SAPIENZA
UNIVERSITÀ DI ROMA

Signal Setting Design for Via Prenestina

Traffic Engineering and ITS

Professor Gaetano Fusco

Contributors:

Mohammadreza Keramati 1937929

Jovan Nazari 1883854

1. Introduction

The design of traffic signal timing parameters, such as green time ratio, can significantly affect the operational efficiency and safety of intersections and the total amount of traffic emissions in the network. It is, therefore, crucial for the authority to design the traffic signal timing parameters carefully so as to create a safe, efficient, and sustainable urban transportation system.
The purpose of this assignment is to study and analyze the actual traffic situation of 4 Signalized junctions in the Metropolitan City of Rome, based on the Highway Capacity Manual (HCM) methodology, to reduce intersection delay and determine Level of Service, by optimization of signal setting in each approach. The final step is synchronization that the results maximizing the bandwidth, is related to the minimization of the delay along the artery.

2. Study Area

The road artery chosen for this project is a part of Via Prenestina, located in the Eastern area of Rome .V, a predominantly residential area, this road connects the center of Rome with the Ring Road Circonval lazione Orientale. The study area is composed by 4 junctions from Largo Telese to Via Olevano Romano, with a total length of 700m.

Figure 1: Study Area Overview

Junction number	EB	WB	NB	SB
1	Via Prenestina	Via Prenestina	Largo Telese	Viale Della Stazione Prenestina
2	Via Prenestina	Via Prenestina	Largo Irpinia	Viale Ronchi
3	Via Prenestina	Via Prenestina	Largo Irpinia	Via Dignano d'stria
4	Via Prenestina	Via Prenestina	Via Olevano Romano	----

Table 1: List of junctions

Junction Number	Progressive Distance [m]
1	0
2	230
3	340
4	716

Table 2: The progressive distances of the junctions

Junction Number	Distances [m]
1-2	230
2-3	110
3-4	376

Table 3: The distances between junctions

3. Methodology

The procedure that has been used is based on the Highway Capacity Manual, particularly the topics discussed in Chapter 16, where we can find the guidelines for the computation of the Capacity and the Level of Service (LOS) for signalized intersections. In the analysis, it's considered a variety of conditions as the amount of traffic movements and their distribution, geometric characteristics and other details. In Figure 2 we can see the inputs of the procedure and which are the outputs:

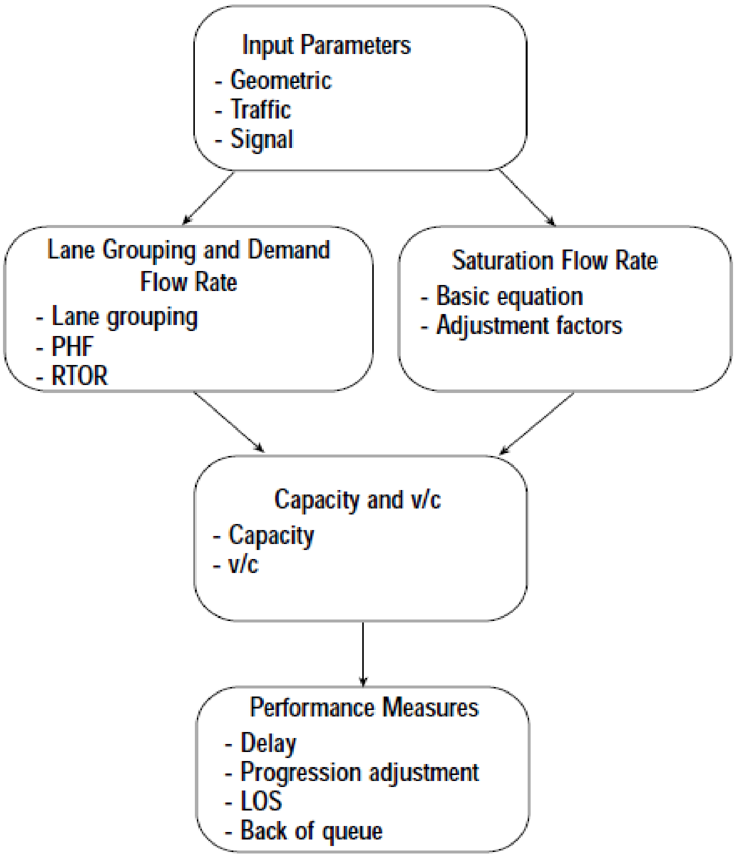


Figure2: HCM Methodology for Signalized Intersection

3.1. Input Parameters

The data needed fall into three main categories: Geometric, traffic and signalization, in figure 3 we have them detailed.

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_o (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 needs for each analysis lane group

An important characteristic that must be quantified to complete an operational analysis of a signalized intersection is the quality of the progression, the parameter that describes this characteristic is the arrival type (AT), for each lane group. Here, it’s been assumed an arrival type 3, considering random arrivals in which the main platoon contains less than 40 percent of the lane group volume.

3.2. Lane Grouping

The methodology for signalized intersections is disaggregate; it is designed to consider individual intersection approaches and individual lane groups within approaches. Segmenting the intersection into lane groups is a relatively simple process that considers both the geometry of the intersection and the distribution of traffic movements. In general, the smallest number of lane groups is used that adequately describes the operation of the intersection. The following guidelines may be applied.

· An exclusive left-turn lane or lanes should normally be designated as a separate lane group unless there is also a shared left-through lane present, in which case the proper lane grouping will depend on the distribution of traffic volume between the movements. The same is true of an exclusive right-turn lane.

· On approaches with exclusive left-turn or right-turn lanes, or both, all other lanes on the approach would generally be included in a single lane group.

· When an approach with more than one lane includes a lane that may be used by both left-turning vehicles and through vehicles, it is necessary to determine whether equilibrium conditions exist or whether there are so many left turns that the lane essentially acts as an exclusive left-turn lane, which is referred to as a de facto left-turn lane. De facto left-turn lanes cannot be identified effectively until the proportion of left turns in the shared lane has been computed. If the computed proportion of left turns in the shared lane equals 1.0 (i.e., 100 percent), the shared lane must be considered a de facto left-turn lane. When two or more lanes are included in a lane group for analysis purposes, all subsequent computations treat these lanes as a single entity. Exhibit 16-5 shows some common lane groups used for analysis.

Number of Lanes	Movements by Lanes	Number of Possible Lane Groups
1	LT + TH + RT	① (Single-lane approach)
2	EXC LT TH + RT	②
2	LT + TH TH + RT	① OR ②
3	EXC LT TH TH + RT	② OR ③

Figure 4: Typical lane groups for Analysis

3.3. Determining Flow Rate

Movements volumes are adjusted to flow rates for each desired period of analysis, if necessary, and lane groups for analysis are established. There is mainly three alternative ways in which an analyst might proceed for a given study.

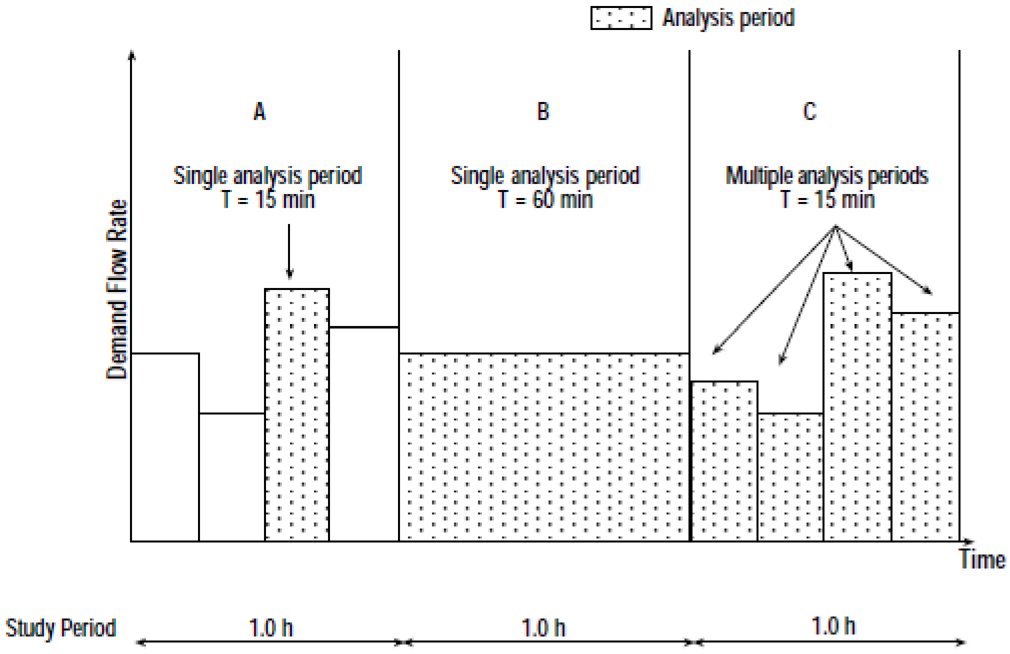


Figure 5: Three alternative study approaches

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.

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

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

In this project, the 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. Determining Saturation Flow Rate

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.

3.4.1. Base Saturation Flow Rate

Computations begin with the selection of a base saturation flow rate, usually 1,900 passenger cars per hour per lane (veh/h/ln). This value is adjusted for a variety of conditions, which considering 2100 (veh/h/ln) in this project.

3.4.2. Adjustment Factor Calculation

The adjustment factors are given in Exhibit 16-7.

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 6: Adjustment factors for saturation flow rate

3.5. Determining Capacity

At a signalized intersection, the capacity is based on the concept of saturation flow and saturation flow rate. 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 (vi) and the saturation flow rate (si). 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), and

g_i/C = effective green ratio for lane group i.

3.6. Determining 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, Xi 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 group i (veh/h),

g_i = effective green time for lane group i (s), and

C = cycle length (s).

3.7. Determining Delay

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

$$d = d_1(PF) + d_2 + d_3$$

where

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

d_1 = uniform control delay assuming uniform arrivals (s/veh);

PF = uniform delay progression adjustment factor, which accounts for effects of signal progression;

d_2 = incremental delay to account for effect of random arrivals and oversaturation queues, adjusted for duration of analysis period and type of signal control; this delay component assumes that there is no initial queue for lane group at start of analysis period (s/veh); and

d_3 = initial queue delay, which accounts for delay to all vehicles in analysis period due to initial queue at start of analysis period (s/veh) (detailed in Appendix F of this chapter).

Progression Adjustment Factor (PF)

$$PF = \frac{(1-P)f_{PA}}{1-\left(\frac{g}{C}\right)}$$

where

PF = progression adjustment factor,

P = proportion of vehicles arriving on green,

g/C = proportion of green time available, and

f_{PA} = supplemental adjustment factor for platoon arriving during green.

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 7: Progression adjustment factor for uniform delay calculation

Delay for uniform arrivals d1

Equation below gives an estimate of delay assuming uniform arrivals, stable flow, and no initial queue. It is based on the first term of Webster's delay formulation and is widely accepted as an accurate depiction of delay for the idealized case of uniform arrivals. Values of x beyond 1.0 are not used in the computation of d1.

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

where

d_1 = uniform control delay assuming uniform arrivals (s/veh);

C = cycle length (s); cycle length used in pretimed signal control, or average cycle length for actuated control (see Appendix B for signal timing estimation of actuated control parameters);

g = effective green time for lane group (s); green time used in pretimed signal control, or average lane group effective green time for actuated control (see Appendix B for signal timing estimation of actuated control parameters); and

X = v/c ratio or degree of saturation for lane group.

Incremental Delay for random and oversaturation queues d2

Equation below is used to estimate the incremental delay due to non-uniform arrivals and temporary cycle failures (random delay) as well as delay caused by sustained periods of oversaturation (oversaturation delay). It is sensitive to the degree of saturation of the lane group (X), the duration of the analysis period (T), the capacity of the lane group (c), and the type of signal control, as reflected by the control parameter (k). The equation assumes that there is no unmet demand that causes initial queues at the start of the analysis period (T).

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

where

d_2 = incremental delay to account for effect of random and oversaturation queues, adjusted for duration of analysis period and type of signal control (s/veh); this delay component assumes that there is no initial queue for lane group at start of analysis period;

T = duration of analysis period (h);

k = incremental delay factor that is dependent on controller settings;

I = upstream filtering/metering adjustment factor;

c = lane group capacity (veh/h); and

X = lane group v/c ratio or degree of saturation.

Incremental Delay for random and oversaturation queues $d2$

This is the standard INPUT WORKSHEET as per HCM that is filled with data that have been provided us through the flow data sheets and geometry sheet of intersection, the flows are meant as equivalent flows, computed using the following coefficients:

Vehicle Classification	Cars	Heavy Vehicles
Coefficients	1	2

3.8. Determining LOS

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

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 for signalized intersections

3.9. Worksheet of Case Study

Operational analysis is divided into five modules: input, volume adjustment, saturation flow rate, capacity analysis, and LOS. The computations for each of these modules are conducted or summarized on the appropriate worksheet. In addition to the module-related worksheets, supplementary worksheets are provided to handle computations that are more complex.

3.9.1. Junction 1. Via Prenestina - Largo Telese -06009

The first analyzed intersection has 4 approaches, Via Prenestina – Via Largo Telese – Via della Stazione Prenstina. The signal setting for this junction is composed by 2 phases.

