1.0 + 0.05P_{LT}}$	P_{LT} = proportion of LTs in lane group	See Exhibit C16-1, Appendix C, for nonprotected phasing alternatives
Right turns	Exclusive lane: $f_{RT} = 0.85$ Shared lane: $f_{RT} = 1.0 - (0.15)P_{RT}$ Single lane: $f_{RT} = 1.0 - (0.135)P_{RT}$	P_{RT} = proportion of RTs in lane group	$f_{RT} \geq 0.050$
Pedestrian-bicycle blockage	LT adjustment: $f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$ RT adjustment: $f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$	P_{LT} = proportion of LTs in lane group A_{pbT} = permitted phase adjustment P_{LTA} = proportion of LT protected green over total LT green P_{RT} = proportion of RTs in lane group P_{RTA} = proportion of RT protected green over total RT green	Refer to Appendix D for step-by-step procedure

Figure 7 - Adjustment factors for saturation flow rate

3.5. Capacity

The capacity is based on the concept of saturation flow and saturation flow rate at a signalized intersection. The flow ratio for a given lane group is defined as the ratio of the actual or projected demand flow rate for the lane group (v_i) and the saturation flow rate (s_i). Capacity of a lane group is stated by the equation:

$$c_i = s_i \frac{g_i}{C}$$

Where
-c_i = capacity of lane group I (veh/h)
-s_i= saturation flow rate for lane group I (veh/h)
-g_i/C = effective green ratio for lane group I

3.6. What is v/c Ratio

The volume to capacity ratio, given by the symbol X is typically referred to as degree of saturation, and for a given lane group i, X_i can be computed through this equation:

$$X_i = \left(\frac{v}{c}\right)_i = \frac{v_i}{s_i\left(\frac{g_i}{C}\right)} = \frac{v_i C}{s_i g_i}$$

Where
X_i= (v/c)i= ratio for lane group i
v_i = actual or projected demand flow rate for lane group i (veh/h)
s_i = saturation flow rate for lane i
g_i = effective green time for lane group i
C = cycle length

3.7. Level of Service (LOS)

LOS is directly related to the control delay value. Mean of control delay per vehicle is estimated for each lane group and aggregated for each approach and for the intersection.

LOS	Control Delay per Vehicle (s/veh)
A	≤ 10
B	> 10–20
C	> 20–35
D	> 35–55
E	> 55–80
F	> 80

Figure 8 - LOS criteria

3.8. Calculation of Delay

The calculated delays represent the average control delay experienced by all vehicles that arrive during the analysis period (T = 15mn). The control delay includes movements at slower speeds and stops on intersection approaches as vehicles move up in queue position or slow down up stream ofan intersection. The average control delay per vehicle for a given lane group is given by the equation:

D(Lane group delay)= D1(Uniform Delay)+D2(Incremental Delay)+D3(=0 for this project)

Where

-d = control delay per vehicle (s/veh)

d1=Uniform control delay per vehicle (s/veh)=

$$d_1 = \frac{0.5C\left(1-\frac{g}{C}\right)^2}{1-\left[\min(1,X)\frac{g}{C}\right]}$$

Where;

g = effective green time for lane group
C= cycle length
X = v/c ratio or degree of saturation for lane group
PF = progression adjustment factor

Green Ratio (g/C)	Arrival Type (AT)					
	AT 1	AT 2	AT 3	AT 4	AT 5	AT 6
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.714	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000
f_{PA}	1.00	0.93	1.00	1.15	1.00	1.00
Default, R_p	0.333	0.667	1.000	1.333	1.667	2.000

Figure 9 - Progression adjustment factor for uniform delay calculation

-d2= incremental delay

$$d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8kIX}{cT}}\right]$$

where
T = duration of analysis period (h)
k = incremental delay factor that is dependent on controller
settingsl = upstream filtering/ metering adjustment factor
c = lane group capacity (veh/h)
X = lane group v/c ratio or degree of saturation

Once known the delay of every single approach, it has to be estimated the delay of the junction as a weighted mean of equation:

$$d_A = \frac{\sum d_i v_i}{\sum v_i}$$

where
d_A = delay for Approach A (s/veh),
d_i = delay for lane group i (on Approach A) (s/veh), and
v_i = adjusted flow for lane group i (veh/h).

d3 = initial queue delay, which accounts for delay to all vehicles in analysis period due to initial queue at start of analysis period (s/veh) (considered zero in this project: no queue at the beginning of the observation period). The delay of the intersection is calculated with the next equation.

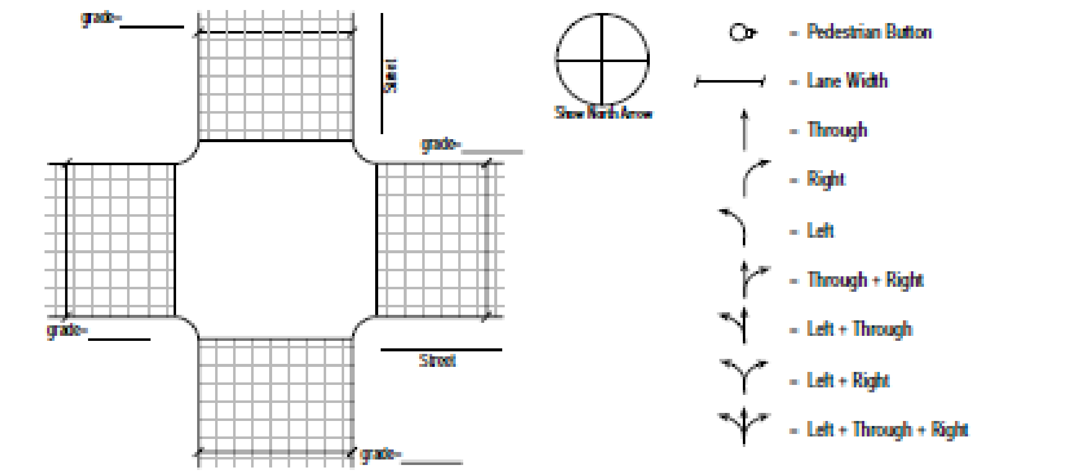
4. Worksheets

All outputs are calculated with the help of excel files of worksheets that is on the below by using formulas. Calculations are determined with Volume, number of vehicles/heavy vehicles, lost time etc. which is given.

INPUT WORKSHEET

General Information		Site Information	
Analyst		Intersection	
Agency or Company		Area Type	❑ CBD ❑ Other
Date Performed		Jurisdiction	
Analysis Time Period		Analysis Year	

Intersection Geometry



Volume and Timing Input

	EB			WB			NB			SB		
	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]	LT	TH	RT[1]
Volume, V (veh/h)												
% heavy vehicles, % HV												
Peak-hour factor, PHF												
Pretimed (P) or actuated (A)												
Start-up lost time, l_1 (s)												
Extension of effective green time, e (s)												
Arrival type, AT												
Approach pedestrian volume,[2] v_{ped} (p/h)												
Approach bicycle volume,[2] v_{bic} (bicycles/h)												
Parking (Y or N)												
Parking maneuvers, N_m (maneuvers/h)												
Bus stopping, N_B (buses/h)												
Min. timing for pedestrians,[3] G_p (s)												

Signal Phasing Plan

	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
DIAGRAM								
Timing	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =	G = Y =

Protected turns | Permitted turns Pedestrian | Cycle length, C = _____ s

Notes

1. RT volumes, as shown, exclude RTOR.
2. Approach pedestrian and bicycle volumes are those that conflict with right turns from the subject approach.
3. Refer to Equation 16-2.

Figure 10 - Input worksheet

The next worksheet is the volume adjustment and saturation flow rate worksheet on the below. Regarding the base saturation flow, we considered a value of 2100veh/h for lanes that have width 4,25 m and 1900 veh/h for 3,5 m lanes used in HCM2000, referred to **US standards.**

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET

General Information

Project Description ____

Volume Adjustment

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)												
Peak-hour factor, PHF												
Adjusted flow rate, v_p = V/PHF (veh/h)												
Lane group												
Adjusted flow rate in lane group, v (veh/h)												
Proportion[1] of LT or RT (P_{LT} or P_{RT})		-			-			-			-	

Saturation Flow Rate (see Exhibit 16-7 to determine adjustment factors)

	EB LT	EB TH	EB RT	WB LT	WB TH	WB RT	NB LT	NB TH	NB RT	SB LT	SB TH	SB RT
Base saturation flow, s_0 (pc/h/ln)												
Number of lanes, N												
Lane width adjustment factor, f_w												
Heavy-vehicle adjustment factor, f_{HV}												
Grade adjustment factor, f_g												
Parking adjustment factor, f_p												
Bus blockage adjustment factor, f_{bb}												
Area type adjustment factor, f_a												
Lane utilization adjustment factor, f_{LU}												
Left-turn adjustment factor, f_{LT}												
Right-turn adjustment factor, f_{RT}												
Left-turn ped/bike adjustment factor, f_{Lpb}												
Right-turn ped/bike adjustment factor, f_{Rpb}												
Adjusted saturation flow, s (veh/h) $s = s_0 N f_w f_{HV} f_g f_p f_{bb} f_a f_{LU} f_{LT} f_{RT} f_{Lpb} f_{Rpb}$												

Notes

1. P_{LT} = 1.000 for exclusive left-turn lanes, and P_{RT} = 1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group.

Figure 11 - Volume adjustment and saturation worksheet

EXHIBIT 16-7. ADJUSTMENT FACTORS FOR SATURATION FLOW RATE[a]

Factor	Formula	Definition of Variables	Notes
Lane width	$f_w = 1 + \frac{(W - 3.6)}{9}$	W = lane width (m)	$W \geq 2.4$ If $W > 4.8$, a two-lane analysis may be considered
Heavy vehicles	$f_{HV} = \frac{100}{100 + \% HV(E_T - 1)}$	% HV = % heavy vehicles for lane group volume	$E_T = 2.0$ pc/HV
Grade	$f_g = 1 - \frac{\% G}{200}$	% G = % grade on a lane group approach	$-6 \leq \% G \leq +10$ Negative is downhill
Parking	$f_p = \frac{N - 0.1 - \frac{18N_m}{3600}}{N}$	N = number of lanes in lane group N_m = number of parking maneuvers/h	$0 \leq N_m \leq 180$ $f_p \geq 0.050$ $f_p = 1.000$ for no parking
Bus blockage	$f_{bb} = \frac{N - \frac{14.4N_B}{3600}}{N}$	N = number of lanes in lane group N_B = number of buses stopping/h	$0 \leq N_B \leq 250$ $f_{bb} \geq 0.050$
Type of area	$f_a = 0.900$ in CBD $f_a = 1.000$ in all other areas		
Lane utilization	$f_{LU} = v_g/(v_{g1}N)$	v_g = unadjusted demand flow rate for the lane group, veh/h v_{g1} = unadjusted demand flow rate on the single lane in the lane group with the highest volume N = number of lanes in the lane group	
Left turns	Protected phasing: Exclusive lane: $f_{LT} = 0.95$ Shared lane: $f_{LT} = \frac{1}{1.0 + 0.05P_{LT}}$	P_{LT} = proportion of LTs in lane group	See Exhibit C16-1, Appendix C, for nonprotected phasing alternatives
Right turns	Exclusive lane: $f_{RT} = 0.85$ Shared lane: $f_{RT} = 1.0 - (0.15)P_{RT}$ Single lane: $f_{RT} = 1.0 - (0.135)P_{RT}$	P_{RT} = proportion of RTs in lane group	$f_{RT} \geq 0.050$
Pedestrian-bicycle blockage	LT adjustment: $f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$ RT adjustment: $f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$	P_{LT} = proportion of LTs in lane group A_{pbT} = permitted phase adjustment P_{LTA} = proportion of LT protected green over total LT green P_{RT} = proportion of RTs in lane group P_{RTA} = proportion of RT protected green over total RT green	Refer to Appendix D for step-by-step procedure

Figure 12 - Adjustment factors for saturation flow rate

In the capacity and LOS worksheet (figure 12), the information and computational results from the input, volume adjustment, and saturation flow rate modules are combined to compute the capacity and v/c of each lane group and the delay and LOS for each lane group and approach and for the intersection as a whole.

CAPACITY AND LOS WORKSHEET

General Information

Project Description ______________________________

Capacity Analysis

Phase number												
Phase type												
Lane group												
Adjusted flow rate, v (veh/h)												
Saturation flow rate, s (veh/h)												
Lost time, t_L (s), $t_L = l_1 + Y - e$												
Effective green time, g (s), $g = G + Y - t_L$												
Green ratio, g/C												
Lane group capacity,[1] $c = s(g/C)$, (veh/h)												
v/c ratio, X												
Flow ratio, v/s												
Critical lane group/phase (√)												
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)												
Total lost time per cycle, L (s)												
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$												

Lane Group Capacity, Control Delay, and LOS Determination

	EB	WB	NB	SB
Lane group				
Adjusted flow rate,[2] v (veh/h)				
Lane group capacity,[2] c (veh/h)				
v/c ratio,[2] X = v/c				
Total green ratio,[2] g/C				
Uniform delay, $d_1 = \frac{0.50\,C\,[1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)				
Incremental delay calibration,[3] k				
Incremental delay,[4] d_2 $d_2 = 900T\left[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}\right]$ (s/veh)				
Initial queue delay, d_3 (s/veh) (Appendix F)				
Uniform delay, d_1 (s/veh) (Appendix F)				
Progression adjustment factor, PF				
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)				
LOS by lane group (Exhibit 16-2)				
Delay by approach, $d_A = \frac{\sum (d)(v)}{\sum v}$ (s/veh)				
LOS by approach (Exhibit 16-2)				
Approach flow rate, v_A (veh/h)				
Intersection delay, $d_I = \frac{\sum (d_A)(v_A)}{\sum v_A}$ (s/veh)		Intersection LOS (Exhibit 16-2)		

Notes

1. For permitted left turns, the minimum capacity is $(1 + P_L)(3600/C)$.
2. Primary and secondary phase parameters are summed to obtain lane group parameters.
3. For pretimed or nonactuated signals, k = 0.5. Otherwise, refer to Exhibit 16-13.
4. T = analysis duration (h); typically T = 0.25, which is for the analysis duration of 15 min.
   I = upstream filtering metering adjustment factor; I = 1 for isolated intersections.

Figure 13 - Capacity and LOS worksheet

And finally, this worksheet used to compute the adjustment factors for pedestrians and bicycles crossing the intersection on permitted left and right turn movements.

SUPPLEMENTAL WORKSHEET FOR PEDESTRIAN-BICYCLE EFFECTS ON PERMITTED LEFT TURNS AND RIGHT TURNS

General Information

Project Description ____

Permitted Left Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)				
Conflicting pedestrian volume,[1] v_{ped} (p/h)				
$v_{pedg} = v_{ped}\ (C/g_p)$				
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \le 1000$) or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if ($1000 < v_{pedg} \le 5000$)				
Opposing queue clearing green,[3,4] g_q (s)				
Effective pedestrian green consumed by opposing vehicle queue, g_q/g_p; if $g_q \ge g_p$ then $f_{Lpb} = 1.0$				
$OCC_{pedu} = OCC_{pedg}\ [1 - 0.5(g_q/g_p)]$				
Opposing flow rate,[3] v_o (veh/h)				
$OCC_r = OCC_{pedu}\ [e^{-(5/3600)v_o}]$				
Number of cross-street receiving lanes,[1] N_{rec}				
Number of turning lanes,[1] N_{turn}				
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$ $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$				
Proportion of left turns,[5] P_{LT}				
Proportion of left turns using protected phase,[6] P_{LTA}				
$f_{Lpb} = 1.0 - P_{LT}(1 - A_{pbT})(1 - P_{LTA})$				

Permitted Right Turns

	EB	WB	NB	SB
Effective pedestrian green time,[1,2] g_p (s)				
Conflicting pedestrian volume,[1] v_{ped} (p/h)				
Conflicting bicycle volume,[1,7] v_{bic} (bicycles/h)				
$v_{pedg} = v_{ped}(C/g_p)$				
$OCC_{pedg} = v_{pedg}/2000$ if ($v_{pedg} \le 1000$), or $OCC_{pedg} = 0.4 + v_{pedg}/10{,}000$ if ($1000 < v_{pedg} \le 5000$)				
Effective green,[1] g (s)				
$v_{bicg} = v_{bic}(C/g)$				
$OCC_{bicg} = 0.02 + v_{bicg}/2700$				
$OCC_r = OCC_{pedg} + OCC_{bicg} - (OCC_{pedg})(OCC_{bicg})$				
Number of cross-street receiving lanes,[1] N_{rec}				
Number of turning lanes,[1] N_{turn}				
$A_{pbT} = 1 - OCC_r$ if $N_{rec} = N_{turn}$ $A_{pbT} = 1 - 0.6(OCC_r)$ if $N_{rec} > N_{turn}$				
Proportion of right turns,[5] P_{RT}				
Proportion of right turns using protected phase,[8] P_{RTA}				
$f_{Rpb} = 1.0 - P_{RT}(1 - A_{pbT})(1 - P_{RTA})$				

Notes

1. Refer to Input Worksheet.
2. If intersection signal timing is given, use Walk + flashing Don't Walk (use G + Y if no pedestrian signals). If signal timing must be estimated, use (Green Time – Lost Time per Phase) from Quick Estimation Control Delay and LOS Worksheet.
3. Refer to supplemental worksheets for left turns.
4. If unopposed left turn, then $g_q = 0$, $v_o = 0$, and $OCC_r = OCC_{pedu} = OCC_{pedg}$.
5. Refer to Volume Adjustment and Saturation Flow Rate Worksheet.
6. Ideally determined from field data; alternatively, assume it equal to (1 – permitted phase f_{LT})/0.95.
7. If $v_{bic} = 0$ then $v_{bicg} = 0$, $OCC_{bicg} = 0$, and $OCC_r = OCC_{pedg}$.
8. P_{RTA} is the proportion of protected green over the total green, $g_{prot}/(g_{prot} + g_{perm})$. If only permitted right-turn phase exists, then $P_{RTA} = 0$.

Figure 14 - Supplemental worksheet for pedestrian-bicycle effects on permitted left and right turns

5. Optimization methods

After input data determination and calculation of delay, we are trying to find the best solution to reduce the delay. And there are some methodologies to calculate delay; Minimum Cycle and Optimum Cycle method and Enumerative Methods (C=Constant and g1/g2=Constant)

5.1. Minimum Cycle

Minimum cycle is the smallest value of Cycle length that satisfies all the capacity constraints, it's computed when known the values of flow ratios and it's obtainable through the following equation:

$$C_{min} = \frac{\sum_i l_i}{1 - \sum_i y_i}$$

Where
l_i= lost time for lane group i
y_i=critical flow ratio for lane group I
It is possible to compute the minimum effective green times, obtained multiplying the minimum cycle and the critical flow ratio when calculated the value of the Minimum Cycle:

$$g_i = C_{min} y_i$$

5.2. Optimum Cycle

In order to find the optimum cycle, we use webster formula. This formula based on vehicles on queue are arriving according to uniform arrival principles and cycle length is computed according to average value of flow. The optimum cycle can be obtained from the equation:

$$C_{opt} = \frac{(1{,}5x \sum_i l_i) + 5}{1 - \sum_i y_i}$$

Effective green time could be found also with the formula on below when optimum cycle is calculated:

$$g_i = \frac{y_i}{\sum_i y_i}\left(C_{opt} - \sum_i l_i\right)$$

Actually, after computing and applying the values of the Optimal cycle and recalculating the delays at the approaches and intersections we obtained substantial improvements.

5.3. Enumerative method

The enumerative method has been set in two different ways:
-Enumerative method 1(C = constant): The cycle length is been kept fixed and equal to the actual cycle length measured in the field while the green shares have been varying. From this procedure, we obtained diagrams showing how the delay varies according to the g1/g2 ratio.
-Enumerative method 2(g1/g2= constant): in this case is the green share that is been kept fixed and equal to the optimal one while the cycle length has been varying. From this procedure, we obtained diagrams showing how the delay varies according to the variations of the cycle length.

6. Input Data Determination

The object of the project is the design of signal settings of intersections on the real situation. The design is carried out based on computations of impacts of the project in the current conditions by using the intersection given with in the field observations that is given with land use, geometry,traffic counts in the field. Specification and calibrations are carried out with the help of tool that HCMMethod.

Table 2 - List of Intersections

Intersection Number	Approaches				Number of Phases
	East Bound	West Bound	North Bound	South Bound	
1	Via Prenestina	Via Prenestina	Largo Irpinia	Viale Ronchi	3
2	Via Prenestina	Via Prenestina	Largo Irpinia	Via Dignano d'Istria	2
3	Via Prenestina	Via Prenestina	Via Olevano Romano	-	2
4	Via Prenestina	Via Prenestina	Via Tor de'Schiavi	Viale della Serenissima	3

Figure 15 – Whole Intersections

6.1. Intersection-1: Via Prenestina & Via Dignano D'istria

This Junction is VIA PRENESTINA – LARGO IRPINIA – VIA DIGNANO D'ISTRIA which is composed by 3 approaches has a signal setting composed by Two phases

Figure 16 - The Movement of Intersection Via Prenestina & Via Dignano D'istria

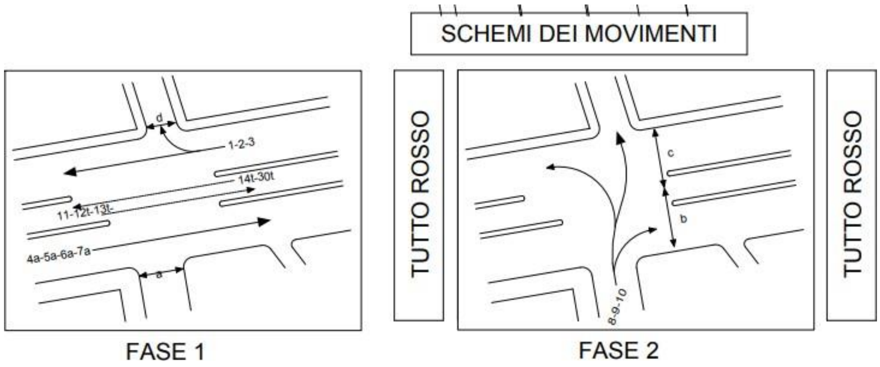


Figure 17- The Phases of Intersection Via Prenestina & Via Dignano D'istria

Table 3 - The Input Parameters (Signalization) Via Prenestina & Via Dignano D'istria

	PHASE 1	PHASE 2
green	76	44
yellow	4	4
red	52	84
cycle length	132	

Table 4 - The Input Parameters (Geometric) Via Prenestina & Via Dignano D'istria

1st Junction				
Direction	Street Name	No. of Lane	Lane Width [m]	Grade %
WB	Via Prenestina (W)	2	3.5	0
EB	Via Prenestina (E)	2	3.5	0
NB	Largo Irpinia	3	3	0
SB	Via Diagnano d'Istria	0	0.0	0

Table 5 - The Volume and Timing Inputs Via Prenestina & Via Dignano D'istria

VOLUME AND TIMING INPUT	EB			WB			NB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT
N of lanes		2			2			3	
Lane width (m)		3.5			3.5			3	
Volume (veh/h)	0	1370	0	0	1018	99	139	265	123
% heavy vehicles HV	0.0%	2.5%	0.0%	0.0%	1.5%	0.0%	0.0%	1.7%	1.5%
% vans	0.0%	0.7%	0.0%	0.0%	1.6%	0.0%	0.0%	0.6%	1.3%
%buses	0.0%	1.8%	0.0%	0.0%	2.1%	0.0%	0.0%	1.3%	0.0%
PHF		0.9			0.9			0.9	
Pretimed(P) or Actuated(A)		P			P			P	
Start up lost time (s)		3			3			3	
Clearance lost time (s)		3			3			3	
Arrival type, AT		3			3			3	
Approach pedestrian volume (p/h)		20			20			20	
Approach bicycles volume (bicycles/h)		10			10			10	
Parking (Y or N)		N			N			N	
Parking maneuvers, Nm		0			0			0	
Bus stopping, Nb (buses/h)		7			0			12	
Min timeing for pedestrian, Gp (s) (pg 16-5)		13.66			20.58			17.35	
PED/CYCLE		***0.366666667***			***0.366666667***			***0.366666667***	

Table 6 - Determining Adjusted Saturation Flow Rate Via Prenestina & Via Dignano D'istria

VOLUME AND SATURATION FLOWRATE ADJUSTMENT	EB			WB			NB		
VOLUME ADJUSTMENT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume	0	1370	0	0	1018	99	139	265	123
PHF	0.9	0.9	0.9		0.9	0.9	0.9	0.9	0.9
adjusted flow rate	0	1522.222222	0.00		1131.11	110.00	154.44	294.44	136.67
adjusted flow rate in lane groups	0	1522.222222	0.00		1241.11			585.56	
Proportion of LT or RT	0%	100%	0%		91%	9%	26%	50%	23%
SATURATION FLOW RATE AND ADJUSTMENT FACTOR									
base saturation flow, s0					2100				
n of lanes		2			2			3	
lane width adj factor fw		0.99			0.99			0.99	
heavy vehs adjustment factor fHV		1.0			1.0			1.0	
Parking adjustment factor, fp		1			1			1	
Bus blockage adjustment facto, fbb		1.00			1			0.98	
Area type adjustment factor, fa		1			1			1	
Lane utilization adjustment factor, fLU		1			1			1	
Left turn adjustment factor, fLT		1			1			1.0	
Right turn adjustment factor, fRT		1			1.00			1.00	
Left turn ped/bike adjustment factor, fLpb		1			1			1	
Right turn ped/bike adjustment factor, fRpb		1.00			1.00			1.00	
Adjusted saturation flow, s (veh/h)		3945.67			3944.54			5844.05	

Table 7 - Capacity Analysis Via Prenestina & Via Dignano D'istria

CAPACITY AND LOS WORKSHEET			
GENERAL INFORMATION			
Project Description			
Capacity Analysis			
	EB	WB	NB
Phase number	1		2
Lane group			
Adjusted flow rate, v (veh/h)	1522	1241	586
Saturation flow rate, s (veh/h)	3946	3945	5844
Lost time, t_L (s),	6.0	6.0	6.0
Effective green time, g (s), $g = G + Y - t_L$	74.0	74.0	42.0
Green ratio, g/C	0.56	0.56	0.32
Lane group capacity, [1] c = s(g/C), (veh/h)	2212.0	2211.3	1859.5
v/c ratio, X	0.69	0.56	0.31
Flow ratio, v/s	0.39	0.31	0.10
Critical lane group/phase (√)	V		V
Sum of flow ratios for critical lane groups, Y_c	0.49		
Total lost time per cycle, L (s)	12.0		
Critical flow rate to capacity ratio, X_c	0.53		

Table 8 - Level of Service Via Prenestina & Via Dignano D'istria

Lane Group Capacity, Control Delay and LOS Determination			
	EB	WB	NB
Lane group			
Adjusted flow rate, [2] v (veh/h)	1522	1241	586
Lane group capacity, [2] c (veh/h)	2212	2211	1859.5
v/c ratio, [2] X = v/c	0.69	0.56	0.31
Total green ratio, [2] g/C	0.56	0.56	0.32
Uniform delay $d_1 = \frac{0.50C\left(1-(g/c)\right)^2}{1-\left[\min(1,X)g/C\right]} \cdot (s/veh)$	20.75	18.59	34.10
Incremental delay calibration, [3] k	0.50	0.50	0.50
Incremental delay $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8klX}{cT}}\right] \cdot (S/veh)$	1.77	1.04	0.44
Initial queue delay, d_3 (s/veh) (Appendix F)	-	-	-
Uniform delay, d_1 (s/veh) (Appendix F)	-	-	-
Progression adjustment factor, PF	1	1	1
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	22.52	19.63	34.54
LOS by lane group (Exhibit 16-2)	C	C	C
Delay by approach $d_A = \frac{\Sigma(d)(v)}{\Sigma v}(s/veh)$	22.52	19.63	34.54
LOS by approach (Exhibit 16-2)	C	C	C
Approach flow rate, v_A (veh/h)	1522.22	1241.11	586
Intersection delay, $d_I = \frac{\Sigma(d_A)(v_A)}{\Sigma v_A}(s/veh)$	23.55	Intersection LOS (Exhbit 16-2)	C

Table 9 - Minimum Cycle Length Via Prenestina & Via Dignano D'istria

Minimum Cycle				
Cmin	23.4	s		
g	9.0		9.0	2.3
r	14.3		14.3	21.0
g/C	0.39		0.39	0.10
c	1523		1523	587
x	1.00		0.82	1.00
d1	7.2		6.4	10.5
d2	23.1		4.9	37.1
d1+d2	30.2		11.4	47.7
Approach	30.2		11.4	47.7
davg	(intersecti	26.30	C	