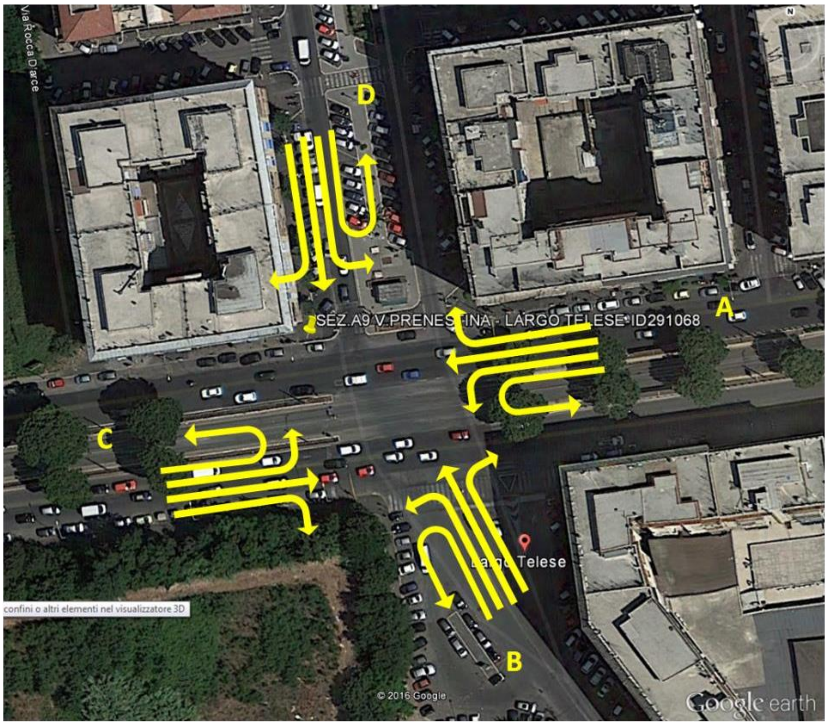


Figure 9: Junction 1

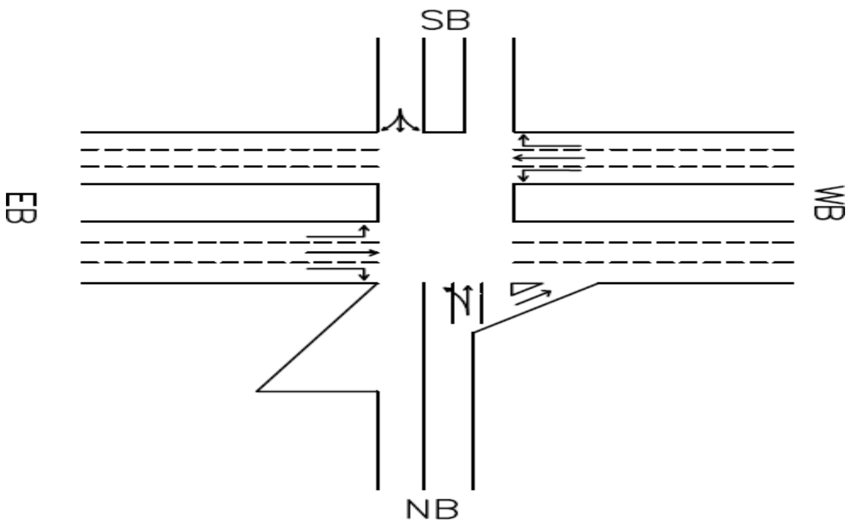
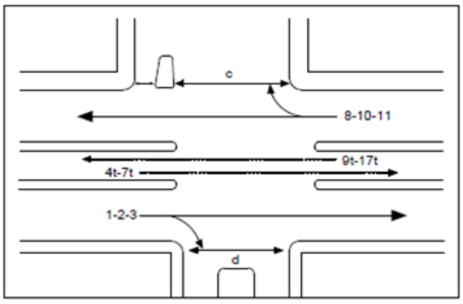
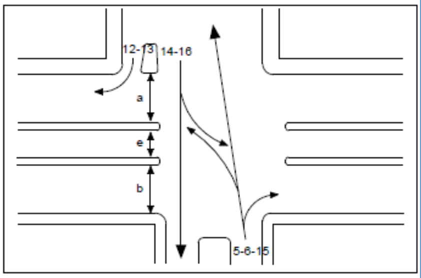


Figure 10: Junction 1-Actual Schema of the junction

BOUNDS	Street denomination	Number of Lane	Average Lane (m)	Total Width (m)
East	Via Prenestina	3	3.5	10.5
West	Via Prenestina	3	3.5	10.5
North	Largo Telese	3	3.5	10.5
South	Viale Della stazione Prenestina	1	3.5	3.5

Table 4: Approaches of 1st intersection

<table>
<tr><th>PHASES</th><th>

FASE 1</th><th>

FASE 2</th></tr>
<tr><td>Green [s]</td><td>57</td><td>39</td></tr>
<tr><td>Yellow [s]</td><td>6</td><td>6</td></tr>
<tr><td>Red [s]</td><td>47</td><td>63</td></tr>
<tr><td>Cycle [s]</td><td colspan="2">108</td></tr>
</table>

Table 5: signal setting of 1st intersection

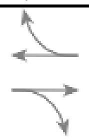
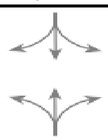
INPUT WORKSHEET

GENERAL INFORMATION		SITE INFORMATION			
ANALYST	Mohammadreza Keramati	INTERSECTION	VIA PRENESTINA - LARGO TELESE		
AGENCY OR		AREA TYPE	CBD		OTHERS
DATE PERFORMED		JURISDICTION			
ANALYSIS TIME	8:45 - 9:00	ANALYSIS YEAR			

VOLUME AND TIME INPUT

	WB			EB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	1456	112	4	628	172	400	132	20	16	108	296
Heavy vehicles,% HV	12.2			16.8			1.4			7.6		
Peak-hour factor, PHF	0.9			0.9			0.9			0.9		
Pretimed (P) or actuated (A)	P			P			P			P		
Start-up lost time, l1 (s)	1.5			1.5			1.5			1.5		
Clearance lost time, l2 (s)	1.5			1.5			1.7			1.7		
Total lost time (s)	3.0			3.0			3.2			3.2		
Arrival type, AT	3			3			3			3		
Approach pedestrian volume,2 vped (p/h)	50			50			50			50		
Approach bicycle volume,2 vbic	20			20			20			20		
Parking (Y or N)	Y			Y			Y			Y		
Parking maneuvers, Nm (maneuvers/h)	32			32			16			32		
Bus stopping, NB (buses/h)	10			8			2			2		
Min. timing for pedestrians,3 Gp (s)	12.1			12.1			12.1			6.5		
pedestrians/cycle	1.5			1.5			1.5			1.5		

SIGNAL PHASING PLAN

	Ø1		Ø2		Ø3		Ø4		Ø5		Ø6		Ø7		Ø8	
DIAGRAM																
Timing	G=	57	G=	39	G=		G=		G=		G=		G=		G=	
	Y=	4	Y=	4	Y=		Y=		Y=		Y=		Y=		Y=	

Permitted Turns	Permitted Turns	Pedestrian	Cycle Length, C=	108	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.

Table 6: Input Worksheet of 1st intersection

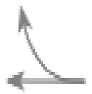
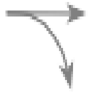
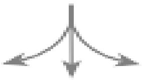
VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET												
General Information												
Project Description: Via Prenestina - Largo Telese												
Volume Adjustment												
	WB			EB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	1456	112	4	628	172	400	132	20	16	108	296
Peak-hour factor, PHF	0.9			0.9			0.9			0.9		
Adjusted flow rate, v_p=V/PHF (veh/h)	0	1618	124	4	698	191.11	444	147	22.2	18	120	329
Lane Group												
Adjusted flow rate in lane group, v (veh/h)	1742			893			444	147	22	467		
Proportion of LT or RT (P_LT , P_RT)	-	-	0.071	-	-	0.214	1	-	1	0.038	-	0.705
Saturation flow rate												
base saturation flow, S_0 (pc/h/ln)	2100			2100			2100	2100	2100	2100		
number of lanes , N	3			3			1	1	1	1		
Lane width adjustment facto, f_W	0.989			0.989			0.99	0.99	0.99	0.989		
Heavy-vehicle adjustment factor, f_HV	0.891			0.856			1	0.986	1	0.929		
Grade adjustment factor, f_g	1			1			1	1	1	1		
Parking adjustment factor, f_p	0.913			0.913			1	1	0.900	0.913		
Bus blockage adjustment factor, f_bb	0.987			0.989			1	1	1	1		
Area type adjustment factor, f_a	1			1			1	1	1	1		
Lane Utilization adjustment factor, f_LU	0.908			0.908			1	1	1	0.908		
Left-Turn adjustment afctor, f_LT	1			1			0.592	1	1	0.970		
Right-Turn adjustment afctor, f_RT	0.989			0.968			1	1	0.850	0.894		
Left-Turn ped/bike adjustment afctor, f_Rpb	1			1			0.978			0.999		
right-Turn ped/bike adjustment afctor, f_Rpb	0.990			0.994			0.936			0.955		
Adjusted saturation flow, s(veh/h) s =	4449			4212			1126	1870	1454	1321		
NOTES												
1. PLT = 1.000 for exclusive left-turn lanes, and PRT = 1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group.												

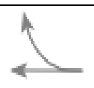
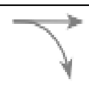
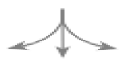
Table 7: Volume Adjustment and Saturation Flow Rate Worksheet of 1st Intersection

CAPACITY AND LOS WORKSHEET

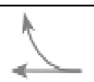
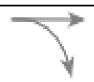
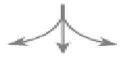
General Information

Project Description: via Prenestina - Largo telese

Capacity Analysis

	WB			EB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Phase number	1			1			2	2	2	2		
Phase type	P			P			P	P	P	P		
Lane Group												
Adjusted flow rate, V (veh/h)	1742			893			444	147	22	467		
Saturation flow rate, S (veh/h)	4449			4212			1126	1870	1454	1321		
Lost time t_L (s), $t_L = l_1 + Y - e$	3			3			3.2			3.2		
Effective green time g (s), $g = G + Y - t_L$	58			58.0			40			39.8		
Green ratio , g/C	0.537			0.537			0.369			0.369		
Lane group capaciry, c=S*(g/C) , (veh/h)	2389			2262			415	689	536	487		
V/c ratio , X	0.729			0.395			1.07	0.21	0.04	0.959		
Flow ratio, V/S	0.392			0.212			0.39	0.08	0.02	0.353		
Critical Lane group/phase (√)	√						√					
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)	0.786											
Total Lost time per cycle, L (s)	6.2											
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$	0.834											

Lane Group Capacity, Control Delay, and LOS Determination

	EB	WB	NB			SB
Lane Group						
Adjusted flow rate, V (veh/h)	1742	893	444	147	22	467
V/c ratio , X=V/c	0.729	0.395	1.07	0.21	0.04	0.959
Total Green ration , g/C	0.537	0.537	0.369			0.369
Uniform delay, $d_1 = \frac{0.50\,C\,[1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)	19	14.69	34.10	23.37	21.87	33.30
Incremental Delay Calibration, K	0.5	0.5	0.5	0.5	0.5	0.5
Incremental delay,[4] d_2 $d_2 = 900T[(X - 1) + \sqrt{(X - 1)^2 + \frac{8kIX}{cT}}]$ (s/veh)	2.0	0.5	64.4	0.70	0.15	31.7
Initial queue Delay, d3 (s/veh)	0	0	0	0	0	0
Progression Adjustment Factor, PF	1	1	1	1	1	1
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	21.02104482	15.20843824	98.5	24.1	22.01	65.05043351
LOS by lane group	C	B	F	C	C	E
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	21.02104482	15.208	77.91374318			65.05043351
LOS by Approach	C	B	E			E
Approach flow rate, V_A (veh/h)	1742	893	613			467
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)	34.5	Intersection LOS				C

NOTES

1. For permitted left turns, the minimum capacity is (1 + PL)(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.

Table 8: Capacity and LOS Worksheet 1st intersection

SUPPLEMENTAL WORKSHEET FOR PERMITTED LEFT TURNS OPPOSED BY SINGLE-LANE APPROACH				
General Information				
Project Description: Via Prenestina - Largo Telese				
Input				
	EB	WB	NB	SB
	LT / TH / RT	LT / TH / RT	LT / TH / RT	LT / TH / RT
Cycle length, C (s)			108	108
Total actual green time for LT lane group,1 G (s)			39	39
Effective permitted green time for LT lane group,1 g (s)			39.8	39.8
Opposing effective green time, go (s)			39.8	39.8
Number of lanes in LT lane group,2 N			1	1
Number of lanes in opposing approach, No			1	1
Adjusted LT flow rate, vLT (veh/h)			444	18
Proportion of LT volume in LT lane group,3 PLT			1.00	0.0381
Proportion of LT volume in opposing flow PLTo			0.70	1
Adjusted flow rate for opposing approach, vo (veh/h)			120	146.7
Lost time for LT lane group, tL			3.2	3.2
Computation				
	EB	WB	NB	SB
LT volume per cycle, LTC = vLT.C/3600			13.33	0.53
Opposing flow per lane, per cycle, $v_{olc} = v_o C/3600$ (veh/C/ln)			3.60	4.40
$g_f = G[e^{-0.860(LTC^{0.629})}] - t_L$ $g_f \le g$ (except exclusive left-turn lanes)[3]			0	18.7
Opposing platoon ratio, Rpo(refer to Exhibit 16-11)			1	1
Opposing queue ratio, qro = max[1 – Rpo(go/C), 0]			0.63	0.63
$g_q = 4.943 v_{olc}^{0.762} qr_o^{1.061} - t_L$ $g_q \le g$			4.86	6.19
gu = g – gq if gq ≥ gf, or gu = g – gf if gq < gf			34.94	21.1
$n = \max[(g_q - g_f)/2, 0]$			2.43	0.00
PTHo=1-PLTo			0.30	0.00
EL1 (refer to Exhibit C16-3)			1.70	1.74
$E_{L2} = \max[(1 - P_{THo}^{n})/P_{LTo}, 1.0]$			1.35	1.00
fmin = 2(1 + PL)/g			0.1005	0.0522
$g_{diff} = \max[g_q - g_f, 0]$ (except when left-turn volume is 0)[4]			4.86	0.00
$f_{LT} = f_m = [g_f/g] + \left[\frac{g_u/g}{1 + P_{LT}(E_{L1} - 1)}\right] + \left[\frac{g_{diff}/g}{1 + P_{LT}(E_{L2} - 1)}\right]$ $(f_{min} \le f_m \le 1.00)$			0.606207662	0.985341152

NOTES

1. Refer to Exhibits C16-4, C16-5, C16-6, C16-7, and C16-8 for case-specific parameters and adjustment factors.
2. For exclusive left-turn lanes, N is equal to the number of exclusive left-turn lanes. For shared left-turn lanes, N is equal to the sum of the shared left-turn, through, and shared right-turn (if one exists) lanes in that approach.
3. For exclusive left-turn lanes, $g_f = 0$, and skip the next step. Lost time, t_L, may not be applicable for protected-permitted case.
4. If the opposing left-turn volume is 0, then $g_{diff} = 0$.

Table 9: Supplemental Worksheet for Permitted Left Turns Opposed by Single-lane Approach-1st intersection

SUPPLEMENTAL WORKSHEET FOR PEDESTRIAN-BICYCLE EFFECTS ON PERMITTED LEFT TURNS AND RIGHT TURNS				
General Information				
Project Description: via Prenestina- largo telese				
Permitted Left Turns				
	EB	WB	NB	SB
	LT TH RT	LT TH RT	LT TH RT	LT TH RT
Effective pedestrian green time,1,2 gp (s)			39	39
Conflicting pedestrian volume,1 vped (p/h)			50	50
vpedg = vped (C/gp)			138.46	138.46
OCCpedg = vpedg/2000 if (vpedg ≤ 1000) or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)			0.069	0.069
Opposing queue clearing green,3,4 gq (s)			0	0
Effective pedestrian green consumed by opposing vehicle queue, gq/gp; if gq ≥ gp then fLpb = 1.0			0	0
OCCpedu = OCCpedg [1 – 0.5(gq/gp)]			0.069	0.069
Opposing flow rate,3 vo (veh/h)			467	613
OCCr = OCCpedu [e^(–(5/3600)vo)]			0.036	0.030
Number of cross-street receiving lanes,1 Nrec			2	2
Number of turning lanes,1 Nturn			1	1
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn			0.978	0.982
Proportion of left turns,5 PLT			1.00	0.038
Proportion of left turns using protected phase,6 PLTA			0	0
fLpb = 1.0 – PLT(1 – ApbT)(1 – PLTA)			0.978	0.999
Permitted Right Turns				
	EB	WB	NB	SB
Effective pedestrian green time,1,2 gp (s)	57	57	39	39
Conflicting pedestrian volume,1 vped (p/h)	50	50	50	50
Conflicting bicycle volume,1,7 vbic (bicycles/h)	20	20	20	20
vpedg = vped(C/gp)	94.74	94.74	138.46	138.46
OCCpedg = vpedg/2000 if (vpedg ≤ 1000), or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)	0.0474	0.0474	0.0692	0.0692
Effective green,1 g (s)	57	57	39	39
vbicg = vbic(C/g)	38	38	55	55
OCCbicg = 0.02 + vbicg/2700	0.034	0.034	0.041	0.041
OCCr = OCCpedg + OCCbicg – (OCCpedg)(OCCbicg)	0.0798	0.0798	0.1069	0.1069
Number of cross-street receiving lanes,1 Nrec	2	1	2	2
Number of turning lanes,1 Nturn	1	1	1	1
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn	0.952	0.920	0.936	0.936
Proportion of right turns,5 PRT	0.21	0.07	1.00	0.70
Proportion of right turns using protected phase,8 PRTA	0	0	0	0
fRpb = 1.0 – PRT(1 – ApbT)(1 – PRTA)	0.990	0.994	0.936	0.955

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 gq = 0, vo = 0, and OCCr = OCCpedu = OCCpedg.
5. Refer to Volume Adjustment and Saturation Flow Rate Worksheet.
6. Ideally determined from field data; alternatively, assume it equal to (1 – permitted phase fLT)/0.95.
7. If vbic = 0 then vbicg = 0, OCCbicg = 0, and OCCr = OCCpedg.
8. PRTA is the proportion of protected green over the total green , gprot/(gprot+ gperm). If only permitted right-turn phase exists, then PRTA = 0.