Table 10 - Optimum Cycle Length Via Prenestina & Via Dignano D'istria

Webster's Optimal Policy				
C	44.8	s		
g	26.0		26.0	6.8
r	18.8		18.8	38.0
g/C	0.58		0.58	0.15
c	2292		2291	883
x	0.66		0.54	0.66
d1	6.41		5.74	17.94
d2	1.5		0.9	3.9
d1+d2	7.9		6.7	21.9
Approach	7.945547544		6.666506866	21.86895677
davg	(intersecti	10	B	

Table 11 - For Pedestrian and Bicycle Effecting Permitted LT and RT Via Prenestina & Via Dignano D’istria

SUPPLEMENTAL WORKSHEET FOR PEDESTRIAN -BYCICLE EFFECTS ON PERMITTED TURNS						
PERMITTED RIGHT TURNS		WB			NB	
Effective pedestrian green time, gp		20.58			17.35	
Conflicting pedestrian volume, vped [p/h]		20			20	
Conflicting bicycle volume, vbic [p/h]		10			10	
Vpedg= vped*(C/gp)		128.29			152.16	
OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤		0.06			0.08	
Effective green, g [s]		74			42	
Vbicg= vbicg*(C/gp)		64.144			76.081	
OCCbicg = 0.02 + vbicg/2700		0.044			0.048	
OCCr = OCCpedg + OCCbicg – (OCCpedg)(OCCbicg)		0.105			0.121	
Number of cross-street receiving lanes, Nrec		2			3	
Number of turning lanes, Nturn		1			2	
ApbT = 1 – 0.6(OCCr) if Nrec > Nturn		0.937			0.928	
Proportion of right turns, PRT		9%			23%	
PRTA		98%			0.961538462	
fRpb = 1.0 – PRT(1 – ApbT)(1 – PRTA)		1.000			0.999	

Table 12 - The results of the analysis Via Prenestina & Via Dignano D’istria

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	132	23.4	44.8	44.4
Intersection Delay	23.55	26.30	10	9.89
Level of service	C	C	B	A
g1	76	9	26	25.9
g2	44	2.3	6.8	6.5
g1/g2	1,73	3.91	3.82	3.98

6.2. Intersection-2: Via Prenestina& Via Olevano Romano

This Junction is Via Prenestina – Via Olevano Romano which is composed by 3 approaches has a signal setting composed by two phases.

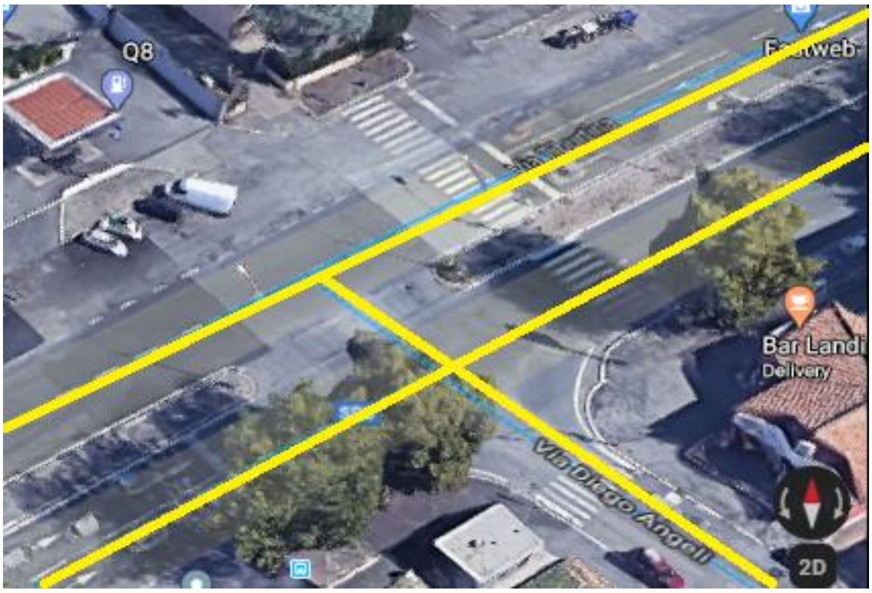


Figure 18 - The Movement of Intersection Via Prenestina & V.Olevano Romano

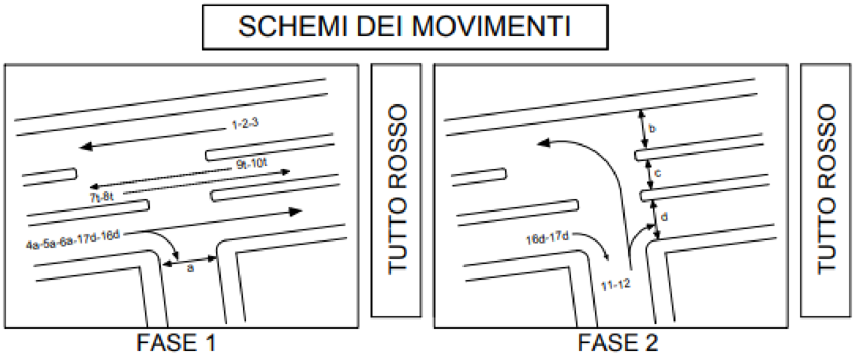


Figure 19 - The Phases of Intersection Via Prenestina & V.Olevano Romano

Table 13 - The Input Parameters (Signalization) Via Prenestina & V.Olevano Romano

	Phase I	Phase II
green	75	45
yellow	4	4
red	49	79
cycle length	128	

Table 14 - The Input Parameters (Geometric) Via Prenestina & V.Olevano Romano

2nd Junction				
Direction	Street Name	No. of Lane	Lane Width [m]	Grade %
WB	Via Prenestina (W)	3	3.5	0
EB	Via Prenestina (E)	2	3.5	0
NB	Via Olevano Romano	2	3	0
SB	-	0	0.0	0

Table 15 - The Volume and Timing Inputs Via Prenestina & V.Olevano Romano

VOLUME AND TIMING INPUT	EB			WB			NB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT
N of lanes		3			2			2	
Lane width (m)		3.5			3.5			3	
Volume (veh/h)	0	1184	310	0	950	0	164	0	61
% heavy vehicles HV	0.0%	2.2%	0.0%	0.0%	1.9%	0.0%	0.0%	0.0%	0.0%
% vans	0.0%	0.9%	0.0%	0.0%	2.1%	0.0%	0.0%	0.0%	0.0%
%buses	0.0%	1.3%	0.0%	0.0%	2.3%	0.0%	0.0%	0.0%	0.0%
PHF		0.9			0.9			0.9	
Pretimed(P) or Actuated(A)		P			P			P	
Start up lost time (s)		3			3			3	
Clearance lost time (s)		3			3			3	
Arrival type, AT		3			3			3	
Approach pedestrian volume (p/h)		20			20			20	
Approach bicycles volume (bicycles/h)		10			10			10	
Parking (Y or N)		N			N			N	
Parking maneuvers, Nm		0			0			0	
Bus stopping, Nb (buses/h)		7			0			12	
Min timeing for pedestrian, Gp (s) (pg 16-5)		16.58	16.73		20.58			19.68	
PED/CYCLE		***0.355555556***			***0.355555556***			***0.355555556***	

Table 16 - Determining Adjusted Saturation Flow Rate Via Prenestina & V.Olevano Romano

VOLUME AND SATURATION FLOWRATE ADJUSTMENT	EB			WB			NB		
VOLUME ADJUSTMENT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume	0	1184	310	0	950	0	164	0	61
PHF	0.9	0.9	0.9		0.9	0.9	0.9	0.9	0.9
adjusted flow rate	0	1315.555556	344.44		1055.56	0.00	182.22	0.00	67.78
adjusted flow rate in lane groups	0	1315.555556	344.44		1055.56			250.00	
Proportion of LT or RT	0%	79%	21%		100%	0%	73%	0%	27%
SATURATION FLOW RATE AND ADJUSTMENT FACTOR									
base saturation flow, s0					2100				
n of lanes		2	1		2			2	
lane width adj factor fw		0.99	0.99		0.99			0.99	
heavy vehs adjustment factor fHV		1.0	1.0		1.0			1.0	
Parking adjustment factor, fp		1	1		1			1	
Bus blockage adjustment facto, fbb		1.00	0.97		1			0.98	
Area type adjustment factor, fa		1	1		1			1	
Lane utilization adjustment factor, fLU		1	1		1			1	
Left turn adjustment factor, fLT		1	1		1			1.0	
Right turn adjustment factor, fRT		1	0.989732		1.00			1.0	
Left turn ped/bike adjustment factor, fLpb		1	1		1			1	
Right turn ped/bike adjustment factor, fRpb		1.00	1.00		1.00			1.00	
Adjusted saturation flow, s (veh/h)		3944.80	1796.97		3944.93			3663.43	

Table 17 - Capacity Analysis Via Prenestina & V.Olevano Romano

CAPACITY AND LOS WORKSHEET				
GENERAL INFORMATION				
Project Description				
Capacity Analysis				
	EB		**WB**	**NB**
Phase number	**1**			**2**
Lane group				
Adjusted flow rate, v (veh/h)	1316	344	1056	250
Saturation flow rate, s (veh/h)	3945	1797	3945	3663
Lost time, t_L (s),	6.0	6.0	6.0	6.0
Effective green time, g (s), $g = G + Y - t_L$	73.0	73.0	73.0	43.0
Green ratio, g/C	0.57	0.57	0.57	0.34
Lane group capacity, [1] c = s(g/C), (veh/h)	2249.8	1024.8	2249.8	1230.7
v/c ratio, X	0.58	0.34	0.47	0.20
Flow ratio, v/s	0.33	0.19	0.27	0.07
Critical lane group/phase (√)	**V**			**V**
Sum of flow ratios for critical lane groups, Y_c	**0.40**			
Total lost time per cycle, L (s)	**12.0**			
Critical flow rate to capacity ratio, X_c	**0.44**			

Table 18 - Level of Service Via Prenestina & V.Olevano Romano

Lane Group Capacity, Control Delay and LOS Determination				
	EB		WB	NB
Lane group				
Adjusted flow rate, [2] v (veh/h)	1316	344	1056	250
Lane group capacity, [2] c (veh/h)	2250	1025	2250	1230.7
v/c ratio, [2] X = v/c	0.58	0.34	0.47	0.20
Total green ratio, [2] g/C	0.57	0.57	0.57	0.34
Uniform delay $d_1 = \frac{0.50C\left(1-g/C\right)^2}{1-\min(1,X)g/C}$ (s/veh)	17.73	14.62	16.13	30.29
Incremental delay calibration, k	0.50	0.50	0.50	0.50
Incremental delay $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8klX}{cT}}\right]$ (s/veh)	1.12	0.89	0.71	0.37
Initial queue delay, d_3 (s/veh) (Appendix F)	-	-	-	-
Uniform delay, d_1 (s/veh) (Appendix F)	-	-	-	-
Progression adjustment factor, PF	1	1	1	1
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	18.85	15.51	16.84	30.66
LOS by lane group (Exhibit 16-2)	C	B	B	C
Delay by approach $d_A = \frac{\Sigma d v}{\Sigma v}$ (s/veh)	18.85	15.51	16.84	30.66
LOS by approach (Exhibit 16-2)	C	B	B	C
Approach flow rate, v_A (veh/h)	1315.56	344.44	1055.56	250
Intersection delay, $d_I = \frac{\Sigma d_A v_A}{\Sigma v_A}$ (s/veh)	18.74		Intersection LOS (Exhbit 16-2)	B

Table 19 - Minimum Cycle Length Via Prenestina & V.Olevano Romano

Minimum Cycle					
Cmin	20.1	s			
g		6.7	6.7	6.7	1.4
r		13.4	13.4	13.4	18.7
g/C		0.33	0.33	0.33	0.07
c		1316	599	1316	251
x		1.00	0.58	0.80	1.00
d1		6.7	5.5	6.1	9.3
d2		24.8	4.0	5.2	56.8
d1+d2		31.5	9.5	11.3	66.2
Approach		31.5	9.5	11.3	66.2
davg	(intersecti	24.70	C		

Table 20 - Optimum Cycle Length Via Prenestina & V.Olevano Romano

Webster's Optimal Policy					
C	38.5	s			
g		22.0	22.0	22.0	4.5
r		16.5	16.5	16.5	34.0
g/C		0.57	0.57	0.57	0.12
c		2252	1026	2252	429
x		0.58	0.34	0.47	0.58
d1		5.32	4.38	4.84	16.09
d2		1.1	0.9	0.7	5.7
d1+d2		6.4	5.3	5.5	21.8
Approach		6.433838	5.271298	5.54160621	21.81048182
davg	(intersecti	7	A		

Table 21 - For Pedestrian and Bicycle Effecting Permitted LT and RT Via Prenestina & V.Olevano Romano

SUPPLEMENTAL WORKSHEET FOR PEDESTRIAN -BYCICLE EFFECTS ON PERMITTED TURNS						
PERMITTED RIGHT TURNS		EB			NB	
Effective pedestrian green time, gp		16.73			19.68	
Conflicting pedestrian volume, vped [p/h]		20			20	
Conflicting bicycle volume, vbic [p/h]		10			10	
Vpedg= vped*(C/gp)		153.06			130.06	
OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤		0.08			0.07	
Effective green, g [s]		73			43	
Vbicg= vbicg*(C/gp)		76.532			65.030	
OCCbicg = 0.02 + vbicg/2700		0.048			0.044	
OCCr = OCCpedg + OCCbicg – (OCCpedg)(OCCbicg)		0.121			0.106	
Number of cross-street receiving lanes, Nrec		3			2	
Number of turning lanes, Nturn		1			1	
ApbT = 1 – 0.6(OCCr) if Nrec > Nturn		0.927			0.936	
Proportion of right turns, PRT		21%			27%	
PRTA		98%			0.961538462	
fRpb = 1.0 – PRT(1 – ApbT)(1 – PRTA)		1.000			0.999	

Table 22 - The results of the analysis Via Prenestina & V.Olevano Romano

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	128	20.1	38.5	52.1
Intersection Delay	18.74	24.70	7	6.93
Level of service	B	C	A	A
g1	75	6.7	22	34.3
g2	45	1.4	4.5	5.7
g1/g2	1.67	4.79	4.89	6.02

6.3. Intersection-3: Via Prenestina & Via Tor de'Schiavi

This intersection is composed by 4 approaches as East Bound, West Bound, South Bound and North Bound; Via Prenestina from East to West Bound, Via Tor de'Schiavi from North Bound and Viale della Serenissima South Bound. The intersection has signal setting composed by 3 phases, in first phases there are movements to left, through and right from West to East Bound and vice versa. In Second phase includes just movements that are from North Bound to left, through and right. In third phase, there are movements from South Bound to left, through and right.

Figure 20 - Intersection Via Prenestina & Via Tor de'Schiavi

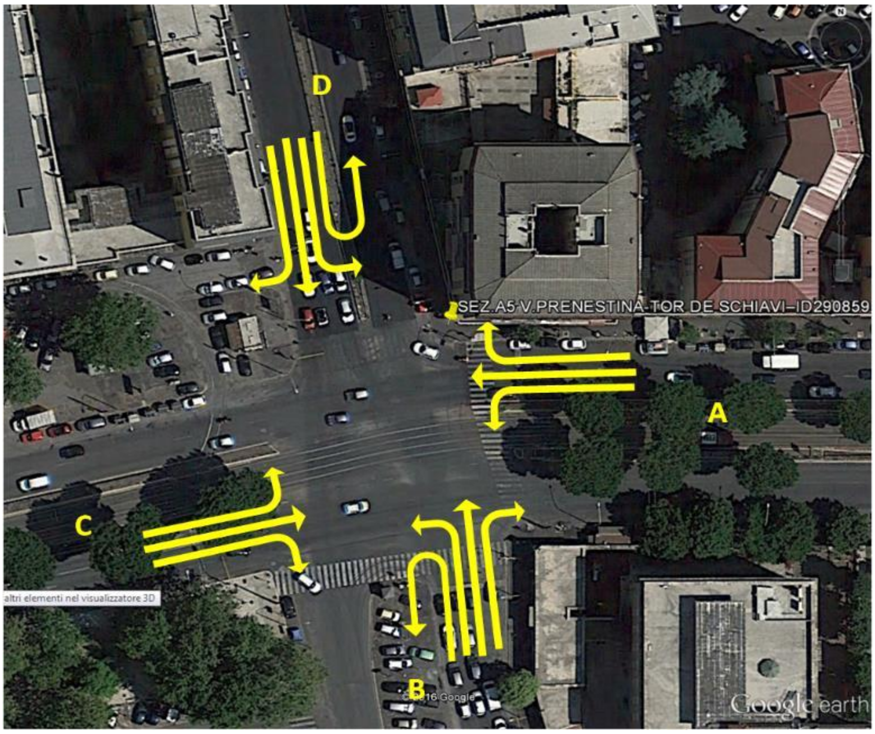


Figure 21 - The Movement of Intersection Via Prenestina & Via Tor de'Schiavi

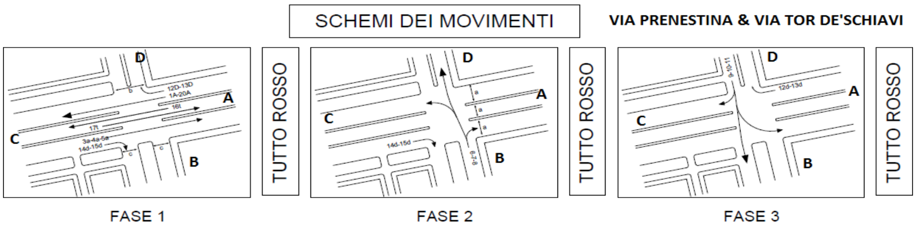


Figure 22 - The Phases of Intersection Via Prenestina & Via Tor de'Schiavi

Table 23 - The Input Parameters (Signalization) Via Prenestina & Via Tor de'Schiavi

	PHASE 1	PHASE 2	PHASE 3
green	48	42	36
yellow	4	4	4
red	80	86	92
cycle length	132		

Table 24 - The Input Parameters (Geometric) Via Prenestina & Via Tor de'Schiavi

3rd Junction				
Direction	Street Name	No. of Lane	Lane Width [m]	Grade %
WB	Via Prenestina (W)	3	3.5	0
EB	Via Prenestina (E)	3	3.5	0
NB	Via Tor de'Schiavi	3	3.5	0
SB	Viale della Serenissima	3	3.5	0

Table 25 - The Volume and Timing Inputs Via Prenestina & Via Tor de'Schiavi

VOLUME AND TIMING INPUT	EB			WB				NB			SB		
	LT	TH	RT	LT		TH	RT	LT	TH	RT	LT	TH	RT
N of lanes	3			3				3			3		
Lane width (m)	3.5			3.5				3.5			3.5		
Volume (veh/h)	0	810	85	0		556	211	398	609	60	175	616	99
% heavy vehicles HV	0.0%	4.5%	1.9%	0.0%		8.3%	1.2%	0.7%	3.1%	0.7%	2.0%	2.7%	0.1%
% vans	0.0%	6.7%	1.8%	0.0%		17.1%	4.8%	2.4%	6.1%	0.7%	2.7%	8.5%	0.2%
%buses	0.0%	6.4%	0.0%	0.0%		8.0%	0.8%	0.2%	2.9%	0.0%	0.6%	3.1%	0.0%
PHF		0.9				0.9			0.9			0.9	
Pretimed(P) or Actuated(A)		P				P			P			P	
Start up lost time (s)		3				3			3			3	
Clearance lost time (s)		3				3			3			3	
Arrival type, AT		3				3			3			3	
Approach pedestrian volume (p/h)		20				20			20			20	
Approach bicycles volume (bicycles/h)		10				10			10			10	
Parking (Y or N)		N				N			N			N	
Parking maneuvers, Nm		0				0			0			0	
Bus stopping, Nb (buses/h)		7				0			12			20	
Min timeing for pedestrian, Gp (s) (pg 16-5)		16.58				26.41			16.16			0.00	
PED/CYCLE		0.366666667				0.36666667			0.36666667			0.36666667	

Table 26 - Determining Adjusted Saturation Flow Rate Via Prenestina & Via Tor de'Schiavi

VOLUME AND SATURATION FLOWRATE ADJUSTMENT	EB			WB				NB			SB		
VOLUME ADJUSTMENT	LT	TH	RT	LT		TH	RT	LT	TH	RT	LT	TH	RT
Volume	0	810	85	0		556	211	398	609	60	175	616	99
PHF		0.9	0.9			0.9	0.8	0.9	0.9	0.7	0.9	0.9	0.9
adjusted flow rate		922.5512528	96.81			617.78	234.44	456.12	697.92	68.76	191.37	680.16	109.31
adjusted flow rate in lane groups		922.5512528	96.81			617.78	234.44	1154.04		68.76	191.37	789.47	
Proportion of LT or RT		91%	9%			72%	28%	37%	57%	6%	20%	69%	11%
SATURATION FLOW RATE AND ADJUSTMENT FACTOR													
base saturation flow, s0	2100												
n of lanes		2	1			2	1	2		1	1	2	
lane width adj factor fw		0.99	0.99			0.99	0.99		0.99	0.99	0.99	0.99	
heavy vehs adjustment factor fHV		1.0	1.0			1.0	1.0		1.0	1.0	1.0	1.0	
Parking adjustment factor, fp		1	1			1	1		1	1	1	1	
Bus blockage adjustment facto, fbb		0.99	0.97			1	1		0.98	0.95	0.92	0.96	
Area type adjustment factor, fa		1	1			1	1		1	1	1	1	
Lane utilization adjustment factor, fLU		1	1			1	1		1	1	1	1	
Left turn adjustment factor, fLT		1	1			1	1		1.0	1	0.95	1	
Right turn adjustment factor, fRT		1	0.85			1.00	0.85		1.00	0.85	1.00	0.98	
Left turn ped/bike adjustment factor, fLpb		1	1			1	1		1	1	1	0.843	
Right turn ped/bike adjustment factor, fRpb		1.00	1.00			0.99	0.99		1.00	1.00	1	1	
Adjusted saturation flow, s (veh/h)	3874.06		1538.07	3914.21			1577.12	3763.00		1505.76	1633.18	3138.77	

Table 27 - Capacity Analysis Via Prenestina & Via Tor de'Schiavi

CAPACITY AND LOS WORKSHEET										
GENERAL INFORMATION										
Project Description										
Capacity Analysis										
	EB		WB		EB	NB		SB		WB
Phase number	1	1	1	1	2	2	2	3	3	3
Lane group										
Adjusted flow rate, v (veh/h)	923	97	618	234	97	1154	69	789.47	191.37	234
Saturation flow rate, s (veh/h)	3874	1538	3914	1577	1538	3763	1506	3139	1633	1577
Lost time, t_L (s),	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
Effective green time, g (s), $g = G + Y - t_L$	46.0	46.0	46.0	46.0	40.0	40.0	40.0	34.0	34.0	34.0
Green ratio, g/C	0.35	0.35	0.35	0.35	0.30	0.30	0.30	0.26	0.26	0.26
Lane group capacity,[1] c = s(g/C), (veh/h)	1350.1	536.0	1364.0	549.6	466.1	1140.3	456.3	808.5	420.7	406.23
v/c ratio, X	0.68	0.18	0.45	0.43	0.00	1.01	0.15	0.98	0.45	0.58
Flow ratio, v/s	0.24	0.06	0.16	0.15	0.06	0.31	0.05	0.25	0.12	0.15
Critical lane group/phase (√)	V					V		v		
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)	0.80									
Total lost time per cycle, L (s)	12.0									
Critical flow rate to capacity ratio,X_c $X_c = (Y_c)(C)/(C - L)$	0.88									

Table 28 - Level of Service Via Prenestina & Via Tor de'Schiavi

Lane Group Capacity, Control Delay and LOS Determination												
	EB			WB			NB			SB		
Lane group												
Adjusted flow rate,[2] v (veh/h)		923	97		618	234		1154	69	191	789.47	
Lane group capacity,[2] c (veh/h)		1350	536		1364	550		1140.3	456.3	421	808.5	
v/c ratio,[2] X = v/c		0.68	0.18		0.45	0.43		1.01	0.15	0.45	0.98	
Total green ratio,[2] g/C		0.35	0.35		0.35	0.35		0.30	0.30	0.26	0.26	
Uniform delay $d_1 = \frac{0.50C(1-g/c)^2}{1-[\min(1,X)g/C]}$ s/veh		36.77	29.90		33.27	32.91		46.00	33.59	41.21	48.60	
Incremental delay calibration,[3] k		0.50	0.50		0.50	0.50		0.50	0.50	0.50	0.50	
Incremental delay $d_2 = 900T\left[(X-1)+\sqrt{(X-1)^2+\frac{8klX}{cT}}\right]$ (s/veh)		2.82	0.74		1.09	2.41		29.66	0.70	3.52	26.43	
Initial queue delay, d_3 (s/veh) (Appendix F)		-	-		-	-		-	-	-	-	
Uniform delay, d_1 (s/veh) (Appendix F)		-	-		-	-		-	-	-	-	
Progression adjustment factor, PF		1	1		1	1		1	1	1	1	
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)		39.59	30.64		34.35	35.32		75.66	34.29	44.73	75.04	
LOS by lane group (Exhibit 16-2)		D	C		D	D		F	D	E	F	
Delay by approach $d_A = \frac{\Sigma d(v)}{\Sigma v}$ s/veh	38.74			34.62			73.33			69.12		
LOS by approach (Exhibit 16-2)	D			C			E			E		
Approach flow rate, v_A (veh/h)	1019.36			852.22			1223			980.8		
Intersection delay, $d_I = \frac{\Sigma d_A(v_A)}{\Sigma vA}$ s/veh	55.57			Intersection LOS (Exhbit 16-2)						E		

Table 29 - Minimum Cycle Length Via Prenestina & Via Tor de'Schiavi

Minimum Cycle										
Cmin		55.4	s					0.0	14.0	
g		12.9	12.9		12.9	12.9	16.5	16.5	14.0	14.0
r		42.5	42.5		42.5	42.5	38.9	38.9	41.3	41.3
g/C		0.23	0.23		0.23	0.23	0.30	0.30	0.25	0.25
c		900	357		909	366	1120	448	414	795
x		1.00	0.27		0.68	0.64	1.00	0.15	0.47	1.00
d1		21.2	17.4		19.4	19.2	19.4	14.3	17.5	20.7
d2		30.0	1.8		4.1	8.4	26.9	0.7	3.8	31.9
d1+d2	(lane grou	51.2	19.2		23.5	27.5	46.3	15.0	21.3	52.6
Approach		48.2			24.6		44.6		46.4	
davg	(intersecti	41.68	D							

Table 30 - Optimum Cycle Length Via Prenestina & Via Tor de'Schiavi

Webster's Optimal Policy										
C		106.1	s							
g		27.9	27.9		27.9	27.9	35.8	35.8	30.4	30.4
r		78.2	78.2		78.2	78.2	70.3	70.3	75.7	75.7
g/C		0.26	0.26		0.26	0.26	0.34	0.34	0.29	0.29
c		1019	405		1030	415	1268	507	468	900
x		0.88	0.23		0.60	0.57	0.88	0.13	0.42	0.88
d1		37.53	30.71		34.21	33.86	33.20	24.40	30.64	36.14
d2		11.0	1.4		2.6	5.5	9.1	0.5	2.7	12.3
d1+d2	(lane grou	48.6	32.1		36.8	39.4	42.3	24.9	33.4	48.4
Approach		46.98609			37.50767		41.35348766		45.42876	
davg	(intersecti	43	D							

Table 31 - The results of the analysis Via Prenestina & Via Tor de'Schiavi

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	132	55.4	106.1	105.4
Intersection Delay	55.57	41.68	43	27.93
Level of service	E	D	D	C
g1	48	12.9	27.9	49.5
g2	42	16.5	35.8	43.8
g3	36	14	30.4	30.4
g1/g2	1.14	0.782	0.779	1.130

6.4 Intersection-4: Via Prenestina & Via Giacomo Bresadola

The intersection has 2 approaches, NB and EB, the road is Via Prenestina,. The signal setting for this junction is composed by 2 phases in which every movement is allowed for the approaches with green light.