Table 10: Supplemental Worksheet for Pedestrian- bicycle Effects on Permitted Left Turns and Right turns-1st intersection

3.9.2. Junction 2. Via Prenestina – Viale Ronchi-0610

The Second analyzed intersection has 3 approaches, Via Prenestina – Viale Ronchi. The signal setting for this junction is composed by 3 phases.

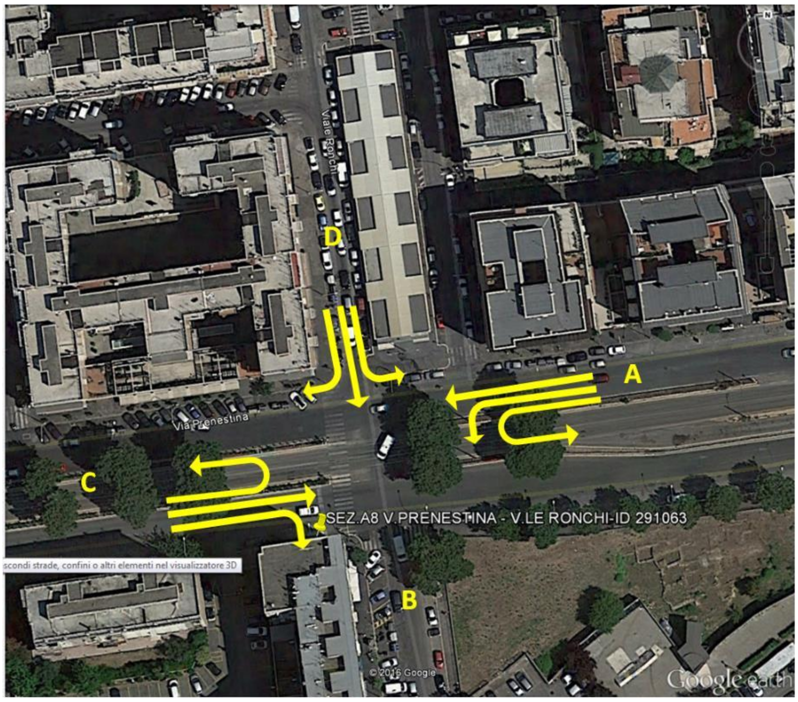


Figure 11: Junction 2

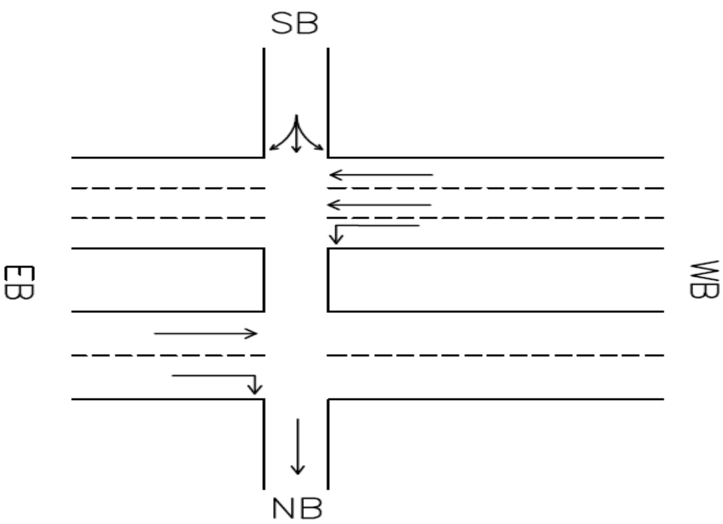
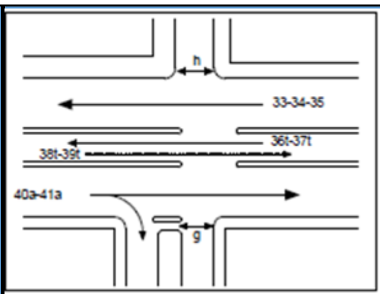
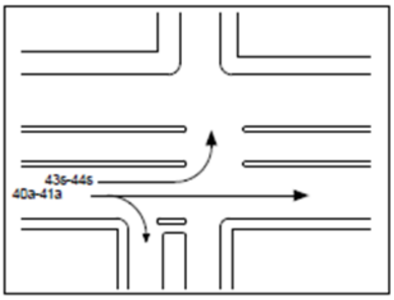


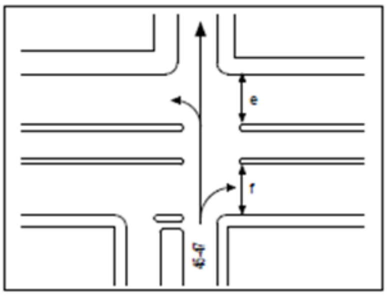
Figure 12: Junction 2 -Actual Schema of the junction

BOUNDS	Street denomination	Number of Lane	Average Lane (m)	Total Width (m)
East	Via Prenestina	3	3.5	10.5
West	Via Prenestina	3	3.5	10.5
South	Viale Ronchi	1	3.5	3.5

Table 11: Approaches of 2th intersection

<table>
<tr><th>PHASES</th><td>

FASE 1</td><td>

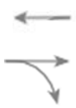
FASE 2</td><td>

FASE 3</td></tr>
<tr><td>Green [s]</td><td>43</td><td>33</td><td>38</td></tr>
<tr><td>Yellow [s]</td><td>4</td><td>4</td><td>4</td></tr>
<tr><td>Red [s]</td><td>85</td><td>97</td><td>90</td></tr>
<tr><td>Cycle [s]</td><td colspan="2">132</td><td></td></tr>
</table>

Table 12: Approaches of 2th intersection

INPUT WORKSHEET			
GENERAL INFORMATION		SITE INFORMATION	
ANALYST	Mohammadreza Keramati	INTERSECTION	Viale Ronchi - Largo Irpinia - via Pren - Via pren
AGENCY OR		AREA TYPE	CBD / OTHERS
DATE PERFORMED		JURISDICTION	
ANALYSIS TIME	8:45 - 9:00	ANALYSIS YEAR	

VOLUME AND TIME INPUT												
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	604	92	172	1596	0				68	196	36
Heavy vehicles,% HV	13.7			9.5						6		
Peak-hour factor, PHF	0.9			0.9						0.9		
Pretimed (P) or actuated (A)	P			P						P		
Start-up lost time, l1 (s)	1.5			1.5						1.5		
Clearance lost time, l2 (s)	2.0			1.0						2.0		
Total lost time (s)	3.5			2.5						3.5		
Arrival type, AT	3			3						3		
Approach pedestrian volume,2 vped (p/h)	50			50						50		
Approach bicycle volume,2 vbic	20			20						20		
Parking (Y or N)	Y			Y						Y		
Parking maneuvers, Nm (maneuvers/h)	8			8						12		
Bus stopping, NB (buses/h)	6			10						N		
Min. timing for pedestrians,3 Gp (s)	12.1			12.1						6.5		
pedestrians/cycle	1.5			1.5						1.5		

SIGNAL PHASING PLAN								
	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
DIAGRAM								
Timing	G= 43	G= 33	G= 38	G=	G=	G=	G=	G=
	Y= 4	Y= 4	Y= 4	Y=	Y=	Y=	Y=	Y=

Permitted Turns	Permitted Turns	Pedestrian	Cycle Length, C=	132	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.

Table 13: Input Worksheet of 2nd intersection

VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET												
General Information												
Project Description: Via Prenestina - Largo Telese												
Volume Adjustment												
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	604	92	172	1596	0				68	196	36
Peak-hour factor, PHF		0.9			0.9						0.9	
Adjusted flow rate, v_p=V/PHF (veh/h)	0	671	102	191	1773	0.00				76	218	40
Lane Group												
Adjusted flow rate in lane group, v (veh/h)		773		191	1773						333	
Proportion of LT or RT (P_LT , P_RT)	-	-	0.132	1.0	-	-				0.227	-	0.12
Saturation flow rate												
base saturation flow, S_0 (pc/h/ln)		2100		2100	2100						2100	
number of lanes , N		3		1	2						1	
Lane width adjustment facto, f_W		0.989		0.989	0.989						0.989	
Heavy-vehicle adjustment factor, f_HV		0.880		0.913	0.913						0.929	
Grade adjustment factor, f_g		1		1	1						1	
Parking adjustment factor, f_p		0.953		0.953	0.953						0.947	
Bus blockage adjustment factor, f_bb		0.992		0.987	0.987						1	
Area type adjustment factor, f_a		1		1	1						1	
Lane Utilization adjustment factor, f_LU		0.950		1	0.950						1	
Left-Turn adjustment afctor, f_LT		1		0.952	1						0.989	
Right-Turn adjustment afctor, f_RT		0.980		0.850	1						0.982	
Left-Turn ped/bike adjustment afctor, f_Rpb		1		0.836	1						0.931	
right-Turn ped/bike adjustment afctor, f_Rpb		0.954		1	1						0.957	
Adjusted saturation flow, s(veh/h) s =		4574		1208	3388						1577	
NOTES												
1. PLT = 1.000 for exclusive left-turn lanes, and PRT = 1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group.												

Table 14: Volume Adjustment and Saturation Flow Rate Worksheet of 2nd Intersection

CAPACITY AND LOS WORKSHEET

General Information

Project Description: via Prenestina - Largo telese

Capacity Analysis

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Phase number		1			1&2						2	
Phase type		P			P						P	
Lane Group												
Adjusted flow rate, V (veh/h)		773		191	1773	-					333	
Saturation flow rate, S (veh/h)		4574		1208	3388	-					1577	
Lost time t_L (s), $t_L = l_1 + Y - e$		3			3						3.2	
Effective green time g (s), $g = G + Y - t_L$		44		34.0	84.0	-					38.8	
Green ratio , g/C		0.333		0.258	0.636	-					0.294	
Lane group capaciry, c=S*(g/C) , (veh/h)		1525		311	2156	-					464	
V/c ratio , X		0.507		0.614	0.822	-					0.719	
Flow ratio, V/S		0.169		0.158	0.523	-					0.211	
Critical Lane group/phase (√)					√						√	
Sum of flow ratios for critical lane groups, Y_c $Y_c = \Sigma$ (critical lane groups, v/s)	0.735											
Total Lost time per cycle, L (s)	9.2											
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$	0.790											

Lane Group Capacity, Control Delay, and LOS Determination

	EB			WB			NB			SB		
Lane Group												
Adjusted flow rate, V (veh/h)		773		191	1773	-					333	
V/c ratio , X=V/c		0.507		0.614	0.822	-					0.719	
Total Green ration , g/C		0.333		0.258	0.636	-					0.294	
Uniform delay, $d_1 = \frac{0.50\,C\,[1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)		35		43.22	18.31	-					41.72	
Incremental Delay Calibration, K		0.5		0.5	0.5	-					0.5	
Incremental delay,[4] d_2 $d_2 = 900T[(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}]$ (s/veh)		1.2		8.8	3.7	-					9.3	
Initial queue Delay, d3 (s/veh)		0		0	0	-					0	
Progression Adjustment Factor, PF		1		1	1	-					1	
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)		36.5		52.0	22						51.0	
LOS by lane group		D		D	C	-					D	
Delay by approach, $d_A = \frac{\Sigma(d)(v)}{\Sigma v}$ (s/veh)		36.5		52.0	22.0	-					51.0	
LOS by Approach		D		D	C	-					D	
Approach flow rate, V_A (veh/h)		773		191	1773	-					333	
Intersection delay, $d_I = \frac{\Sigma(d_A)(v_A)}{\Sigma v_A}$ (s/veh)		30.7		Intersection LOS							C	

NOTES

1. For permitted left turns, the minimum capacity is (1 + PL)(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.

Table 15: Capacity and LOS Worksheet Approach-2nd intersection

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

General Information

Project Description: via Prenestina- largo telese

Permitted Left Turns

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Effective pedestrian green time,1,2 gp (s)					12.1						6.5	
Conflicting pedestrian volume,1 vped (p/h)					50						50	
vpedg = vped (C/gp)					545.5						1015.4	
OCCpedg = vpedg/2000 if (vpedg ≤ 1000) or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)					0.27						0.51	
Opposing queue clearing green,3,4 gq (s)					0						0	
Effective pedestrian green consumed by opposing vehicle queue, gq/gp; if gq ≥ gp then fLpb = 1.0					0						0	
OCCpedu = OCCpedg [1 – 0.5(gq/gp)]					0.27						0.51	
Opposing flow rate,3 vo (veh/h)					0						0	
OCCr = OCCpedu [e^(–(5/3600)vo)]					0.27						0.51	
Number of cross-street receiving lanes,1 Nrec					1						2	
Number of turning lanes,1 Nturn					1						1	
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn					0.84						0.70	
Proportion of left turns,5 PLT					1						0.23	
Proportion of left turns using protected phase,6 PLTA					0						0	
fLpb = 1.0 – PLT(1 – ApbT)(1 – PLTA)					0.84						0.93	

Permitted Right Turns

	EB	WB	NB	SB
Effective pedestrian green time,1,2 gp (s)	12.1			6.5
Conflicting pedestrian volume,1 vped (p/h)	50			50
Conflicting bicycle volume,1,7 vbic (bicycles/h)	20			20
vpedg = vped(C/gp)	545.5			1015.4
OCCpedg = vpedg/2000 if (vpedg ≤ 1000), or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)	0.27			0.51
Effective green,1 g (s)	12.1			6.5
vbicg = vbic(C/g)	218			406
OCCbicg = 0.02 + vbicg/2700	0.10			0.17
OCCr = OCCpedg + OCCbicg – (OCCpedg)(OCCbicg)	0.35			0.59
Number of cross-street receiving lanes,1 Nrec	1			2
Number of turning lanes,1 Nturn	1			1
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn	0.654			0.645
Proportion of right turns,5 PRT	0.13			0.12
Proportion of right turns using protected phase,8 PRTA	0			0
fRpb = 1.0 – PRT(1 – ApbT)(1 – PRTA)	0.954			0.957

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 gq = 0, vo = 0, and OCCr = OCCpedu = OCCpedg.
5. Refer to Volume Adjustment and Saturation Flow Rate Worksheet.
6. Ideally determined from field data; alternatively, assume it equal to (1 – permitted phase fLT)/0.95.
7. If vbic = 0 then vbicg = 0, OCCbicg = 0, and OCCr = OCCpedg.
8. PRTA is the proportion of protected green over the total green , gprot/(gprot+ gperm). If only permitted right-turn phase exists, then PRTA = 0.