Figure 23 - Intersection Via Prenestina & Via Giacomo Bresadola

Figure 24 - The Movement of Intersection Via Prenestina & Via Giacomo Bresadola

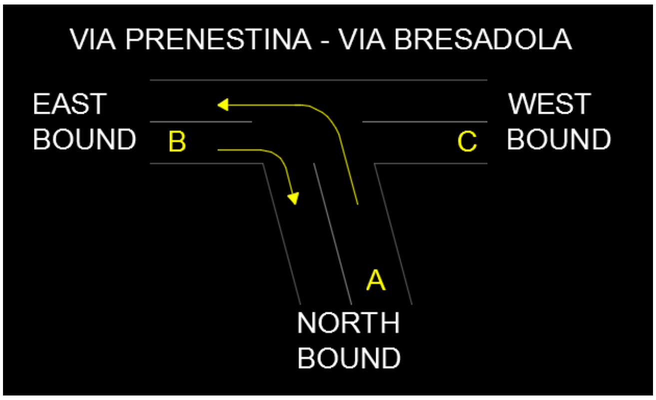


Figure 25 - The Movement of Intersection Via Prenestina & Via Giacomo Bresadola

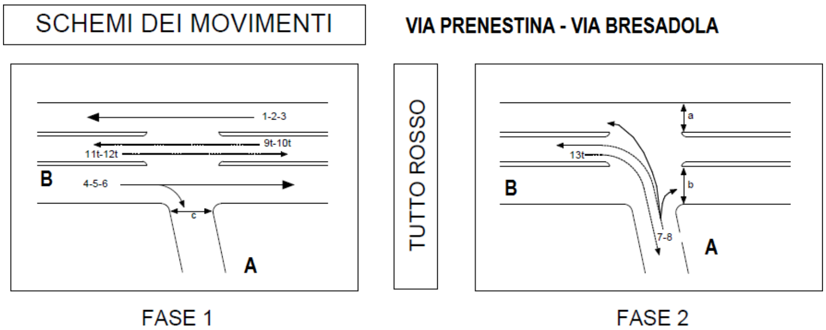


Figure 26 - The Phases of Intersection Via Prenestina & Via Giacomo Bresadola

Table 32 - The Input Parameters (Signalization) Via Prenestina & Via Giacomo Bresadola

	PHASE 1	PHASE 2
green	76	44
yellow	4	4
red	52	84
cycle length	132	

Table 33 - The Input Parameters (Geometric) Via Prenestina & Via Giacomo Bresadola

2nd Junction				
Direction	Street Name	No. of Lane	Lane Width [m]	Grade %
WB	Via Prenestina (W)	2	3.5	0
EB	Via Prenestina (E)	2	3.5	0
NB	Via Giacomo Bresadola	1	3.5	0
SB	-	0	0.0	0

Table 34 - The Volume and Timing Inputs Via Prenestina & Via Giacomo Bresadola

VOLUME AND TIMING INPUT	EB			WB			NB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT
N of lanes	2			2			1		
Lane width (m)	3.5			3.5			3.5		
Volume, V (veh/h)	/	/	310	/	/	/	80	/	/
% heavy vehicles HV			1%				0%		
% vans			7%				23%		
%buses			4%				29%		
PHF			0.9				0.9		
Pretimed(P) or Actuated(A)			P				P		
Start up lost time (s)			3				3		
Clearance lost time (s)			3				3		
Exstension of effective green time			-				-		
Arrival type, AT			3				3		
Approach pedestrian volume (p/h)			20				20		
Approach bycicles volume (bycicles/h)			10				10		
Parking (Y or N)			N				N		
Parking maneuvers, Nm			0				0		
Bus stopping, Nb (buses/h)			7.00				0.00		
Min timeing for pedestrian, Gp (s) (pg 16-5)			10.75				13.66		
PED FLOW/CYCEL			**0.366666667**				**0.366666667**		

Table 35 - Determining Adjusted Saturation Flow Rate Via Prenestina & Via Giacomo Bresadola

VOLUME AND SATURATION FLOWRATE ADJUSTMENT	EB			WB			NB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT
VOLUME ADJUSTMENT									
Volume			310				80		
PHF			0.9				0.9		
adjusted flow rate			344.4444444				88.9		
adjusted flow rate in lane groups			344.4444444				88.9		
Proportion of LT or RT			100%				100%		
SATURATION FLOW RATE AND ADJUSTMENT FACTOR	2100								
base saturation flow, s0									
n of lanes			2				1		
lane width adj factor fw			0.99				0.99		
heavy vehs adjustment factor fHV			1.00				1		
Parking adjustment factor, fp			0.95				0.9		
Bus blockage adjustment facto, fbb			0.986				1		
Area type adjustment factor, fa			1				1		
Lane utilization adjustment factor, fLU			0.5				1		
Left turn adjustment factor, fLT			/				0.95		
Right turn adjustment factor, fRT			0.85				/		
Left turn ped/bike adjustment factor, fLpb			1				1		
Right turn ped/bike adjustment factor, fRpb			0.891821697				/		
Adjusted saturation flow, s (veh/h)			1474.471063				1780		

Table 36 - Capacity Analysis Via Prenestina & Via Giacomo Bresadola

	PHASE 1						PHASE 2		
CAPACITY AND LOS									
Worksheet	EB			WB			NB		
Capacity Analysis	LT	TH	RT	LT	TH	RT	LT	TH	RT
Adjusted flow rate, v [veh/h]			344.4444444				88.9		
Saturation flow rate, s [veh/h]			1474.471063				1780		
Lost time, tL [s]			6				6		
Effective green time, g [s]			74				42		
Green ratio, g/C			0.56				0.32		
Lane group capacity, c = s(g/C), (veh/h)			826.60				566.36		
v/c ratio, X			0.42				0.16		
Flow ratio, v/s			0.23				0.05		
Critical lane group/phase (√)			0.23				0.05		
Sum of flow ratios for critical lane groups, Yc = ∑ (critical lane groups, v/s)	0.284								
Total lost time per cycle, L (s)	12								
Critical flow rate to capacity ratio, Xc = (Yc)(C)/(C – L)	0.312								

Table 37 - Level of Service Via Prenestina & Via Giacomo Bresadola

Lane Group Capacity, Control Delay, and LOS Determination	EB			WB			NB		
Uniform delay, d1			16.63				32.29		
Incremental delay calibration, k			0.50				0.50		
Incremental delay, d2			1.55				0.59		
Progression adjustment factor, PF			1.00				1.00		
Delay, d [s/veh]			18.17				32.89		
LOS by approach			B				C		
Delay by approach, dA [s/veh]			18.17				32.89		
Approach flow rate, vA [veh/h]			344.44				88.89		
Intersection delay, dI [s/veh]	21.1909	C							

Table 38 - Minimum Cycle Length Via Prenestina & Via Giacomo Bresadola

Minimum Cycle	PHASE 1						PHASE 2		
	EB			WB			NB		
Cmin	16.74909	s							
g			3.912677477				0.836408803		
r			12.8364088				15.91267748		
g/C			0.233605428				0.049937578		
c			344.4444444				88.88888889		
x			1				1		
d1			6.42				7.96		
d2			48.49				95.46		
d1+d2			54.91				103.42		
Approach			54.91				103.42		
Approach flow rate			344.44				88.89		
intersection delay	64.8612	E							

Table 39 - Optimum Cycle Length Via Prenestina & Via Giacomo Bresadola

Webster's Optimal Policy	PHASE 1						PHASE 2		
	EB			WB			NB		
C	32.10242	s							
g			16.56				3.54		
r			15.54				28.56		
g/C			0.52				0.11		
c			760.70				196.31		
x			0.45				0.45		
d1			4.91				13.37		
d2			1.94				7.37		
d1+d2			6.85				20.74		
Approach			6.85				20.74		
Approach flow rate			344.44				88.89		
intersection delay	9.700018	A							

Table 40 - The results of the analysis Via Prenestina & Via Giacomo Bresadola

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	132	16.749	32.102	39.7
Intersection Delay	21.19	64.861	9.7	9.25
Level of service	C	E	A	A
g1	76	3.91	16.56	22.5
g2	44	0.836	3.54	5.3
g1/g2	1.73	4.68	4.68	4.25

7 Optimization of the Intersections

7.1 Optimization of the 1st intersection

The intersection is VIA PRENESTINA – VIA DIGNANO D'ISTRIA. The current overall delay of the 1st intersection is calculated about 23.55 [sec] and the level of service of intersection is C.

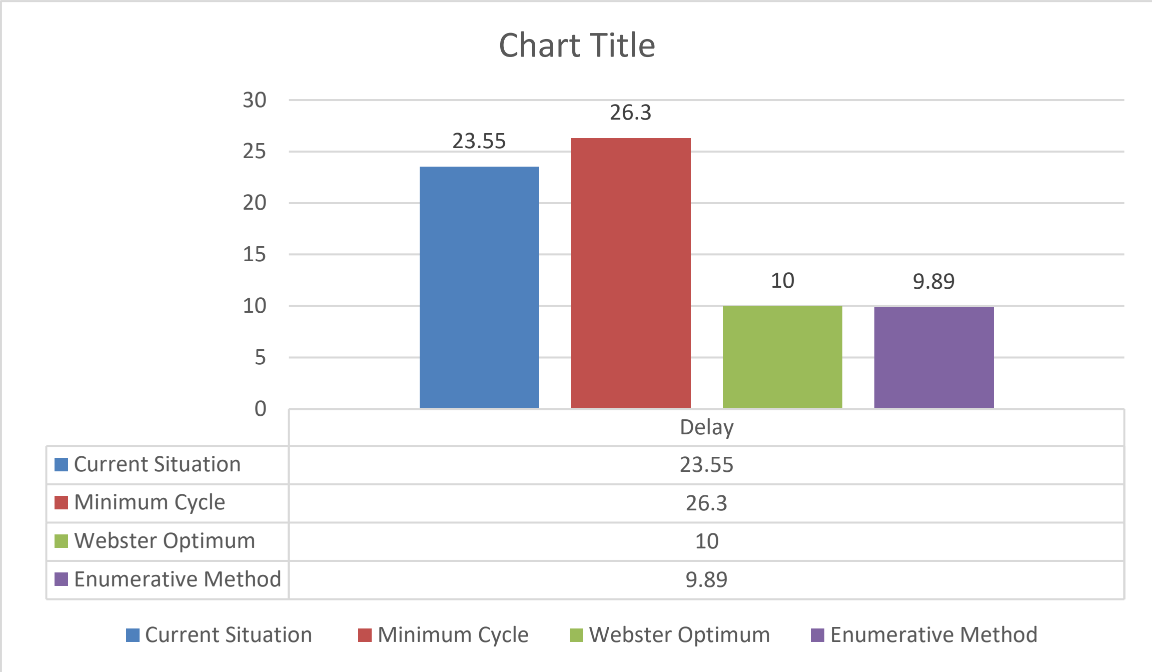
The first step is to consider the capacity constraints to obtain the minimum cycle length to serve all the demand and its respective green duration; this is reported in Table together with the optimum cycle for Webster. The delay for the Minimum Cycle is 26.30 (sec/veh), and for Webster Optimum Cycle is 10 (sec/veh), and it's necessary to note that they correspond to different objective function.

The optimization is done by iteration method. The optimum cycle for this intersection is 44.4 [s]. Moreover, the green ratio is 3.98. By considering the optimum cycle of the intersection, the minimum delay is 9.89 [s]. It represents that level of service of our intersection is A.

Table 41 - The results of the analysis Via Prenestina & Via Dignano D'istria

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	132	23.4	44.8	44.4
Intersection Delay	23.55	26.30	10	9.89
Level of service	C	C	B	A
g1	76	9	26	25.9
g2	44	2.3	6.8	6.5
g1/g2	1,73	3.91	3.82	3.98

Figure 27 - Delays Comparison Table



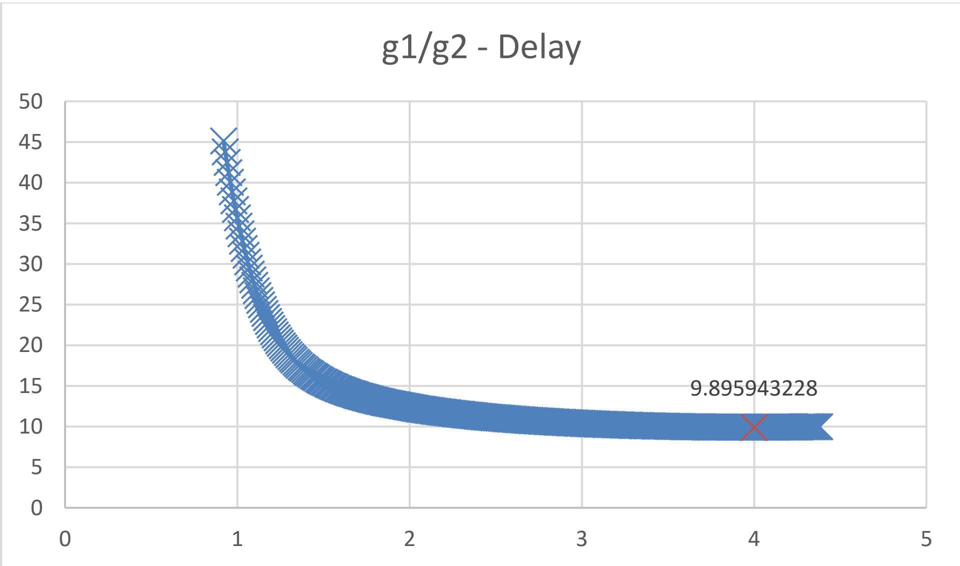


Figure 28 - Variation of delay with variation of green ratio

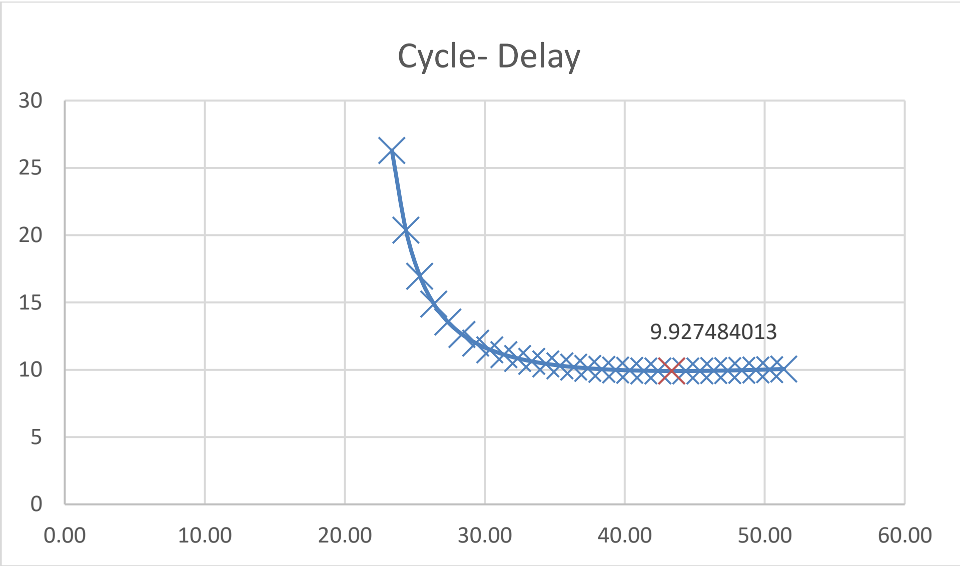


Figure 29 - Variation of delay with respect to C keeping G1/G2 unvaried

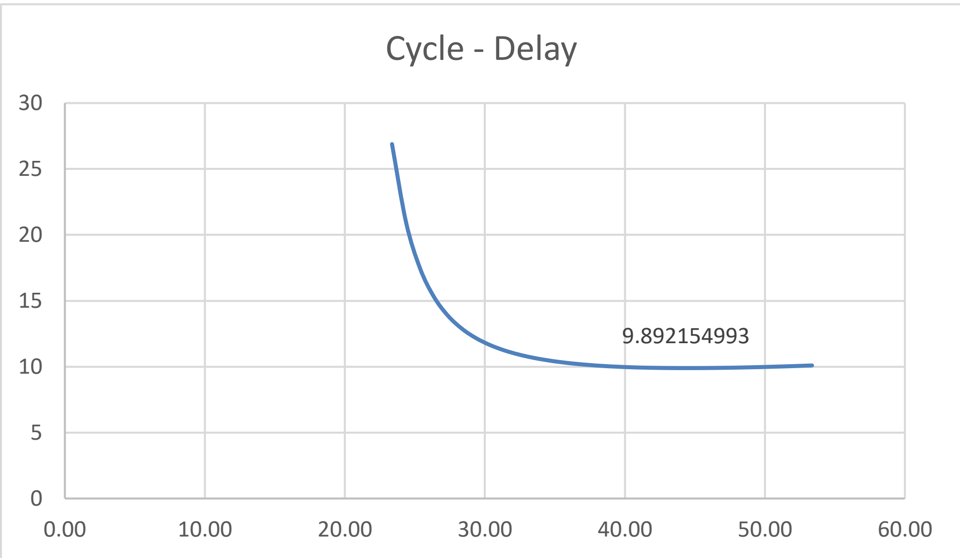


Figure 30 - Variation of delay with variation of green ratio and Cycle time

7.2 Optimization of 2nd Intersection

The intersection is VIA PRENESTINA – VIA OLEVANO ROMANO. The current overall delay of the 2nd intersection is calculated about 18.74 [sec] and the level of service of intersection is B.

The first step is to consider the capacity constraints to obtain the minimum cycle length to serve all the demand and its respective green duration; this is reported in Table together with the optimum cycle for Webster. The delay for the Minimum Cycle is 24.70 (sec/veh), and for Webster Optimum Cycle is 7 (sec/veh), and it's necessary to note that they correspond to different objective function.

The optimization is done in iteration method. The optimum cycle for this intersection is 52.1 [s]. Moreover, the green ratio is 6.02. By considering the optimum cycle of the intersection, the minimum delay is 6.93 [s]. It represents that level of service of our intersection decreases from B to A.

Table 42 - The results of the analysis Via Prenestina & Via Olevano Romano

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	128	20.1	38.5	52.1
Intersection Delay	18.74	24.70	7	6.93
Level of service	B	C	A	A
g1	75	6.7	22	34.3
g2	45	1.4	4.5	5.7
g1/g2	1.67	4.79	4.89	6.02

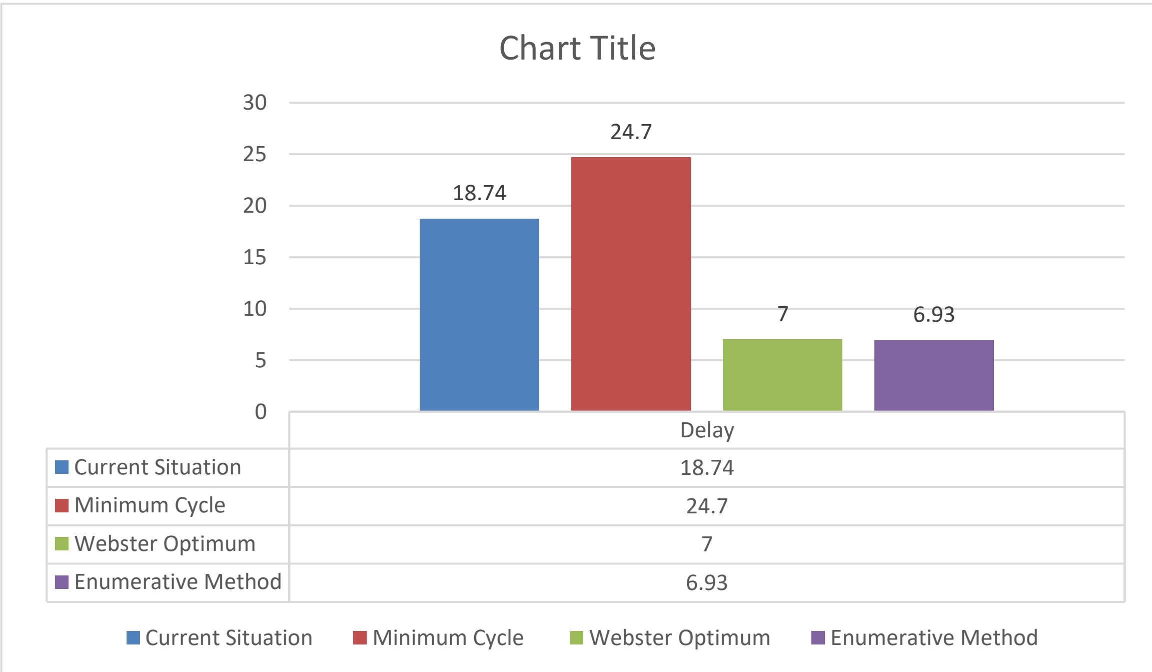


Figure 31 - Delays Comparisons Table

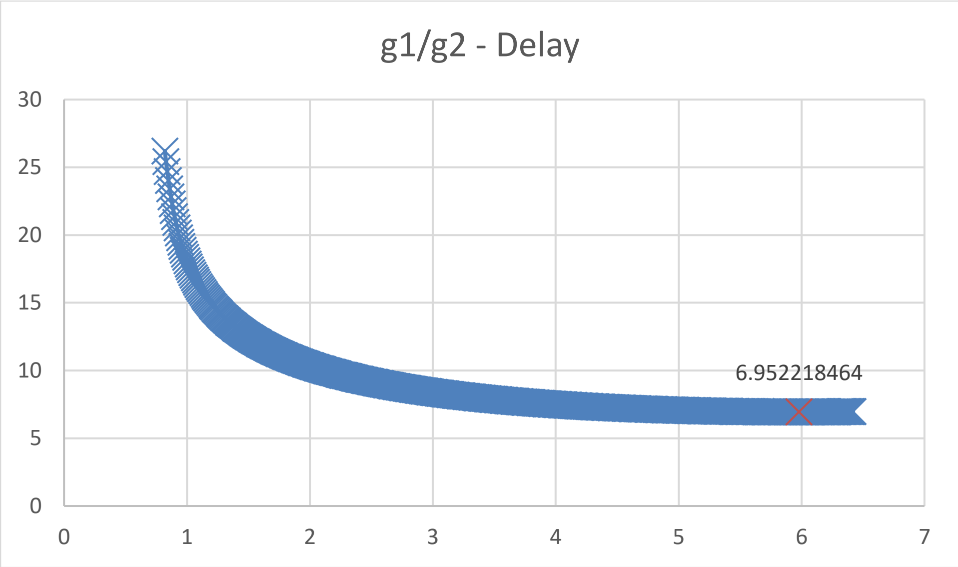


Figure 32 - Variation of delay with variation of Green ratio

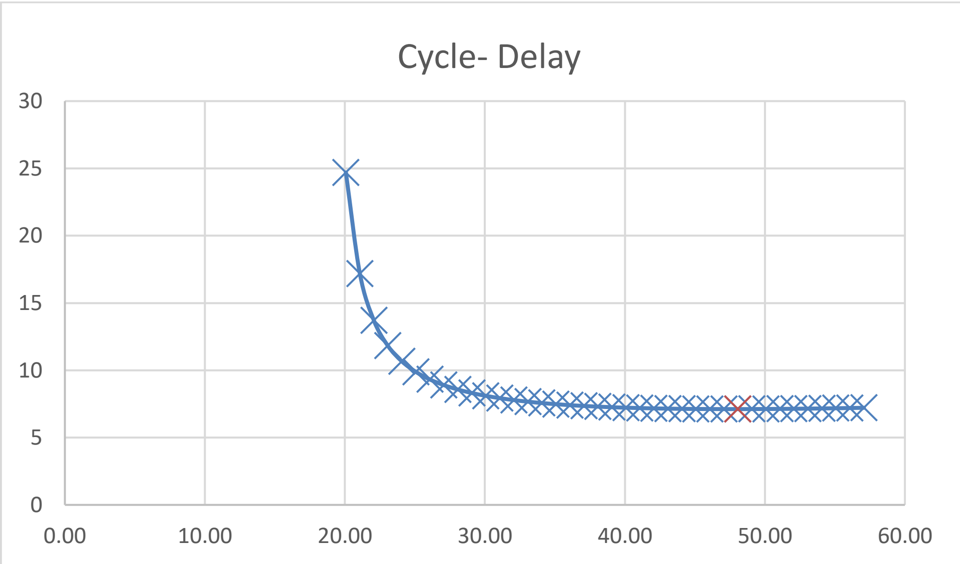


Figure 33 - Variation of delay with respect to C keeping G1/G2 unvaried

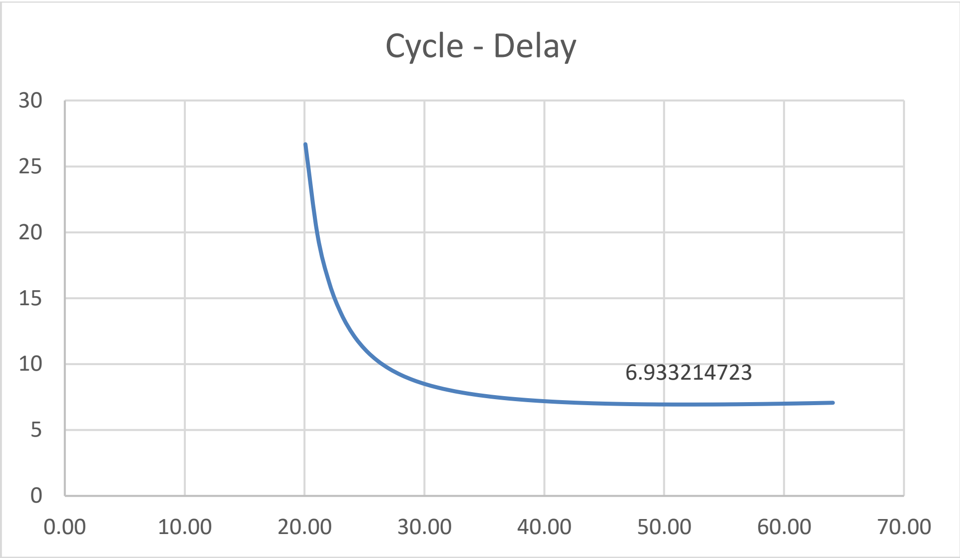


Figure 34 - Variation of delay with respect to C keeping G1/G2 unvaried

7.3 Optimization of 3rd Intersection

The intersection is Via prenestina and Via Tor de'Schiavi. The intersection delay is calculated as 55.57 seconds and the level of service of intersection is defined as 'E' for current situation values.

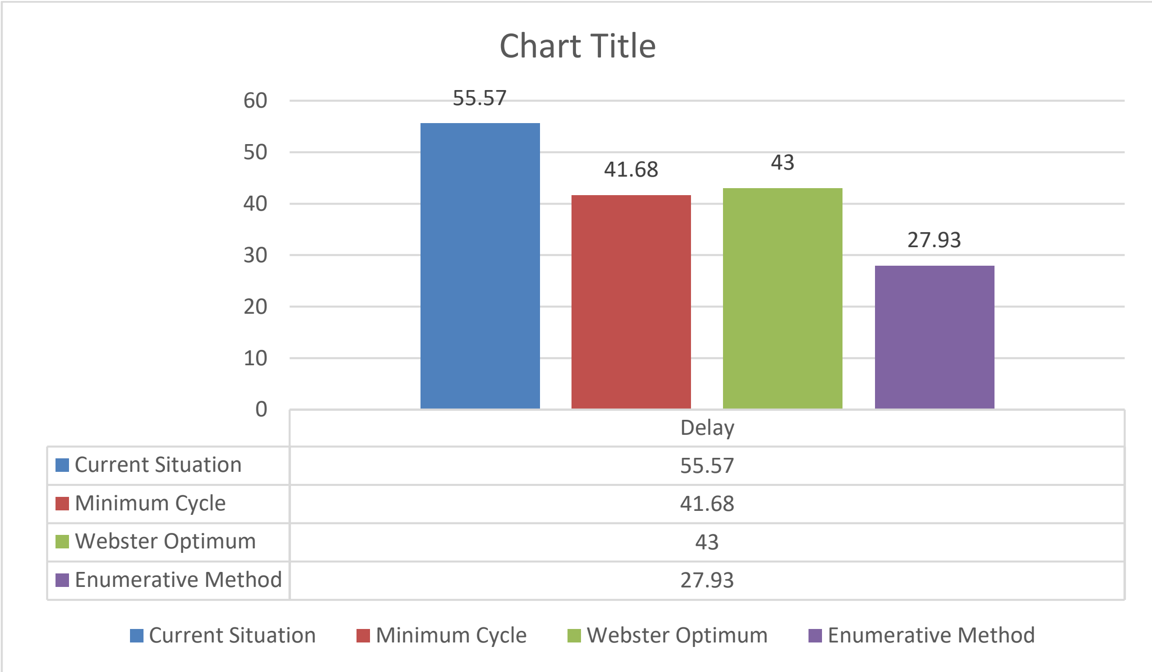
As a first step, based on the demand and its respective green duration, the minimum cycle length required is obtained. The table shows the information about the cycle length for Webster andminimum cycle length. For the Minimum Cycle, the delay is 41.68 seconds, and the level of service is 'D'. For the Webster Optimum Cycle, the delay is 43 seconds, and the level of service is 'D'. it's necessary to note that they correspond to different objective function.

For Enumerative method, the value of intersection is obtained by performing the iteration method. the delay is 27.93 seconds, and the level of service is 'C'. In addition to that, the green ratiois 1.130. When the webster optimum cycle is compared with enumerative method, it gives that the level of service of the intersection up- grated from E to C.