Table 16: Supplemental Worksheet for Pedestrian- bicycle Effects on Permitted Left Turns and Right turns-2nd intersection

3.9.3. Junction 3. Via Prenestina – Via Dignano_Distria-0611

This Junction is Via Prenestina – Largo Irpinia – Via Dignano D' Istria which is composed by 3 approaches has a signal setting composed by 2 phases

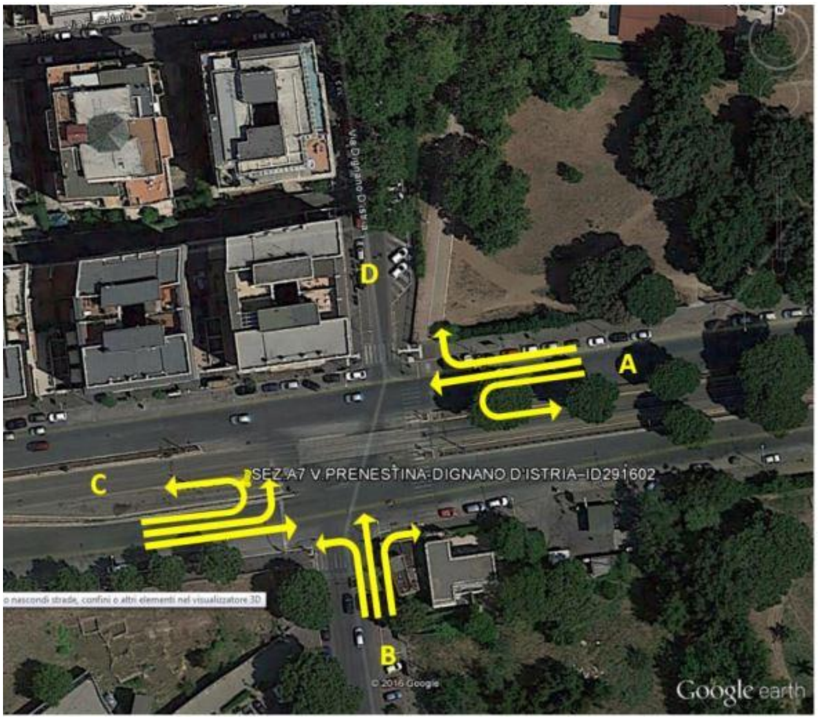


Figure 12 junction 3

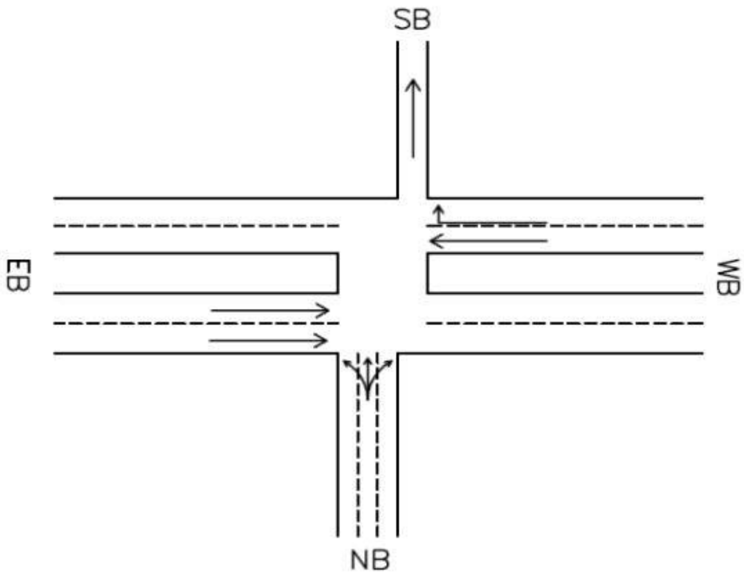


Figure 13 junction 3 scheme

Approach	Name	Number Of Lanes	Average Lane (m)	Total Width (m)
NB	Largo irpinia	1	3.5	3.5
EB	Via Prenestina	3	3.5	10.5
WB	Via Prenestina	3	3.5	10.5

Table 17 3th junction approaches

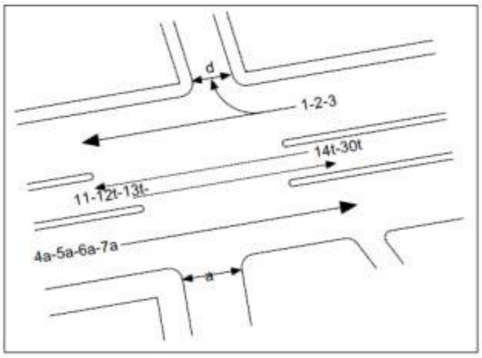
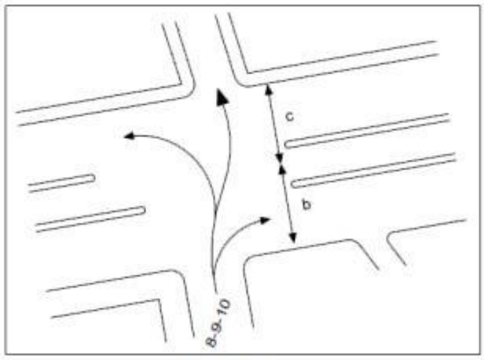
Phases	 FASE 1	 FASE 2
Green (s)	76	44
Yellow (s)	6	6
Red (s)	50	82
Cycle (s)	132	

Table 18 Signal setting of 3th approach

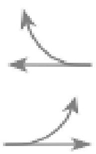
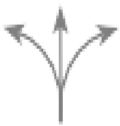
INPUT WORKSHEET

GENERAL INFORMATION		SITE INFORMATION			
ANALYST	Jovan Nazari	INTERSECTION	V.PRENESTINA-DIGNANO D'ISTRIA		
AGENCY OR COMPANY		AREA TYPE	CBD		OTHERS
DATE PERFORMED		JURISDICTION			
ANALYSIS TIME PERIOD	8:45-9:00	ANALYSIS YEAR			

VOLUME AND TIME INPUT

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	344	-	-	1596	176	116	276	104	-	-	-
Heavy vehicles,% HV	20			4.5			11					
Peak-hour factor, PHF	0.9			0.9			0.9					
Pretimed (P) or actuated (A)	P			P			P					
Start-up lost time, l1 (s)	2.0			2.0			2.0					
Clearance lost time (s)	2.5			2.6			2.8					
Total lost time (s)	4.5			4.6			4.8					
Arrival type, AT	3			3			3					
Approach pedestrian volume,2 vped (p/h)	50			50			50					
Approach bicycle volume,2 vbic (bicycles/h)	20			20			20					
Parking (Y or N)	Y			Y			Y					
Parking maneuvers, Nm (maneuvers/h)	8			8			12					
Bus stopping, NB (buses/h)	6			8			6					
Min. timing for pedestrians,3 Gp (s)	12.1			12.1			6.5					

SIGNAL PHASING PLAN

	Ø1		Ø2		Ø3		Ø4		Ø5		Ø6		Ø7		Ø8	
DIAGRAM																
Timing	G=	76	G=	44	G=		G=		G=		G=		G=		G=	
	Y=	6	Y=	6	Y=		Y=		Y=		Y=		Y=		Y=	

Permitted Turns	Permitted Turns	- - - - Pedestrian	Cycle Length, C=	132 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.

Table 19 input worksheet 3th junction

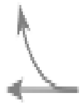
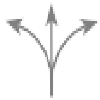
VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET												
General Information												
V.PRENESTINA-DIGNANO D'ISTRIA												
Volume Adjustment												
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	344	0	0	1596	176	116	276	104			
Peak-hour factor, PHF	0.9			0.9			0.9					
Adjusted flow rate, v_p=V/PHF (veh/h)	0	382	0	0	1773	195.56	128.889	306.7	115.56			
Lane Group												
Adjusted flow rate in lane group, v (veh/h)	382			1969			551					
Proportion of LT or RT (P_LT , P_RT)	0.000	-	0.00	0.00	-	0.10	0.234	-	0.210			
Saturation flow rate												
base saturation flow, S_0 (pc/h/ln)	2100			2100			2100					
number of lanes , N	3			3			1					
Lane width adjustment facto, f_W	1.044			1.044			1.04					
Heavy-vehicle adjustment factor, f_HV	0.833			0.957			0.901					
Grade adjustment factor, f_g	1			1			1					
Parking adjustment factor, f_p	0.953			0.953			0.840					
Bus blockage adjustment factor, f_bb	0.992			0.989			0.976					
Area type adjustment factor, f_a	1			1			1					
Lane Utilization adjustment factor, f_LU	0.95			0.95			1					
Left-Turn adjustment afctor, f_LT	1.000			1			0.950					
Right-Turn adjustment afctor, f_RT	1.000			0.985			0.950					
Left-Turn ped/bike adjustment afctor, f_Rpb	1			1			0.98					
right-Turn ped/bike adjustment afctor, f_Rpb	0.997			0.988			0.988					
Adjusted saturation flow, s(veh/h) s = s0*N*fw*fhv*fg*fp*fbb*fa*fLU*fLT*fRT*fLpb*fRpb	4906			5485			1410					
NOTES												
1. PLT = 1.000 for exclusive left-turn lanes, and PRT = 1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group.												

Table 20 Volume Adjustment and Saturation Flow Rate Worksheet of 3rd junction

CAPACITY AND LOS WORKSHEET				
General Information				
V.PRENESTINA-DIGNANO D'ISTRIA				
Capacity Analysis				
	EB	WB	NB	SB
	LT, TH, RT	LT, TH, RT	LT, TH, RT	LT, TH, RT
Phase number	1	1	2	
Phase type	P	P	P	
Lane Group				
Adjusted flow rate, V (veh/h)	382	1969	551	
Saturation flow rate, S (veh/h)	4906	5485	1410	
Lost time t_L (s), $t_L = l_1 + Y - e$	4.5	4.6	4.8	
Effective green time g (s), $g = G + Y - t_L$	77.5	77.4	45.2	
Green ratio , g/C	0.587	0.586	0.342	
Lane group capaciry, c=S*(g/C) , (veh/h)	2880	3214	482	
V/c ratio , X	0.133	0.501	1.14	
Flow ratio, V/S	0.078	0.359	0.39	
Critical Lane group/phase (√)		√	√	
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)	0.749			
Total Lost time per cycle, L (s)	9.4			
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$	0.806			

Lane Group Capacity, Control Delay, and LOS Determination				
	EB	WB	NB	SB
Lane Group				
Adjusted flow rate, V (veh/h)	382	1969	551	
V/c ratio , X=V/c	0.133	0.501	1.14	
Total Green ration , g/C	0.587	0.586	0.342	
Uniform delay, $d_1 = \frac{0.50\,C\,[1 - (g/C)]^2}{1 - [\min(1, X)g/C]}$ (s/veh)	12	16.01	43.43	
Incremental Delay Calibration, K	0.5	0.5	0.5	
Incremental delay,[4] d_2 $d_2 = 900T[(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}]$ (s/veh)	0.1	0.6	85.4	
Initial queue Delay, d3 (s/veh)	0	0	0	
Progression Adjustment Factor, PF	1	1	1	
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	12.1	16.61	128.83	
LOS by lane group	B	B	F	
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	12.1	16.610	128.83	
LOS by Approach	B	B	F	
Approach flow rate, V_A (veh/h)	382	1969	551	
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)	37.3	Intersection LOS		D

NOTES

1. For permitted left turns, the minimum capacity is (1 + PL)(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.

Table 21: Capacity and LOS Worksheet Approach-3th intersection

SUPPLEMENTAL WORKSHEET FOR PEDESTRIAN-BICYCLE EFFECTS ON PERMITTED LEFT TURNS AND RIGHT TURNS												
General Information												
V.PRENESTINA-DIGNANO D'ISTRIA												
Permitted Left Turns												
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Effective pedestrian green time,1,2 gp (s)							6.90					
Conflicting pedestrian volume,1 vped (p/h)							50					
vpedg = vped (C/gp)							956.52					
OCCpedg = vpedg/2000 if (vpedg ≤ 1000) or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)							0.478					
Opposing queue clearing green,3,4 gq (s)							0.00					
Effective pedestrian green consumed by opposing vehicle queue, gq/gp; if gq ≥ gp then fLpb = 1.0							0.000					
OCCpedu = OCCpedg [1 – 0.5(gq/gp)]							0.4780					
Opposing flow rate,3 vo (veh/h)							0					
OCCr = OCCpedu [e^(–(5/3600)vo)]							0.478					
Number of cross-street receiving lanes,1 Nrec							2					
Number of turning lanes,1 Nturn							1					
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn							0.713					
Proportion of left turns,5 PLT							0.23					
Proportion of left turns using protected phase,6 PLTA							0					
fLpb = 1.0 – PLT(1 – ApbT)(1 – PLTA)							0.934					

Permitted Right Turns				
	EB	WB	NB	SB
Effective pedestrian green time,1,2 gp (s)		13.3	6.9	
Conflicting pedestrian volume,1 vped (p/h)		50	50	
Conflicting bicycle volume,1,7 vbic (bicycles/h)		20	20	
vpedg = vped(C/gp)		406.02	782.61	
OCCpedg = vpedg/2000 if (vpedg ≤ 1000), or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)		0.2030	0.3913	
Effective green,1 g (s)		77.4	45.2	
vbicg = vbic(C/g)		28	58	
OCCbicg = 0.02 + vbicg/2700		0.030	0.041	
OCCr = OCCpedg + OCCbicg – (OCCpedg)(OCCbicg)		0.2269	0.4163	
Number of cross-street receiving lanes,1 Nrec		1	2	
Number of turning lanes,1 Nturn		1	1	
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn		0.773	0.750	
Proportion of right turns,5 PRT		0.10	0.21	
Proportion of right turns using protected phase,8 PRTA		0	0	
fRpb = 1.0 – PRT(1 – ApbT)(1 – PRTA)		0.977	0.948	

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 gq = 0, vo = 0, and OCCr = OCCpedu = OCCpedg.
5. Refer to Volume Adjustment and Saturation Flow Rate Worksheet.
6. Ideally determined from field data; alternatively, assume it equal to (1 – permitted phase fLT)/0.95.
7. If vbic = 0 then vbicg = 0, OCCbicg = 0, and OCCr = OCCpedg.
8. PRTA is the proportion of protected green over the total green , gprot/(gprot+ gperm). If only permitted right-turn phase exists, then PRTA = 0.

Table 22: Supplemental Worksheet for Pedestrian- bicycle Effects on Permitted Left Turns and Right turns-3th junction

3.9.4 Via Prenestina – Via Olevano Romano 06012

This Junction is Via Prenestina – Via Olevano Romano which is composed by 3 approaches has a signal setting composed by 2 phases.