Table 43 - The results of the analysis Via Prenestina & Via Tor de'Schiavi

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	132	55.4	106.1	105.4
Intersection Delay	55.57	41.68	43	27.93
Level of service	E	D	D	C
g1	48	12.9	27.9	49.5
g2	42	16.5	35.8	43.8
g3	36	14	30.4	30.4
g1/g2	1.14	0.782	0.779	1.130

Figure 35 - Delays Comparisons Table



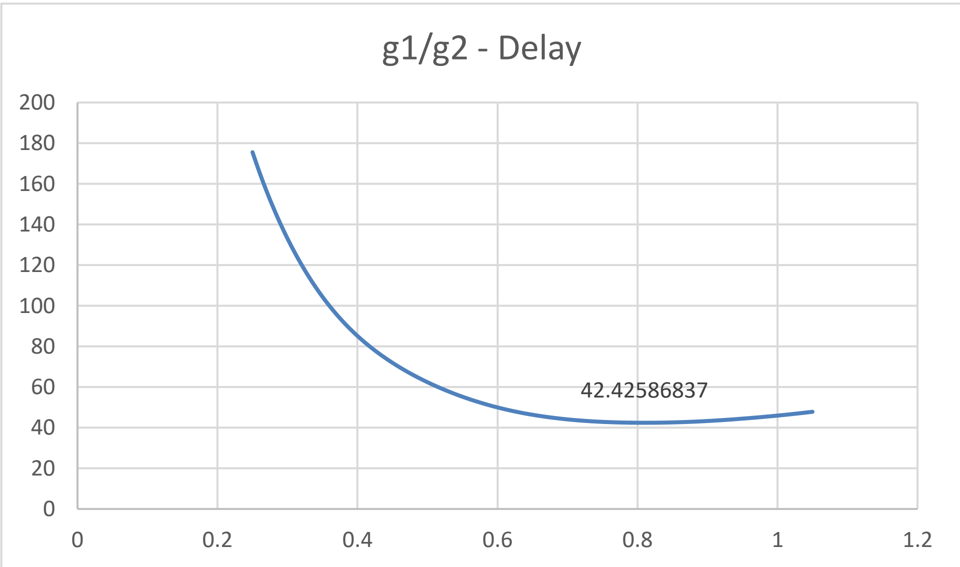


Figure 36 - Variation of delay with variation of green ratio

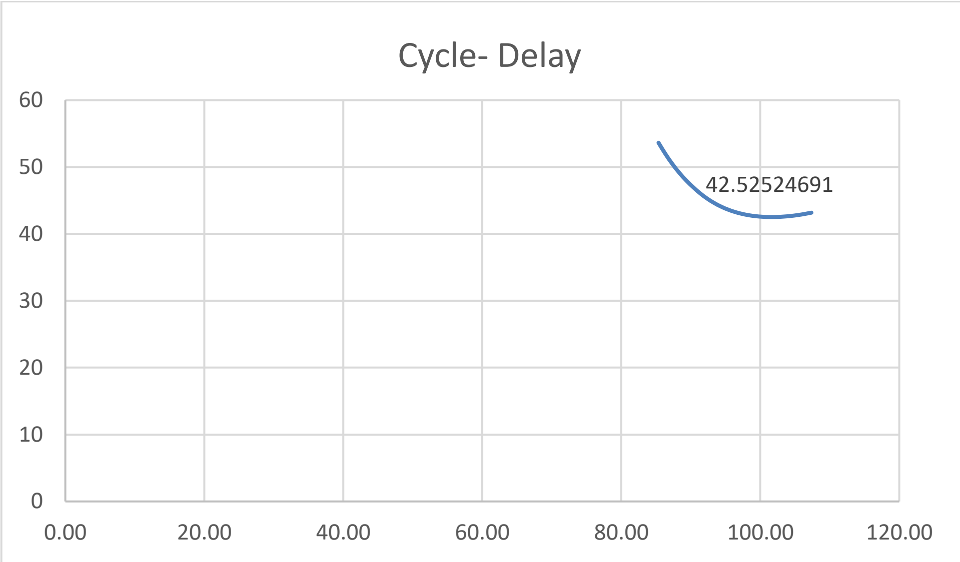
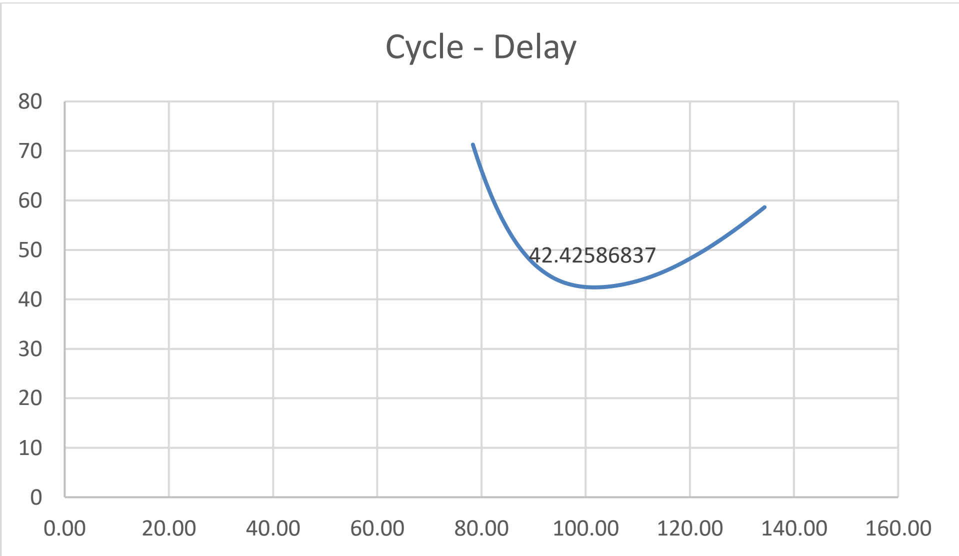


Figure 37 - Variation of delay with respect to C keeping G1/G2 unvaried



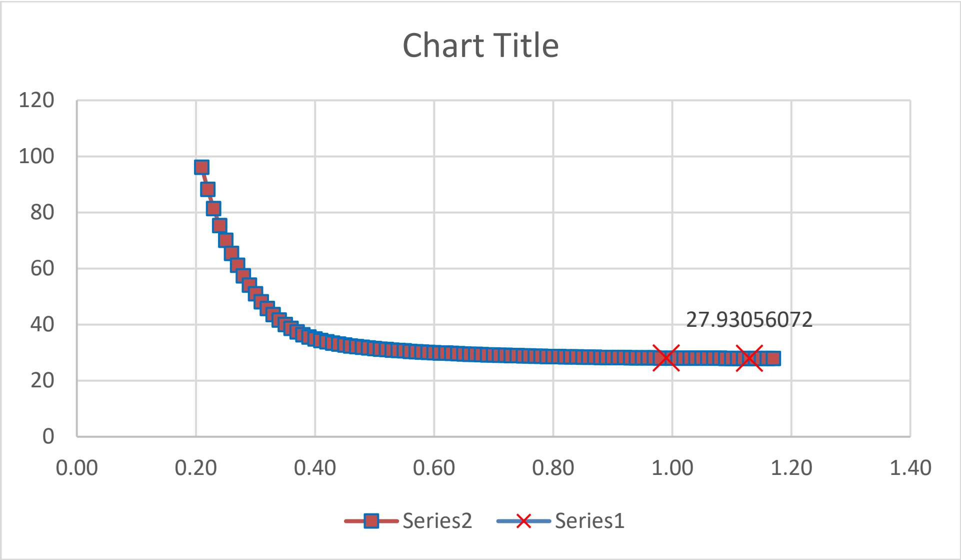


Figure 38 - Variation of delay with variation of green ratio and Cycle time

7.4 Optimization of 4th Intersection

The intersection is Via prenestina and Via Giacomo Bresadola. The intersection delay is calculated as 21,19 seconds and the level of service of intersection is defined as ‘C’ for current situation values.

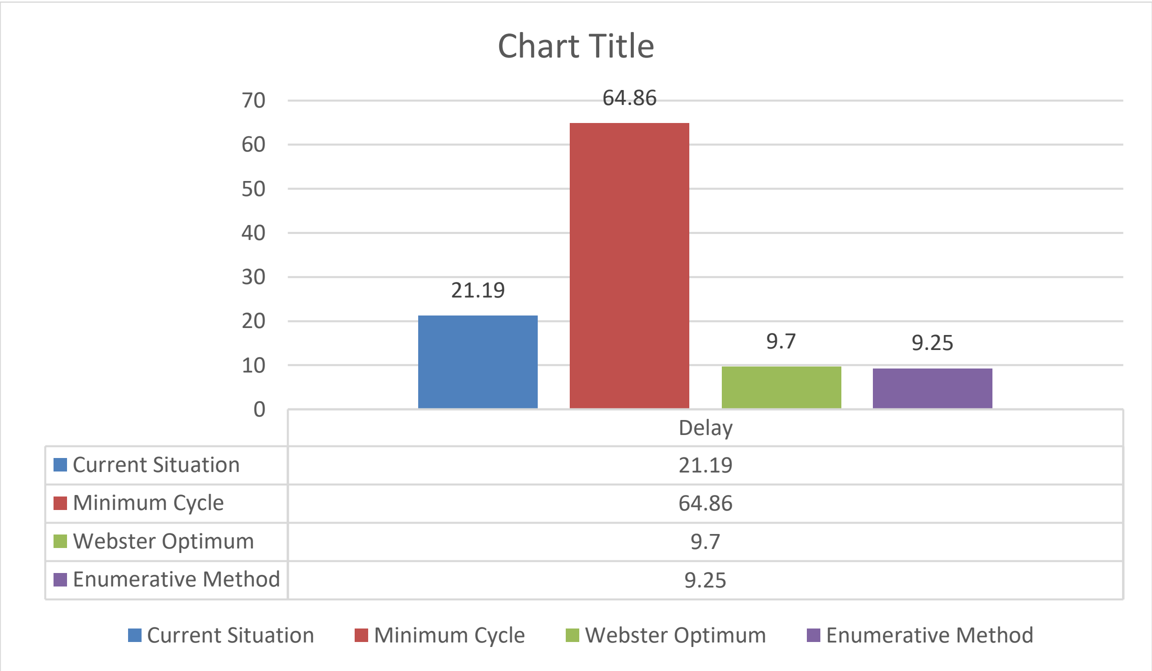
As a first step, Based on the demand and its respective green duration, the minimum cycle length required is obtained. The table shows the information about the cycle length for Webster andminimum cycle length. For the Minimum Cycle, the delay is 64.861 seconds and the level of service is ‘E’. . For the Webster Optimum Cycle, the delay is 9,7 seconds and the level of service is ‘A’. it’s necessary to note that they correspond to different objective function.

For Enumerative method, the delay is 9.25 seconds and the level of service is ‘A’. In addition to that, the green ratio is 4.25. When the webster optimum cycle is compared with enumerative method, it gives that the level of service of the intersection is changed from C to A.

Table 44 - The results of the analysis Via Prenestina & Via Giacomo Bresadola

	Current Situation	Minimum Cycle Method	Webster Optimum Cycle	Enumerative Method
Cycle	132	16.749	32.102	39.7
Intersection Delay	21.19	64.861	9.7	9.25
Level of service	C	E	A	A
g1	76	3.91	16.56	22.5
g2	44	0.836	3.54	5.3
g1/g2	1.73	4.68	4.68	4.25