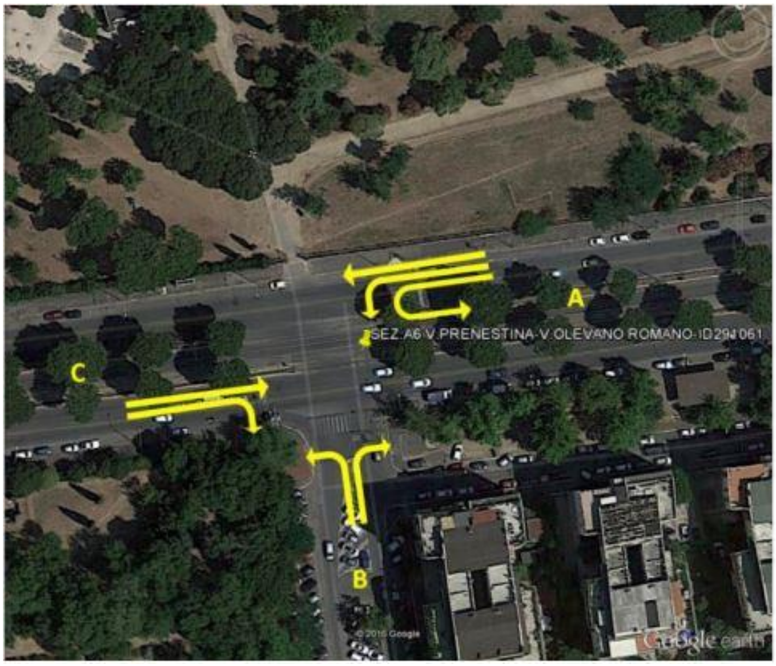


Figure 14 4th junction

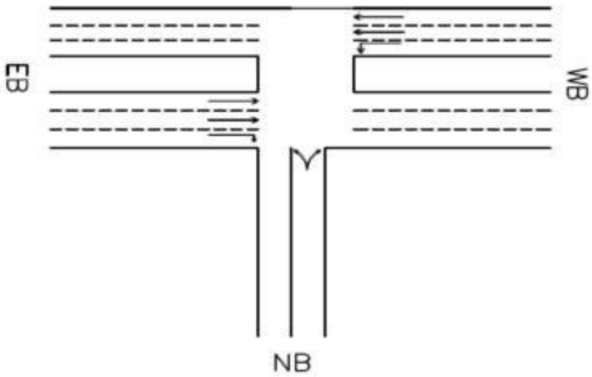


Figure 15 : Junction 4 -Actual Schema of the junction

Approach	Name	Number Of Lanes	Average Lane Width (m)	Total Width (m)
NB	via Olevano Romano	1	3.5	3.5
EB	Via Prenestina	3	3.5	10.5
WB	Via Prenestina	2	3.5	7

Table 23 Approaches of 4th intersection

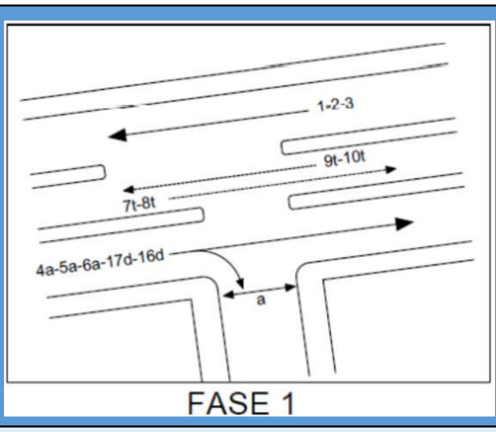
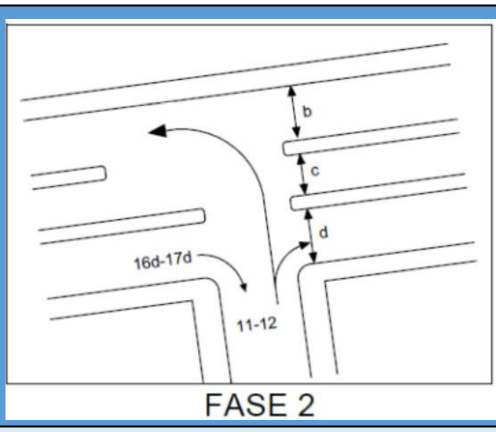
Phases		
Green (s)	76	44
Yellow (s)	6	6
Red (s)	50	82
Cycle (s)	132	

Table 24: signal setting of 4th intersection

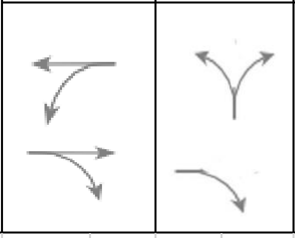
INPUT WORKSHEET

GENERAL INFORMATION		SITE INFORMATION			
ANALYST	Jovan Nazari	INTERSECTION	V.PRENESTINA-V.OLEVANO ROMA		
AGENCY OR COMPANY		AREA TYPE	CBD		OTHERS
DATE PERFORMED		JURISDICTION			
ANALYSIS TIME PERIOD	13:15-13:30	ANALYSIS YEAR			

VOLUME AND TIME INPUT

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	756	156	0	864	0	148	0	52	-	-	-
Heavy vehicles,% HV	8			9			0					
Peak-hour factor, PHF	0.9			0.9			0.9					
Pretimed (P) or actuated (A)	P			P			P					
Start-up lost time, l1 (s)	2.0			2.0			2.0					
Clearance lost time (s)	2.3			2.4			2.0					
Total lost time (s)	4.3			4.4			4.0					
Arrival type, AT	3			3			3					
Approach pedestrian volume,2 vped (p/h)	50			50			50					
Approach bicycle volume,2 vbic (bicycles/h)	20			20			20					
Parking (Y or N)	Y			Y			Y					
Parking maneuvers, Nm (maneuvers/h)	8			8			8					
Bus stopping, NB (buses/h)	6			6			6					
Min. timing for pedestrians,3 Gp (s)	12.1			9.2			6.5					

SIGNAL PHASING PLAN

	Ø1	Ø2	Ø3	Ø4	Ø5	Ø6	Ø7	Ø8
DIAGRAM								
Timing	G= 75	G= 45	G=	G=	G=	G=	G=	G=
	Y= 6	Y= 6	Y=	Y=	Y=	Y=	Y=	Y=

Permitted Turns	Permitted Turns	- - - - Pedestrian	Cycle Length, C= 132 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.

Table 25 Input Worksheet of 4th intersection

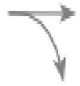
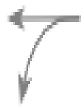
VOLUME ADJUSTMENT AND SATURATION FLOW RATE WORKSHEET												
General Information												
V.PRENESTINA-V.OLEVANO ROMANO												
Volume Adjustment												
	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Volume, V (veh/h)	0	756	156	0	864	0	148	0	52			
Peak-hour factor, PHF	0.9			0.9			0.9					
Adjusted flow rate, v_p=V/PHF (veh/h)	0	840	173	0	960	0.00	164.444	0	57.778			
Lane Group												
Adjusted flow rate in lane group, v (veh/h)	1013			960			222					
Proportion of LT or RT (P_LT , P_RT)	0.000	-	0.17	0.00	-	0.00	0.741	-	0.260			
Saturation flow rate												
base saturation flow, S_0 (pc/h/ln)	2100			2100			2100					
number of lanes , N	3			2			1					
Lane width adjustment facto, f_W	0.989			0.989			0.99					
Heavy-vehicle adjustment factor, f_HV	0.926			0.917			1.000					
Grade adjustment factor, f_g	1			1			1					
Parking adjustment factor, f_p	0.953			0.930			0.860					
Bus blockage adjustment factor, f_bb	0.992			0.988			0.976					
Area type adjustment factor, f_a	1			1			1					
Lane Utilization adjustment factor, f_LU	0.952			0.952			1					
Left-Turn adjustment afctor, f_LT	1.000			1			0.950					
Right-Turn adjustment afctor, f_RT	0.975			1.000			0.950					
Left-Turn ped/bike adjustment afctor, f_Rpb	1			1			0.98					
right-Turn ped/bike adjustment afctor, f_Rpb	0.997			0.988			0.988					
Adjusted saturation flow, s(veh/h) s = s0*N*fw*fhv*fg*fp*fbb*fa*fLU*fLT*fRT*fLpb*fRpb	5048			3292			1525					
NOTES												
1. PLT = 1.000 for exclusive left-turn lanes, and PRT = 1.000 for exclusive right-turn lanes. Otherwise, they are equal to the proportions of turning volumes in the lane group.												

Table 26 Volume Adjustment and Saturation Flow Rate Worksheet of 4th intersection

CAPACITY AND LOS WORKSHEET

General Information

V.PRENESTINA-V.OLEVANO ROMANO

Capacity Analysis

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Phase number		1			1			2				
Phase type		P			P			P				
Lane Group												
Adjusted flow rate, V (veh/h)		1013			960			222				
Saturation flow rate, S (veh/h)		5048			3292			1525				
Lost time t_L (s), $t_L = l_1 + Y - e$		4.3			4.4			4				
Effective green time g (s), $g = G + Y - t_L$		76.7			76.6			47				
Green ratio , g/C		0.581			0.580			0.356				
Lane group capaciry, c=S*(g/C) , (veh/h)		2933			1909			543				
V/c ratio , X		0.345			0.411			0.41				
Flow ratio, V/S		0.201			0.292			0.15				
Critical Lane group/phase (√)					√			√				
Sum of flow ratios for critical lane groups, Y_c $Y_c = \sum$ (critical lane groups, v/s)	0.442											
Total Lost time per cycle, L (s)	8.4											
Critical flow rate to capacity ratio, X_c $X_c = (Y_c)(C)/(C - L)$	0.472											

Lane Group Capacity, Control Delay, and LOS Determination

	EB	WB	NB	SB
Lane Group				
Adjusted flow rate, V (veh/h)	1013	960	222	
V/c ratio , X=V/c	0.345	0.411	0.41	
Total Green ration , g/C	0.581	0.580	0.356	
Uniform delay, $d_1 = \frac{0.50\, C\, [1 - (g/C)]^2}{1 - [\min(1, X) g/C]}$ (s/veh)	14	15.29	32.05	
Incremental Delay Calibration, K	0.5	0.5	0.5	
Incremental delay,[4] d_2 $d_2 = 900T[(X-1) + \sqrt{(X-1)^2 + \frac{8kIX}{cT}}]$ (s/veh)	0.3	0.7	2.3	
Initial queue Delay, d3 (s/veh)	0	0	0	
Progression Adjustment Factor, PF	1	1	1	
Delay, $d = d_1(PF) + d_2 + d_3$ (s/veh)	14.3	15.99	34.35	
LOS by lane group	B	B	C	
Delay by approach, $d_A = \frac{\sum(d)(v)}{\sum v}$ (s/veh)	14.3	15.990	34.35	
LOS by Approach	B	B	C	
Approach flow rate, V_A (veh/h)	1013	960	222	
Intersection delay, $d_I = \frac{\sum(d_A)(v_A)}{\sum v_A}$ (s/veh)	17.1	Intersection LOS		B

NOTES

1. For permitted left turns, the minimum capacity is (1 + PL)(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.

Table 27 Capacity and LOS Worksheet Approach-4th intersection

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

General Information

V.PRENESTINA-V.OLEVANO ROMANO

Permitted Left Turns

	EB			WB			NB			SB		
	LT	TH	RT	LT	TH	RT	LT	TH	RT	LT	TH	RT
Effective pedestrian green time,1,2 gp (s)												
Conflicting pedestrian volume,1 vped (p/h)												
vpedg = vped (C/gp)												
OCCpedg = vpedg/2000 if (vpedg ≤ 1000) or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)												
Opposing queue clearing green,3,4 gq (s)												
Effective pedestrian green consumed by opposing vehicle queue, gq/gp; if gq ≥ gp then fLpb = 1.0												
OCCpedu = OCCpedg [1 – 0.5(gq/gp)]												
Opposing flow rate,3 vo (veh/h)												
OCCr = OCCpedu [e^(–(5/3600)vo)]												
Number of cross-street receiving lanes,1 Nrec												
Number of turning lanes,1 Nturn												
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn												
Proportion of left turns,5 PLT												
Proportion of left turns using protected phase,6 PLTA												
fLpb = 1.0 – PLT(1 – ApbT)(1 – PLTA)												

Permitted Right Turns

	EB	WB	NB	SB
Effective pedestrian green time,1,2 gp (s)	12.1		6.5	
Conflicting pedestrian volume,1 vped (p/h)	50		50	
Conflicting bicycle volume,1,7 vbic (bicycles/h)	20		20	
vpedg = vped(C/gp)	545.45		1015.38	
OCCpedg = vpedg/2000 if (vpedg ≤ 1000), or OCCpedg = 0.4 + vpedg/10,000 if (1000 < vpedg ≤ 5000)	0.2727		0.5077	
Effective green,1 g (s)	76.7		47	
vbicg = vbic(C/g)	34		56	
OCCbicg = 0.02 + vbicg/2700	0.033		0.041	
OCCr = OCCpedg + OCCbicg – (OCCpedg)(OCCbicg)	0.2967		0.5279	
Number of cross-street receiving lanes,1 Nrec	1		3	
Number of turning lanes,1 Nturn	1		1	
ApbT = 1 – OCCr if Nrec = Nturn ApbT = 1 – 0.6(OCCr) if Nrec > Nturn	0.703		0.683	
Proportion of right turns,5 PRT	0.17		0.26	
Proportion of right turns using protected phase,8 PRTA	0		0	
fRpb = 1.0 – PRT(1 – ApbT)(1 – PRTA)	0.950		0.918	

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 gq = 0, vo = 0, and OCCr = OCCpedu = OCCpedg.
5. Refer to Volume Adjustment and Saturation Flow Rate Worksheet.
6. Ideally determined from field data; alternatively, assume it equal to (1 – permitted phase fLT)/0.95.
7. If vbic = 0 then vbicg = 0, OCCbicg = 0, and OCCr = OCCpedg.
8. PRTA is the proportion of protected green over the total green , gprot/(gprot+ gperm). If only permitted right-turn phase exists, then PRTA = 0.

Table 28 Supplemental Worksheet for Pedestrian- bicycle Effects on Permitted Left Turns and Right turns-4th intersection

5. Optimization:

Optimization is finding an alternative with the most cost effective or highest achievable performance under the given constraints, by maximizing desired factors and minimizing undesired ones. In this case, after the computation through the HCM method of the delays for each intersection, our purpose is to minimize that delay achieving better performances, higher level of services (LOS). Different methods are used to optimize the signal settings and hence reduce the junction delay and improve the Level of Service for each junction.

5.1. Minimum Cycle:

Minimum green time is the lowest green time needed for all vehicles in the critical phase to pass the intersection without need for the other cycle, if both phases has the critical movements so the corresponding cycle is called minimum cycle length. The minimum cycle length and the minimum green times can be calculated with following equation:

$$C_{min} = \frac{L}{1 - (y_1 + y_2)}$$

$$g_1 = y_1 C,\ g_2 = y_2 C,\ g_1 + g_2 = C - L$$

Equation 9.Min Cycle

Where y1 and y2 are the saturation degrees of the critical movements in each phase and the L is the total lost time for the intersection.