Figure 39 – Delays Comparion Table



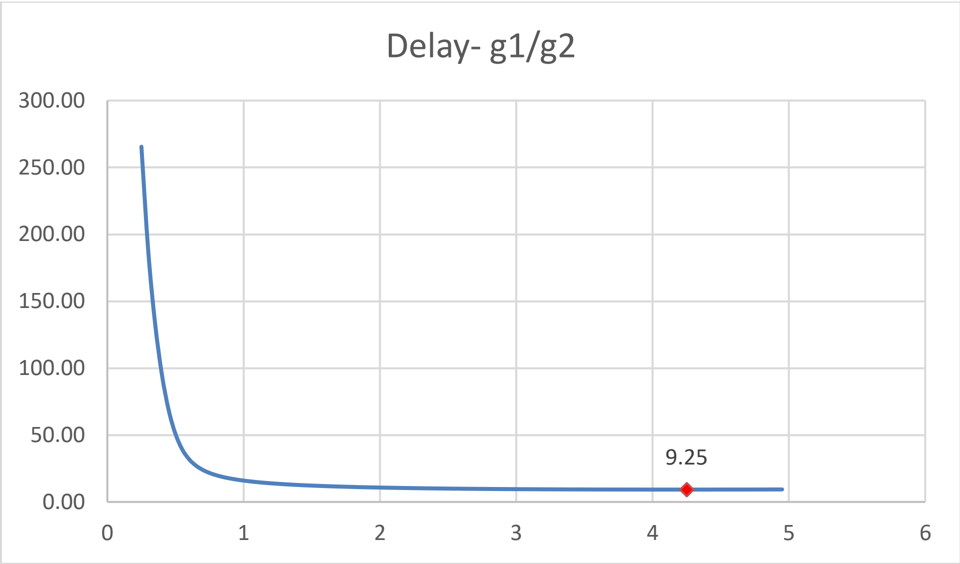


Figure 40 - Variation of delay with variation of Green ratio

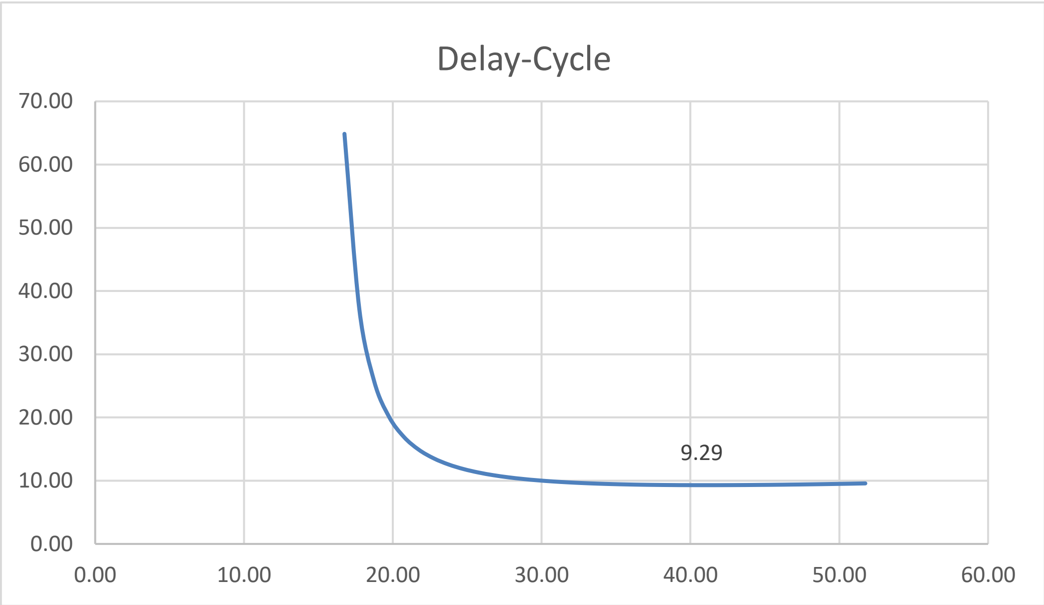


Figure 41 - Variation of delay with respect to C keeping G1/G2 unvaried

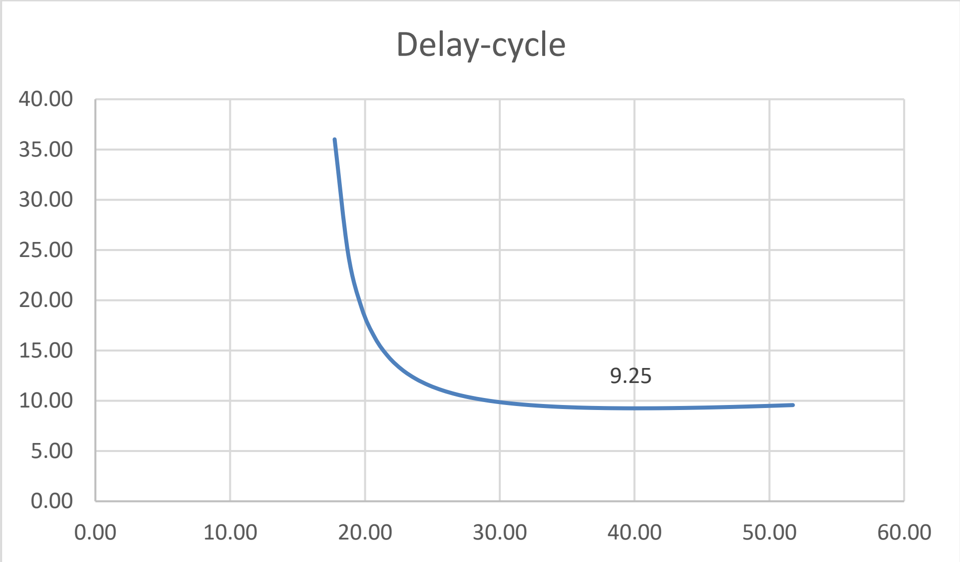


Figure 42 - Variation of delay with variation of Green ratio and Cycle time

8. Synchronization

After the analysis of single junctions, the artery can be studied. The goal is to minimize the delay of the entire artery. Each intersection is affected by the departure and the flow from the close upstream intersection. The reason of a signal's coordination (or synchronization), which regulates thestarting of green at intersections, is to allow vehicles arriving at the downstream intersection during the green phase, in order to reduce the delay.

A synchronization problem takes as given:

- ✓ the distance between consecutive intersections li
- ✓ the signal setting at each intersection gi, Ci (in order to have a periodical repetition of coordination, the cycle length has to be chosen equal for all the intersections)
- ✓ the synchronization speed in the two directions, assuming that vehicles travel along the artery at that speed, v1, v2
- ✓

and gives as output a vector of the signals' offsets θij, which minimizes the delay of vehicles travelling along the artery.

In the ideal synchronization:

- ✓ all the intersections are equally spaced lij=A ∀i≠j i=1, 2…. n
- ✓ there is no entering or exiting flow along the artery, it means that the green and the cycle are the same for each intersection.
- ✓ traffic flow is lower than a given value

Under the condition of constant traffic flow, the green will be the same for all the intersections and so also the cycle will be unique for the artery. In this way all the vehicles will move at the same constant speed, forming a compact platoon and bands of vehicles' trajectory along the artery can be individuated. A vector of offsets, which make the delay at intersections nil, can be found: at each node the green will start as soon as the first vehicle of the platoon arrives and will end as soon as the last vehicles passes, such that it is possible to have a platoon moving within bands without being stopped. This can be expressed by imposing the offset (ϑij) equals the running time between two consecutive nodes:

$$t_{ij} + t_{ji} = \frac{A}{v_1} + \frac{A}{v_2} = mC$$

where m is an integer, If v1=v2 =v it can be obtained:

$$A = \frac{mvC}{2}$$

So that the solution of the ideal synchronization is:

$$\vartheta_{ij} = 0 \text{ if } \frac{x_j - x_i}{2A} = m \qquad \text{for } m=0,1,2, \dots$$
$$\vartheta_{ij} = \frac{C}{2} \quad \text{if } \frac{x_j - x_i}{2A} = \frac{2m+1}{2} \qquad \text{for } m=0,1, 2\dots$$

The ideal synchronization is usually unfeasible in real cases, because:

- ✓ junctions are not equally spaced
- ✓ flow is not uniform along the artery
- ✓ green splits are not equal

These cases can be solved by two different approaches:

- ✓ Minimum delay problem
- ✓ Maximal green bandwidth problem

The problem of the minimum delay is non convex, so it doesn't have a unique solution. It can be solved by simulation programs, which reproduce the traffic conditions.

The minimum delay solution, but correlated to it because by increasing the green bandwidth (which is defined as the set of possible trajectories at constant speed that are uninterrupted along the artery) the number of vehicles within it, so not delayed, increases.

In the formulation of the problem the bandwidths in oppose directions are different and the offset can be a value between 0 and 1. By imposing the same bandwidth in the opposite directions (b=b') the symmetric problem (MB1) is obtained, which is:

$$max\ f = (b+b')$$
$$b = b' > 0$$
$$\theta_{ij} = 0 \text{ or } ½$$
$$v_{min} \leq v \leq v_{max}$$
$$max\ \{C_{min,\ i}\} \leq C \leq C_{max}$$
$$C_{min,i} = \frac{L_i}{(1 - max_h\{y_{i,h}\} - max_k\{y_{i,k}\})}$$
$$max_h\{y_{i,h}\} \leq g_i \leq 1 - \frac{L_i}{C} - max_k\{y_{i,k}\}$$

Where:
Li= lost time of node i
yi, h = saturation degree of approach h
yi, k = saturation degree of approach k
b=b' = bandwidth inbound and outbound
gi = green at artery approach i

The problem can be solved using an algorithm based on Equivalent system properties (Papola, Fusco, 1998). For any set of signals an ideal system exists (in which the distance between consecutive nodes is A and the bandwidth equals the duration of green) and has the same solution.
First of all, cycle length, green splits and synchronization speed have to be fixed. The common cycle length can be the minimum or the optimum for the most critical intersection. Green splits can be chosen follow the criterion of the sequestration or optimizing the intersections' delay.
The first pair of intersections is the starting point: depending on the distance between them (lij), the bandwidth (bi, j) and the offset (ϑi, j) can be evaluated.

If $l_{ij} < \frac{A}{2} or\ l_{ij} > \frac{3A}{2}$

$$\vartheta_{ij} = 0$$
$$b_{ij} = b'_{ij} = \frac{1}{2}(g_i + g_j - \frac{l_{ij}}{A})$$

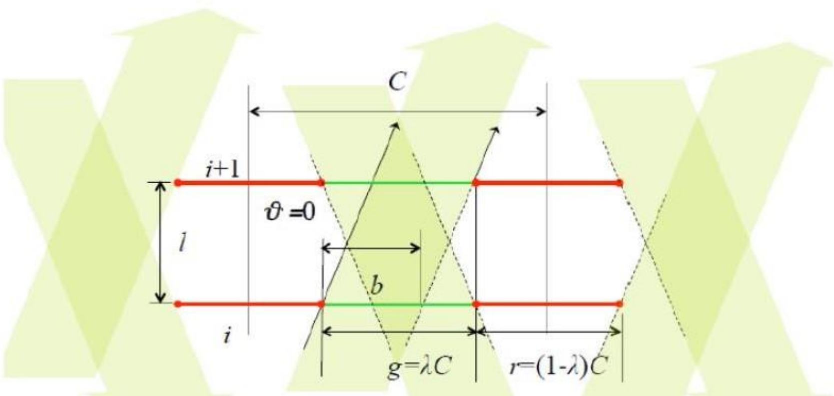


Figure 43 - Junctions in phase

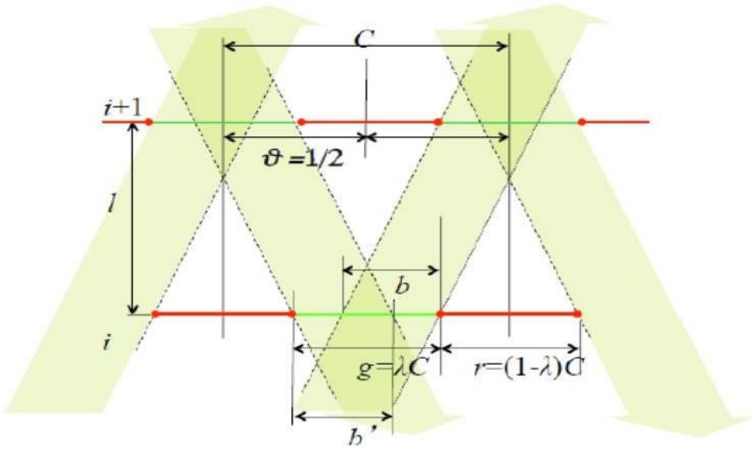


Figure 44 - Junctions in phase opposition

If $\frac{A}{2} < l_{ij} < \frac{3A}{2}$

$$\vartheta_{ij} = \frac{1}{2}$$
$$b_{ij} = b'_{ij} = \frac{1}{2}(g_i + g_j - 1 + \frac{l_{ij}}{A})$$

Then it is necessary to find the position of the ideal node, which is equivalent to the first pair of signals. If the distance between the two nodes is less than ½, the ideal node is between them, otherwise there are two ideal nodes, one under the first node and the other up the second node.

$$x_0 = x_i + (g_i - b_{ij})A \qquad if \ \frac{x_j - x_i}{A} \leq 0.5$$

$$x_0 = x_i - (g_i - b_{ij})A \qquad if \ \frac{x_j - x_i}{A} > 0.5$$

For successive nodes, which can be distant more than A, the operator mantissa is used, in order to have a distance in the range 0,1. The successive node r is synchronized with the equivalent system corresponding to the previous nodes i, j, so that:

$$b_{ij,r} = b'_{ij,r} = \frac{1}{2}\left(g_r + b_{ij} - man\left[\frac{x_r - x_0(i,j)}{A}\right]\right) \quad if \ 0 \leq man[\frac{x_r - x_0(i,j)}{A}] < 0.5$$

$$b_{ij,r} = b'_{ij,r} = \frac{1}{2}\left(g_r + b_{ij} - 1 + man\left[\frac{x_r - x_0(i,j)}{A}\right]\right) \quad if \ 0.5 \leq man[\frac{x_r - x_0(i,j)}{A}] < 1$$

The new ideal system equivalent to the already coordinated signals is individuated. The iterations continue until all the nodes are coordinated. Each node, which reduces the bandwidth, shift the ideal grid, so it’s necessary to evaluate the offsets after that all the nodes have been coordinated, such that:

$$\vartheta = 0 \quad if\ 0 \leq man\left[\frac{x(i) - x_0}{2A}\right] < 0.25\ \cup\ 0.75 \leq man\left[\frac{x(i) - x_0}{2A}\right] < 1$$

$$\vartheta = 0.5 \quad if\ 0.25 \leq man\left[\frac{x(i) - x_0}{2A}\right] < 0.75$$

8.1 Synchronization of Artery

The next step is the synchronization of the artery. A unique cycle is necessary for all the intersections. The Maximum cycle length is chosen among all cycle length of the Enumerative Method optimum cycle lengths, for this reason the 132 [sec] which is the maximum cycle length is chosen and then we choose the green ratio which is connecting to the EB and WB for each intersection. The synchronization speed has been assumed equals to 12 m/s (43.2 km/h), which feasible for urban area.

Table 71 - Synchronization data

Intersection number	1	2	3	4
Location of Intersection	0	330	674	812
Green Time	76	75	48	76
Cycle time	132	128	132	132
g/C	0.58	0.59	0.36	0.58
Fraction of A	0	0.3636	0.183889	0.20202
Offset	0	0,5	0,5	0
Bandwidth	0.183889			

As it is calculated and shown on previous table, we know that the bandwidth for all 4 intersections of our area is 0.183889. In conclusion cars passing in 24.17 [s] of effective green of first intersection can pass the last intersection without stopping.

9. Conclusion

In conclusion, four intersections have been examined and delay was computed with respect to current situation (HCM), minimum cycle length, Webster optimum and enumerative method.

As a result of the investigations made at the first intersection namely Via Prenestina – Via Dignano D'istria, the current situation shows a Level of Service of C and delay is 23.55 s approximately. It represents that level of service of this intersection is A (with delay = 9.90 sec.) in enumerative method.

Second intersection is Via Prenestina – Via Olevano Romano and in this intersection the improvement is clearly seen. While delay reduced from 18.74 seconds to 6.93 second, level of service improved from B to A.

As 3rd Intersection is Via prenestina and Via Tor de'Schiavi. In the current situation, intersection delay is calculated as 55.57 seconds and the level of service of intersection is calculated as E. The delay is 27.93 seconds and level of service is C according to the enumerative method.

Finally last intersection is Via prenestina and Via Giacomo Bresadola and in this intersection, the improvement is clearly seen. While delay reduced from 21.191 seconds to 9.25 second, level of service improved from C to A.

In all the junctions studied we can see an improvement of the delays respect to the current situations. For the synchronization of the artery, critical intersection must be taken into account in terms of cycle length. With respect to current calculations, Via prenestina and Via Dignano D'istria was determined as a critical intersection with 132 sec. cycle length. After synchronization calculations according to maximal bandwidth, our critical bandwidth was found as 0.183889 meaning that car passing in 24.17 seconds of effective green of first intersection can pass the last intersection without stopping.

10. References

1- Highway Capacity Manual (HCM) - Chapter 16. (2000). Washington, United States.
2- Teaching material of "Traffic Engineering and Intelligence Transportation Systems" course, Gaetano Fusco, University Sapienza of Rome