5.2. Webster Optimum Cycle:

Obtained from Webster assumptions, reconditioning the original problem to a one with a single variable, the cycle length, which is computed by assuming the delay model of stationary queue with casual arrivals that overlaps to uniform arrivals. It is equal to the average value of flow; the optimum cycle can be obtained from the equation:

$$C_{opt} = \frac{(1.5 \sum_i l_i) + 5}{1 - \sum_i y_i}$$

Equation 10.Webster Optimum Cycle

Once computed the optimal cycle it is possible to obtain the values for the minimum effective green times with the equation:

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

5.3. Enumerative Method:

The enumerative method has been set in two different ways:

Enumerative method 1(C = constant): The cycle length has 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 has 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. Optimization of Intersections:

6.1. The first intersection is Via Prenestina- Largo Telese

6.1.1 Current Cycle for 1st intersection:

	Cycle	g1	g2	g1/g2	d	LOS
Current Cycle Length	108.00	57.00	39.00	1.46	36.48	D

6.1.2 Minimum Cycle for 1st intersection:

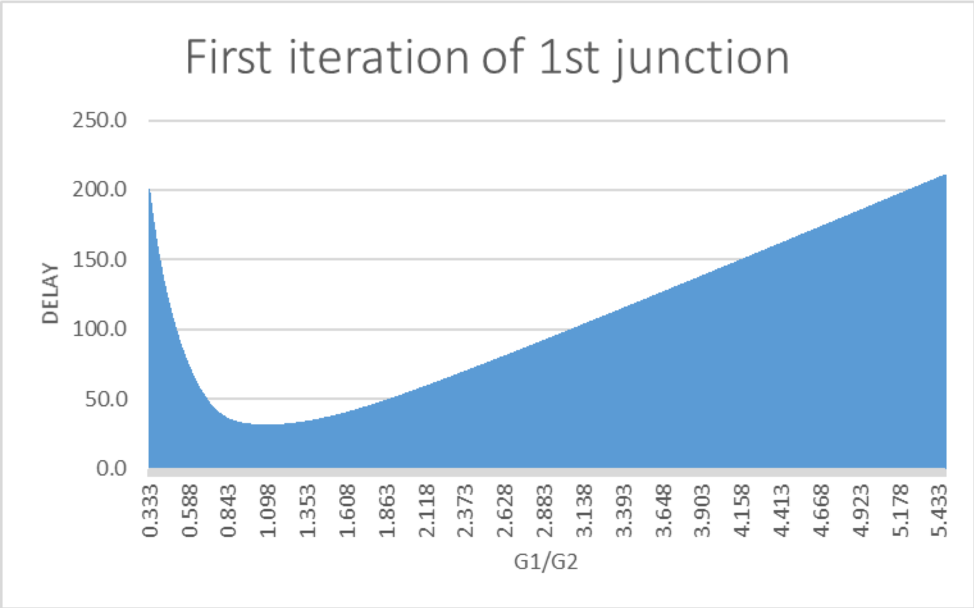
	Cycle	g1	g2	g1/g2	d	LOS
Min Cycle Length	35.21	13.80	13.91	0.99	27.84	C

6.1.3 Webster's Optimum Cycle for 1st intersection:

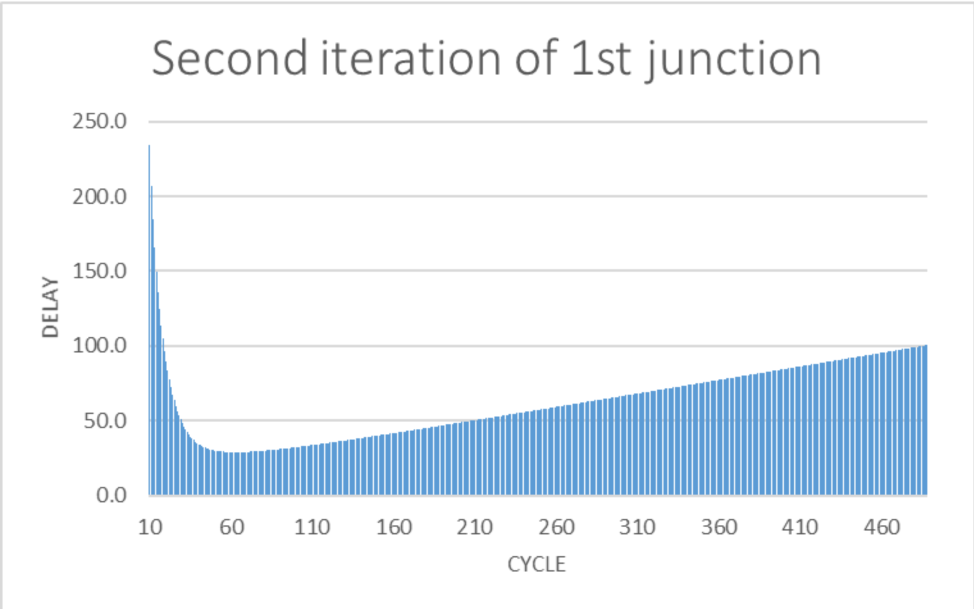
	Cycle	g1	g2	g1/g2	d	LOS
Webster Cycle Length	76.29	34.26	34.53	0.99	23.77	C

6.1.4 Enumerative method for the 1st intersection:

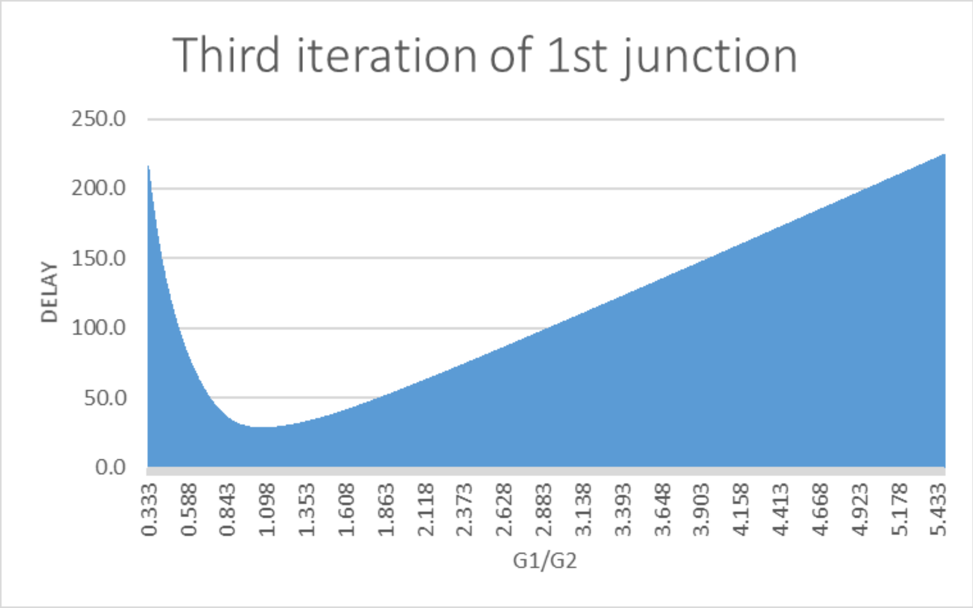
To find the minimum of the delay function, at beginning the cycle has been fixed to 108 sec, making the green ratio varies.



At the first iteration, we found that the minimum delay is 22.432 sec with green ratio (g1/g2) 1.087. In the second iteration the green ratio has been fixed to 1.087, making the cycle time varies.



At the second iteration, the minimum delay is found 21.955 sec with cycle time 75 sec.



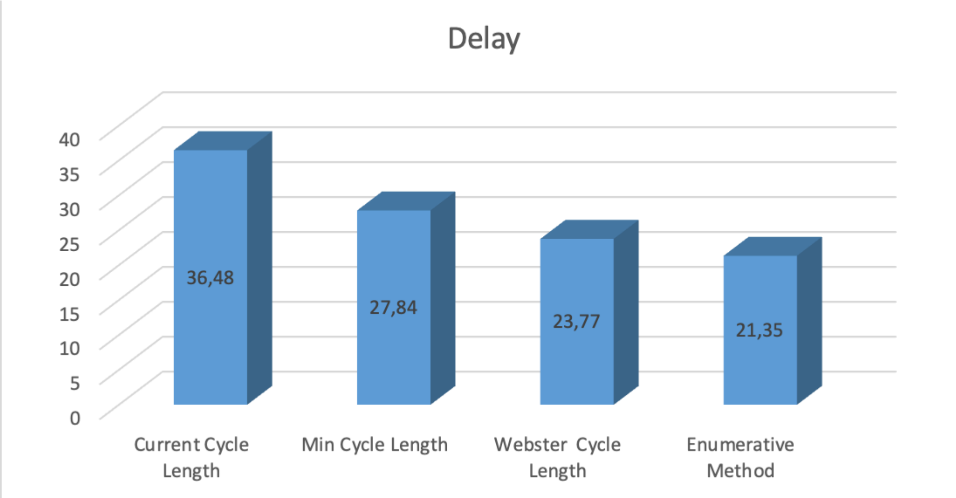
At thethirditeration, we found that the minimum delay is 23.35 c with green ratio (g1/g2) 1.067.

After this iteration we realized that the result didn't get better so we have

	Cycle	g1	g2	g1/g2	d	LOS
Enumerative Method	75.00	32.52	30.48	1.07	21,35	C

Finally for the first junction we have

	Cycle	g1	g2	g1/g2	d	LOS
Current Cycle Length	108.00	57.00	39.00	1.46	36.48	D
Min Cycle Length	35.21	13.80	13.91	0.99	27.84	C
Webster Cycle Length	76.29	34.26	34.53	0.99	23.77	C
Enumerative Method	75.00	32.52	30.48	1.07	21,35	C



6.2. The second intersection is Viale Ronchi - Largo Irpinia - via Prenestina:

6.2.1. Current Cycle for 2nd intersection:

	C	g1	g2	g3	d	LOS
Current Cycle Length	132	43	33	38	31	C

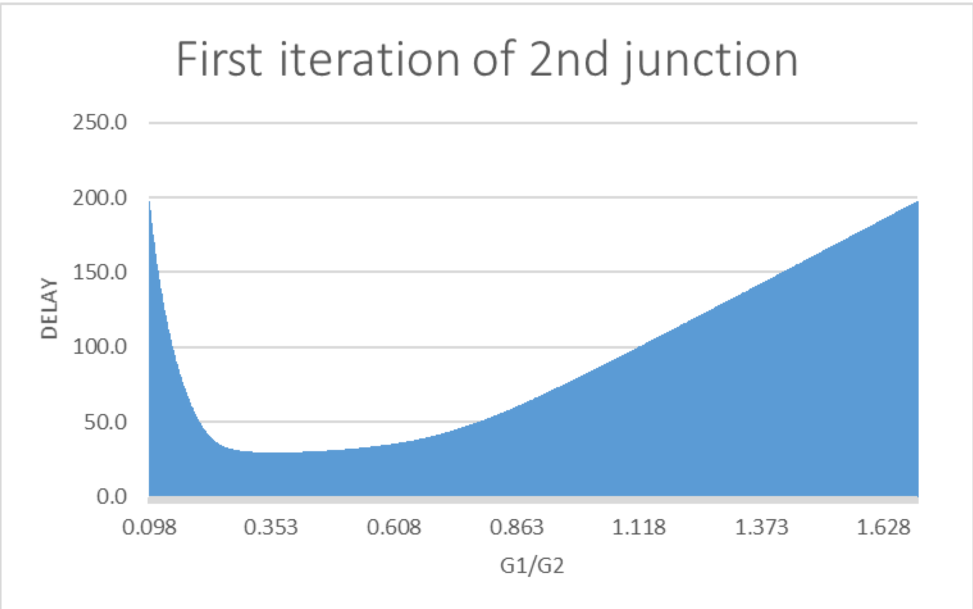
6.2.2. Minimum Cycle for 2nd intersection:

	C	g1	g2	g3	d	LOS
Min Cycle Length	41.41	7.00	21.66	8.75	15.96	B

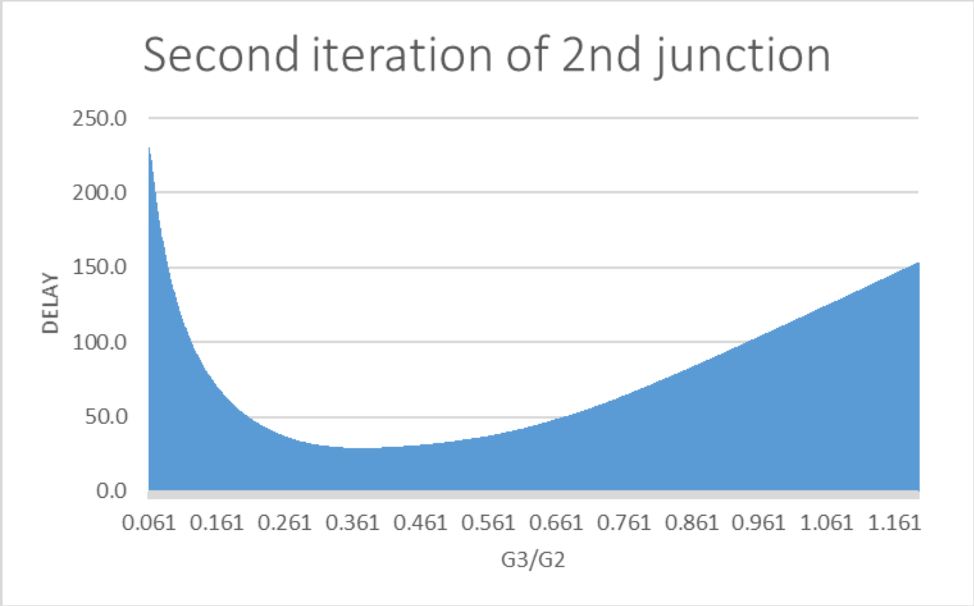
6.2.3. Webster Optimum Cycle for 2nd Intersection:

	C	g1	g2	g3	d	LOS
Webster Cycle Length	80.94	13.68	42.33	17.11	24.55	C

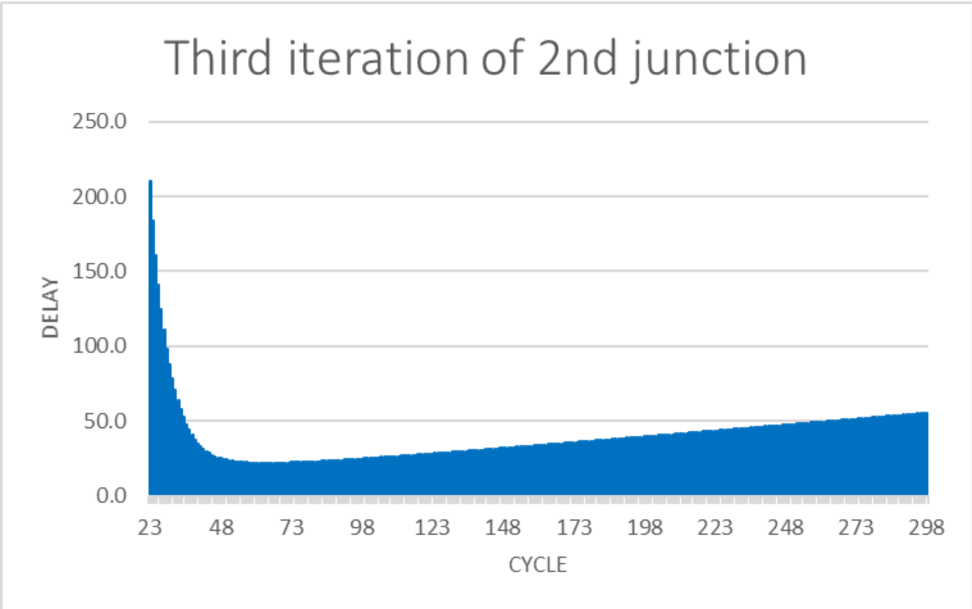
To find the minimum of the delay function, at beginning the cycle has been fixed to 132 sec, making the green ratio varies.



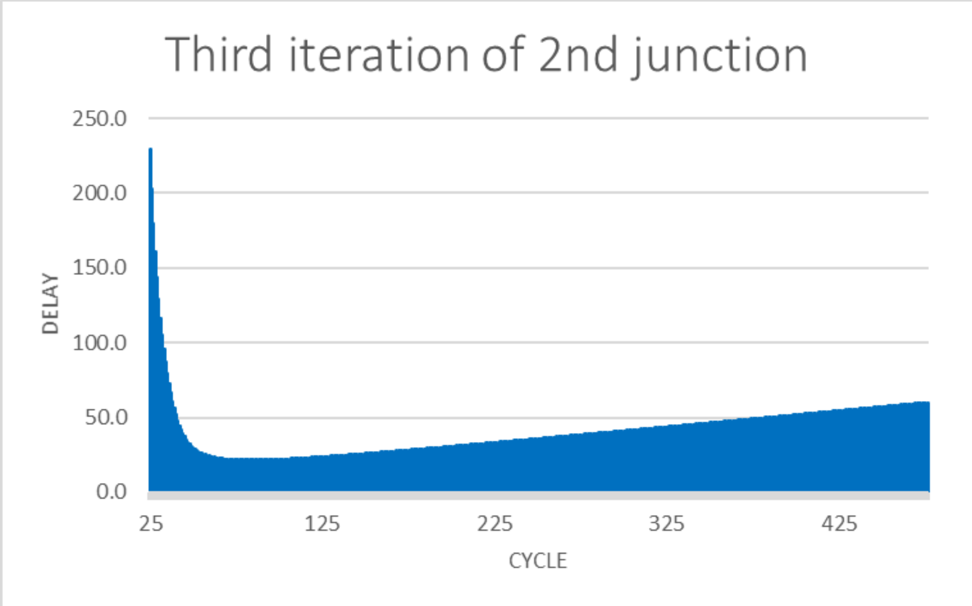
At the first iteration, we found that the minimum delay is 29.5913 sec with green ratio (g1/g2) 0.357. In the second iteration the green ratio (g1/g2) has been fixed to 0.357, making the g3/g2 varies.



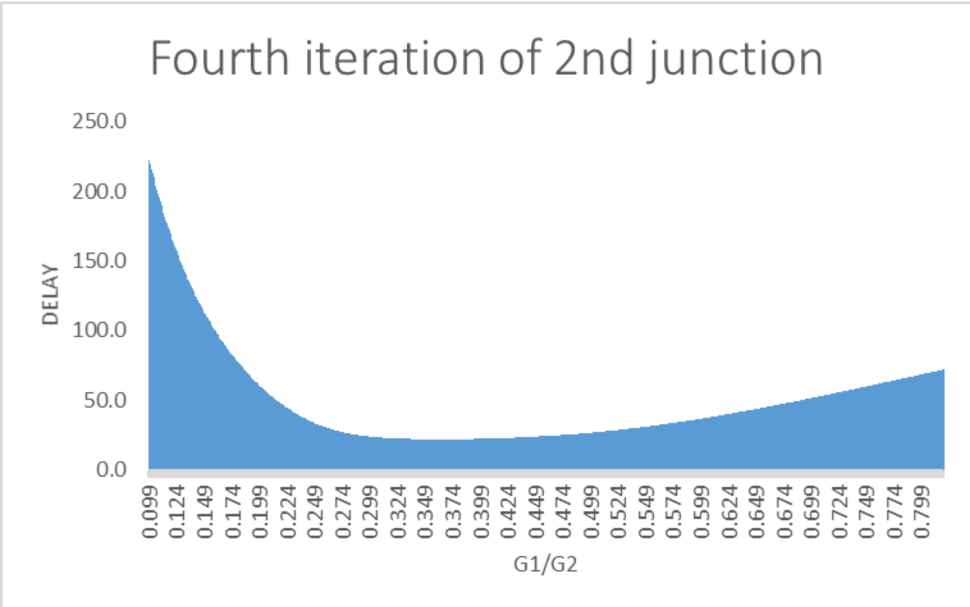
At the second iteration, the minimum delay is found 29.1581 sec with g3/g2 0.373. In the third iteration the green ratio (g1/g2) has been fixed to 0.357, also the g3/g2 0.373, and making the cycle varies.



At the third iteration, the minimum delay is found 22.043 sec with cycle 64 sec. In the third iteration the green ratio (g1/g2) has been fixed to 0.357, also the g3/g2 0.373, making the cycle varies.

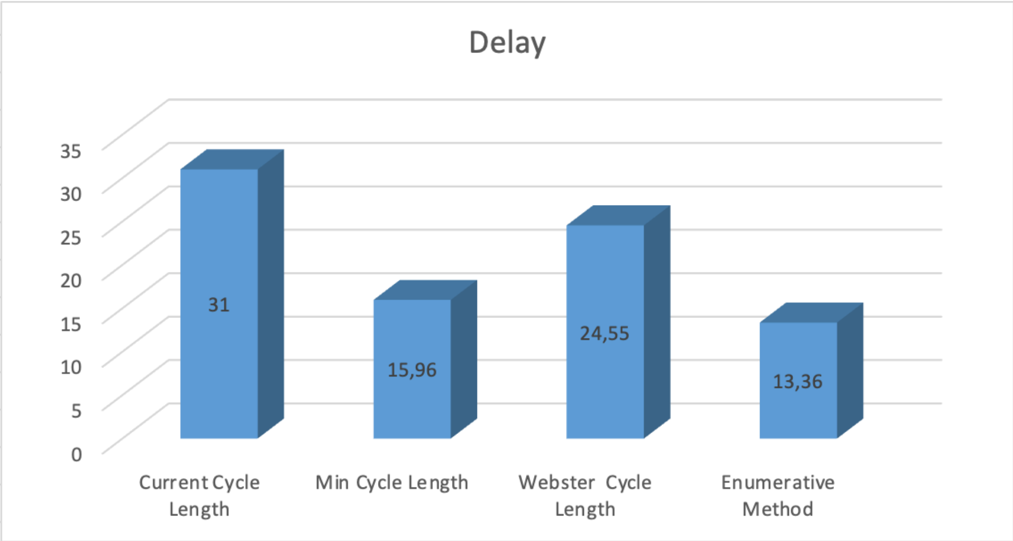


At the fourth iteration, the minimum delay is found 22.0358 sec and (g1/g2) is equal to 0.362 and (g3/g2) is 0.395.



We performed the method until reached to the best solution in this case after four times we have below result

	C	g1	g2	g3	g1/g2	g3/g2	d	LOS
Current Cycle Length	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	C
Min Cycle Length	41.41	7.00	21.66	8.75	0.32	0.40	15.96	B
Webster Cycle Length	80.94	13.68	42.33	17.11	0.32	0.40	24.55	C
Enumerative Method	80.00	13.82	38.18	19.25	0.36	0.50	13.36	C



6.3 The third intersection is Via Prenestina – Via Dignano Distria

6.3.1 Current Cycle for 3rd intersection:

	Cycle	g1	g2	g1/g2	d	LOS
Current Cycle Length	132	76	44	1.73	37.3	D

6.3.2 Minimum Cycle for 3rd intersection:

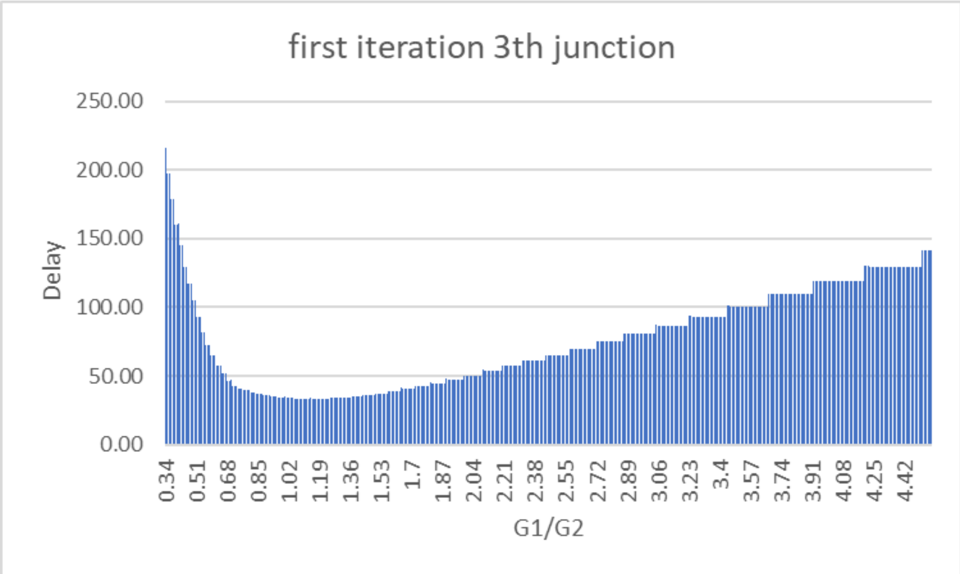
	Cycle	g1	g2	g1/g2	d	LOS
Min Cycle Length	37.5	13.5	14.6	0.924658	33.55	C

6.3.3 Webster Optimum Cycle for 3rd Intersection:

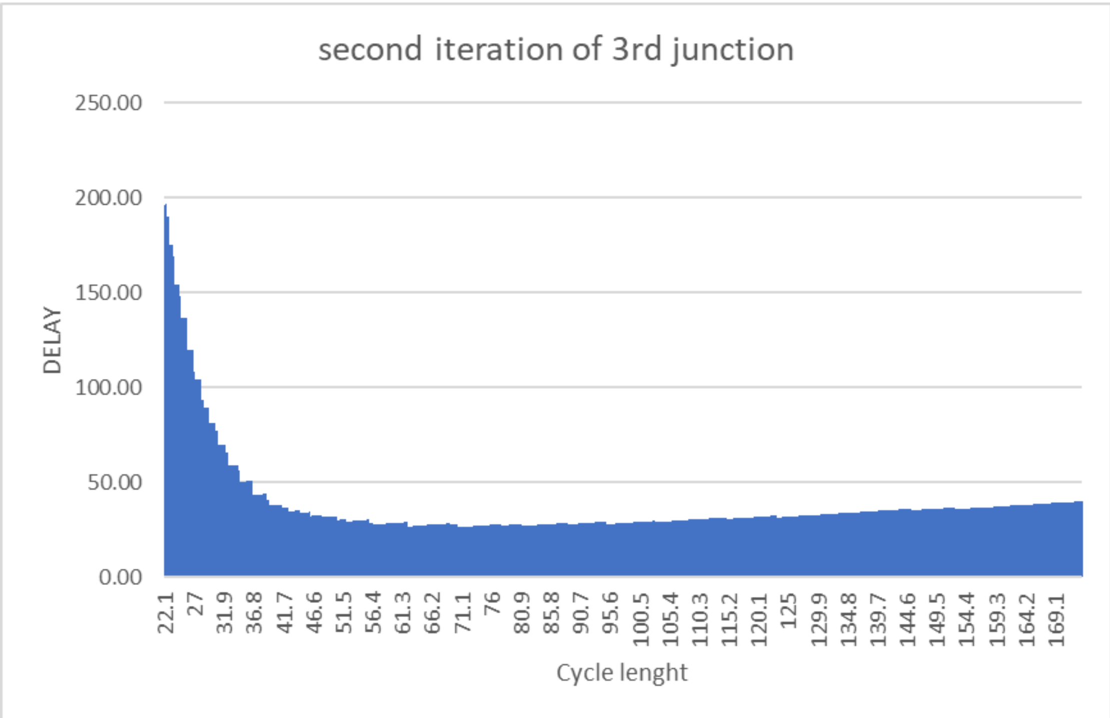
	Cycle	g1	g2	g1/g2	d	LOS
Webster Cycle Length	76.3	32	34.9	0.916905	21.03	C

6.3.4 Enumerative method for the 3rd intersection:

To find the minimum of the delay function, at beginning the cycle has been fixed to 132 sec, making the green ratio varies.

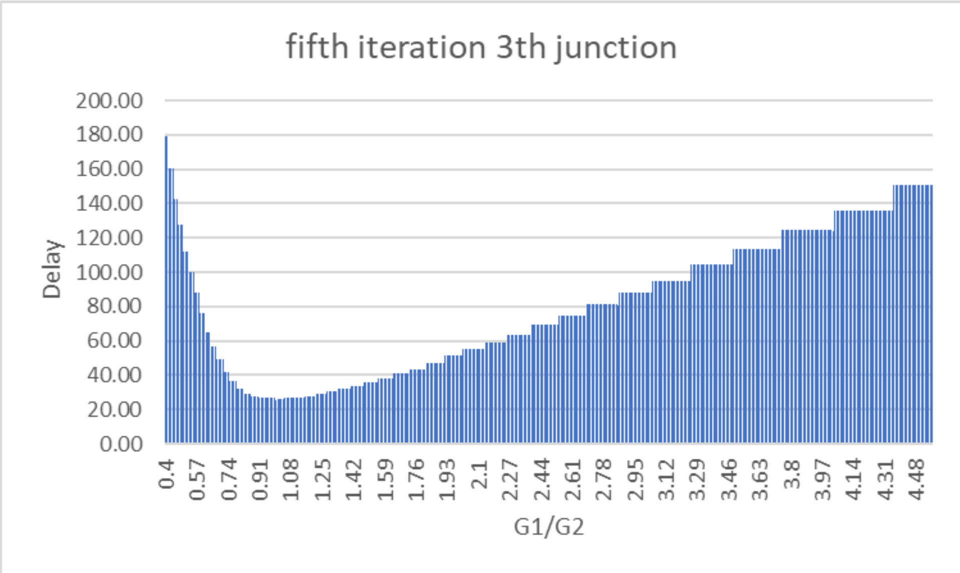


At the first iteration, we found that the minimum delay is 33.17 sec with green ratio (g1/g2) 1.16. In the second iteration the green ratio has been fixed to 1.16, making the cycle time varies



At the second iteration, the minimum delay is found 26.78 sec with cycle time 63.2 sec.

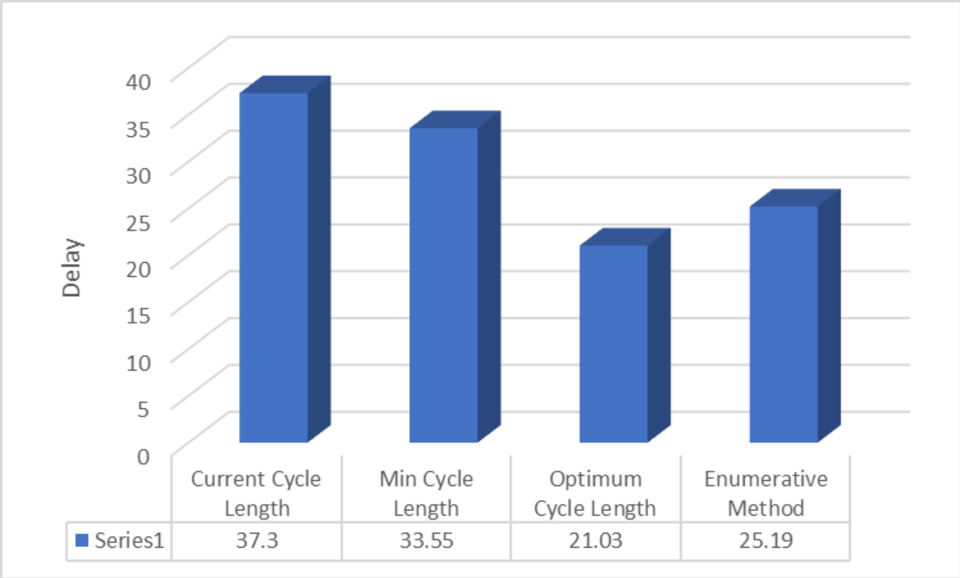
We repeated this procedure until the fifth iteration with the delay of 25.19 sec and green ratio (g1/g2) equal to 1.



	Cycle	g1	g2	g1/g2	d	LOS
Enumerative Method	70.6	29.3	29.3	1	25.19	C

Finally for the third junction we have

	Cycle	g1	g2	g1/g2	d	LOS
Current Cycle Length	132	76	44	1.727273	37.3	D
Min Cycle Length	37.5	13.5	14.6	0.924658	33.55	C
Webster Cycle Length	76.3	32	34.9	0.916905	21.03	C
Enumerative Method	70.6	29.3	29.3	1	25.19	C



6.4. The fourth intersection is Via Prenestina – Via Olevano Romano:

6.4.1. Current Cycle for 4th intersection:

	Cycle	g1	g2	g1/g2	d	LOS
Current Cycle Length	132	75	45	1.666667	17.1	B

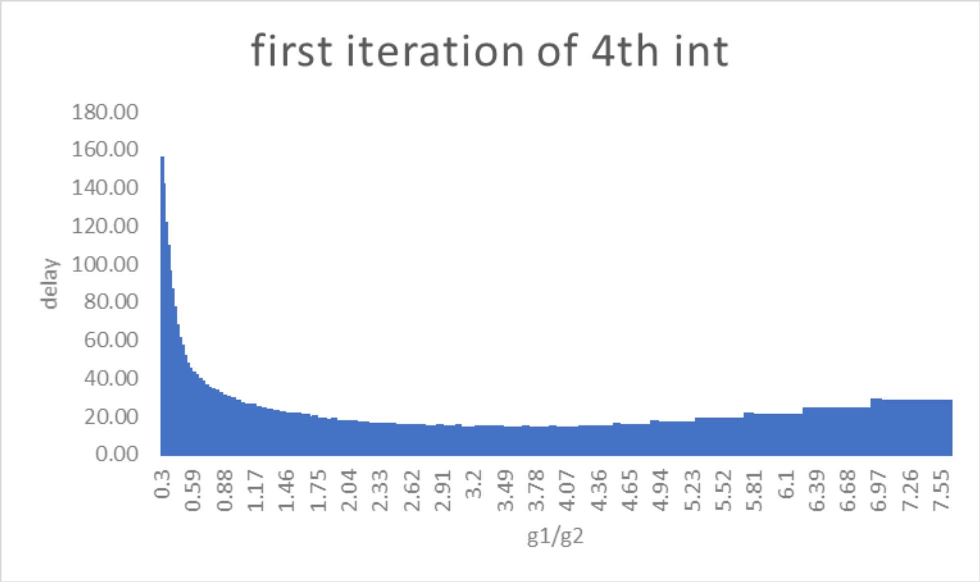
6.4.2. Minimum Cycle for 2nd intersection:

	Cycle	g1	g2	g1/g2	d	LOS
Min Cycle Length	16.7	4.8	2.4	2	26	C

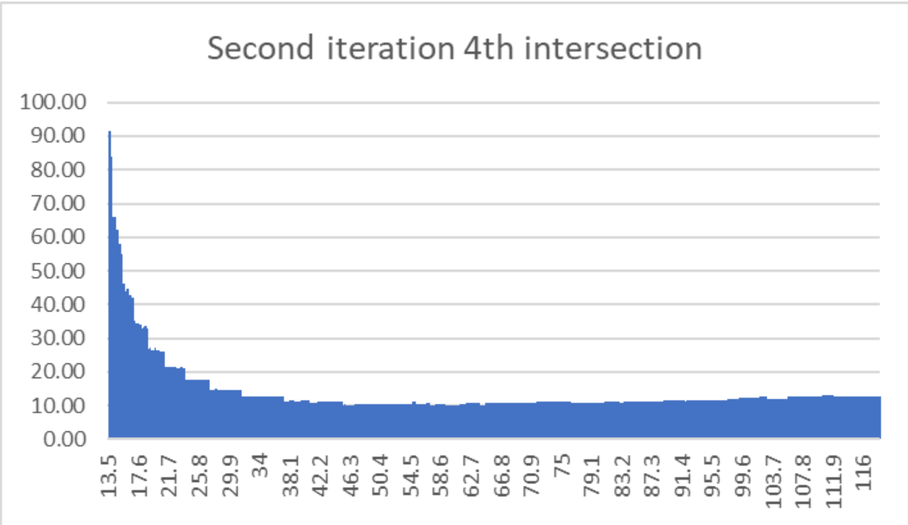
6.2.3. Webster Optimum Cycle for 2nd Intersection:

	Cycle	g1	g2	g1/g2	d	LOS
Webster Cycle Length	33.9	16.3	8.2	1.987805	8.3	A

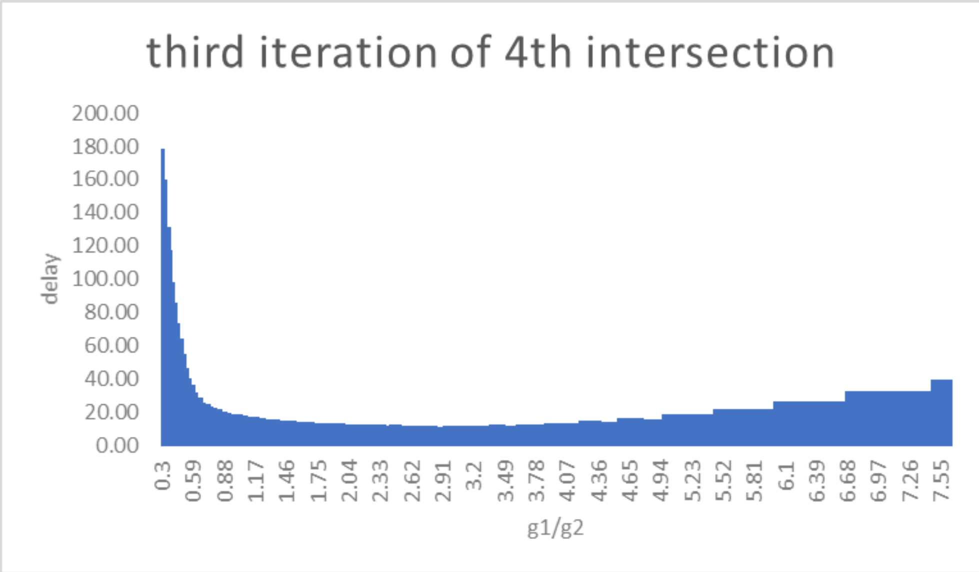
To find the minimum of the delay function, at beginning the cycle has been fixed to 132 sec, making the green ratio varies.



At the first iteration, we found that the minimum delay is 13.33 sec with green ratio (g1/g2) 3.6. In the second iteration the green ratio (g1/g2) has been fixed to 3.6, making the g3/g2 varies.



At the second iteration, the minimum delay is found 9.92 sec with cycle time 61.6 sec.

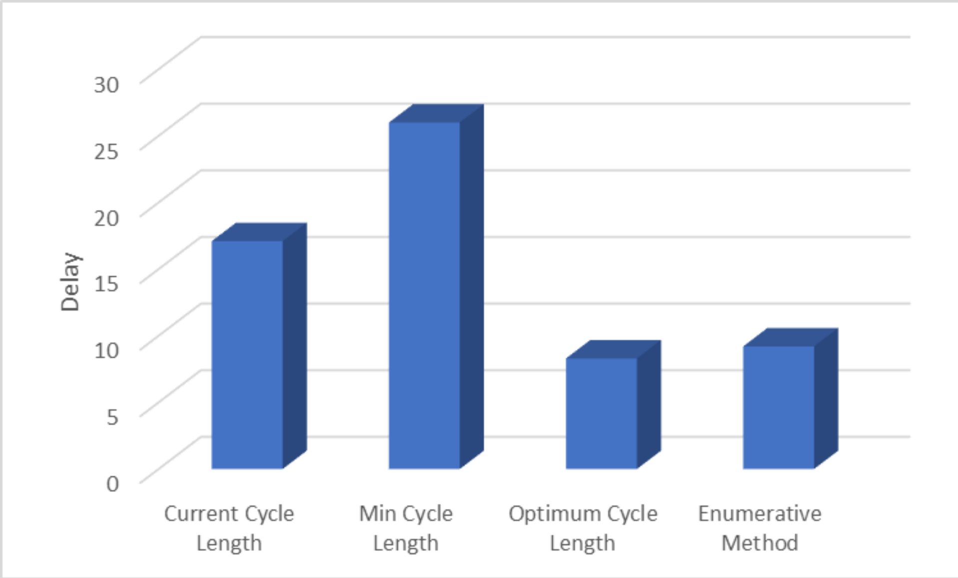


At the third iteration, we found that the minimum delay is 9.92 sec with green ratio (g1/g2) 3.52.

	Cycle	g1	g2	g1/g2	d	LOS
Enumerative Method	61.6	38.6	11	3.52	9.2	A

Finally for the first junction we have

	Cycle	g1	g2	g1/g2	d	LOS
Current Cycle Length	132	75	45	1.666667	17.1	B
Min Cycle Length	16.7	4.8	2.4	2	26	C
Webster Cycle Length	33.9	16.3	8.2	1.987805	8.3	A
Enumerative Method	61.6	38.6	11	3.52	9.2	A



Delay
30
25
20
15
10
5
0
Current Cycle Length
Min Cycle Length
Optimum Cycle Length
Enumerative Method

7. 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 the starting time of the green signal at intersections, is to allow vehicles arriving at the downstream intersection during the green phase, with a reduced 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 (ϑ_{ij}) 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:
ϑij= 0 if $x\ j\ -x\ i\ 2A\ =m$ for m=0,1,2, ...
ϑij = C 2 if $x\ j\ -x\ i\ 2A$ = $2m$ 12 for m=0,1, 2…
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:

$$max\ f = (b+b')$$

$$b = b'>0$$

$$\theta_{ij} = 0 \text{ or } ½$$

$$v_{min} \leq v \leq v_{max}$$

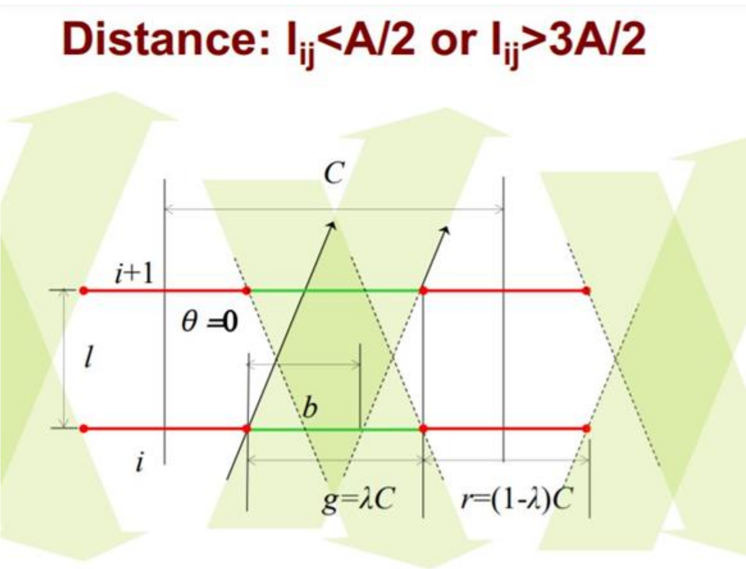
$$max\ \{C_{min,\ i}\} \leq C \leq C_{max}$$

$$C_{min,i} = \frac{L_i}{(1 - max_h\{y_{i,h}\} - max_k\{y_{i,k}\})}$$

$$max_h\{y_{i,h}\} \leq g_i \leq 1 - \frac{L_i}{C} - max_k\{y_{i,k}\}$$

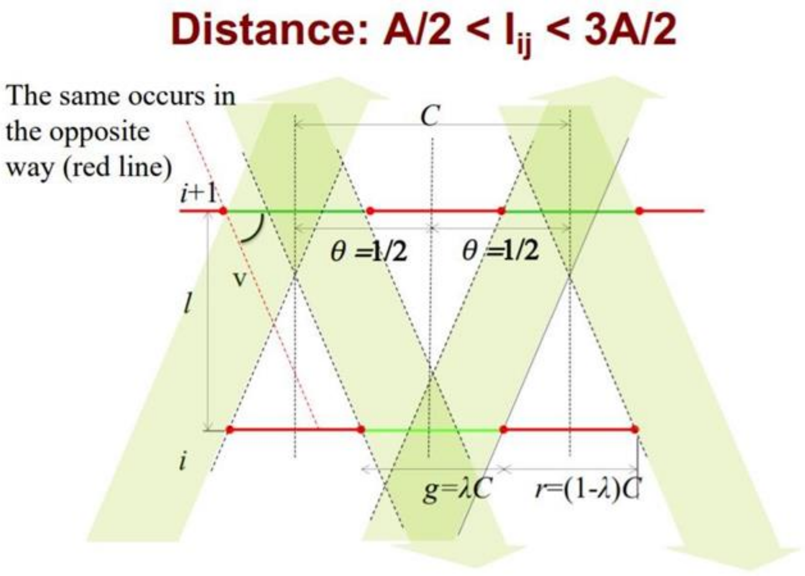
Where:
Li= lost time of node i
yi, h = saturation degree of approach h yi, k = saturation degree of approach k
b=b' = bandwidth inbound and outbound gi = green at artery approach i

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 equisaturation 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.



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

Junctions in phase



$$\vartheta_{ij} = 1/2$$

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

Junctions in phase opposition

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:

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

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

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:

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

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

7.1. Study Case

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 cycle lengths, for this reason the 80 [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 10 m/s (36km/h), which feasible for urban area. In table 2 are shown the Progressive distances of the junctions and in Table 3 the distances between two consecutive junctions.

Intersection	1	2	3	4
Cycle Length	75	80	70.6	61.6
Delay	28.85	25.52	25.19	9.2
g1	32.52	13.82	29.3	38.6
g2	30.48	38.18	29.3	11
g3	-	19.25	-	-
Cycle Length			80	
Delay	28.96	25.52	25.84	10.38
g1	35.102	52	34	52.4
g2	32.9	19.25	34	15.6

Table 35: Synchronization data the delay with respect to the scenario with cycle of 81 s for all intersection

intersection	Distance (m)	Progressive distance(m)	g1	g1/C
1	0	0	35.102	0.439
2	230	230	52	0.65
3	110	340	34	0.425
4	376	716	52.4	0.655

Considering the mentioned assumptions, the Synchronization module (A) is calculated as follows:

V = 10 m/s which is equal to 36 km/h

A = (C*V)/2 = (80*10)/2 = 400

Node		Distance (m)	Progressive distance(m)	lij (m)	A/2 (m)	offset θ	Bandwidth	Absissa of ideal node (m)
1	2	230	230	230	200	0.5	0.46	-43
2	3	110	340	110	200	0	0.33	330
3	4	376	716	376	200	0.5	0.41	-
			The final result	**26.5**				

The maximum bandwidth is 0.33. Therefore, by multiplying bandwidth with cycle (80 [s]) we can obtain the green time which it is 26.5[s]. In this green time each car moving from Largo Telese is faced with green at the last junction.

8. Conclusion

At this study, four intersections in Via Prenestina Artery have deliberated to minimize the delay as objective function. After applying some methods to find an optimal solution for isolated signal setting and some improvement happened in term of LOS.

At the end of this study, coordinated signals has been considered through synchronization. For determining the input of the synchronization, results of Enumerative for isolated intersections is applied. Finally, 26.5 second has been calculated as a maximum bandwidth for this road artery.