MINOR STREET GAPS AND CAPACITY AT UNSIGNALIZED INTERSECTIONS IN ARGENTINA

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ABSTRACT

Capacity analysis for two way stop controlled (TWSC) intersections is mainly based on headway acceptance theory. Critical headways and follow up times are fundamental parameters in capacity estimation. Both factors clearly show the influence of driver behaviour on traffic operations. The critical headway parameter is typically associated with safety and operational performance of this intersection type.

This paper explores critical headways and follow up times by analyzing data from video recorded tapes collected in urban intersections located in the city of Cordoba, Argentina in order to derive local values that can be used in capacity estimates at unsignalized intersections. Maximum likelihood methodology and regression analysis are employed. An exponential model is then used to assess the relationships between headways and capacity. Estimates for both headways are significantly smaller than the values given in version 2010 of the Highway Capacity Manual. Increased capacity, due to critical headway and follow up time reductions, becomes proportionally greater as conflicting flows grow. Percent differences increase while curves tend to get closer. From this point of view the conclusion is that intersections operate more efficiently, but also more dangerously.

Keywords: unsignalized, critical headways, capacity

1. INTRODUCTION

The most common intersection found in urban or rural locations is the unsignalized type. Unsignalized intersections work efficiently under moderate traffic flows and if traffic to and from the minor street is low, the intersection works quite well regardless of the major street's volumes. Still, analyzing its operations remains very important. Another case is when conflicting movements have reasonable volumes, then unsignalized intersections become inefficient and tend to cause great delays on non-priority approaches and consequently on the entire network. In these circumstances signalization becomes necessary (MUTCD, 2009)

At intersections where a major street and a minor street are involved, - two way stop controlled intersection (TWSC) -, only drivers on minor street approaches are required to stop before proceeding into the intersection. In this way delays at the intersection are only experienced by minor stream vehicles whilst major stream vehicles suffer no delays. Drivers arriving from a non-priority approach are forced to come to a full stop, evaluate the headways on the major stream, and finally enter the intersection when there is a headway large enough between two successive vehicles so as to execute the desired manoeuvre safely without interfering with the traffic stream on the mainline. Therefore, drivers on the minor approach must decide which headway allows for a safe entry while also conforming to the right of way hierarchy. In fact, drivers on minor approaches have shown a tendency to accept a headway when "the benefit from entry is greater than the associated risk" (Pollatschek et al., 2002).This decision making process is commonly known as headway acceptance and it depends on three basic factors:

1. Available headways between vehicles on the major stream of a particular size and arrival pattern,

2. Headways usefulness and the extent to which drivers find headways of a particular size useful to perform their intended movement; and

3. Relative priority of movements at the intersection

Minor street drivers are assumed to be waiting for an acceptable headway before moving. The minimum accepted headway for a particular driver is called the critical headway. A particular driver would reject any headway that is smaller than the critical one and accept greater headways. Assessing its value from field observations is impossible but one could say that it lies between the largest rejected headway and the accepted headway
.
A particular driver population is usually represented by a mean critical headway that is used in capacity and delay estimation Several methods for estimating critical headways can be found. In the literature. The maximum likelihood method (Troutbeck, 1992; Tian at al., 1999) has proven to be quite reliable. For this reason it was used to determine critical headways in the Kyte's project (Kyte et al, 1996), main research reference for the 2010 Highway Capacity Manual (TRB, 2010).

Chapter 19 of the 2010 update to the Highway Capacity Manual, includes a methodology for capacity analysis of two-way stop controlled intersections which fits with the operation of intersections of main streets with minor streets in Argentina. This Chapter also includes a set of critical headway and follow-up time values calibrated for the United States' local conditions such as vehicle types, traffic operations and driver behaviour characteristics.

For many years research has been going on concerning local driver behaviour so as to represent local conditions more accurately (Galarraga et al.,2002) (Galarraga et al., 2005) Critical and follow up headways accepted in minor local street movements are fairly larger than those reported by the HCM 2010. An increase in capacity takes place in the minor street due to local estimates of critical and follow up headways. This may affect the upper bound limit in traffic flow at minor streets where a change in intersection operation strategy would be needed (Depiante, 2011).

2. AVAILABLE METHODOLOGIES

2.1. HCM2010 capacity analysis description

The potential capacity of a movement is computed according to the gap acceptance model provided in Equation 1. This model requires the analyst to input the conflicting flow rate, the critical headway and the follow up headway for the movement. It assumes a negative exponential function for the distribution of major street headways

$$c_{p,x} = v_{c,x} \frac{e^{-\frac{v_{c,x} t_{c,x}}{3600}}}{1 - e^{-\frac{v_{c,x} t_{f,x}}{3600}}} \tag{1}$$

where:
$c_{p,x}$ = potential capacity of movement "*x*" (vph),
$v_{c,x}$ = conflicting flow rate for movement "*x*" (vph),
$t_{c,x}$ = critical headway for minor movement "*x*" (s),
$t_{f,x}$ = follow-up headway for minor street movement "*x*" (s).

The potential capacity is defined for a given type of movement from the minor street assuming the following basic conditions:

1. Adjacent intersection traffic does not affect the analysed intersection.
2. Every movement in the minor street has its own lane, there are no shared lanes
3. The arrival pattern from the main road is not affected by the presence of close signalized intersections
4. No other movement of higher priority prevents the analysed movement.

Headways above the critical can be used by more than one vehicle from the minor street. The exponential function of the numerator indicates the probability of finding a headway exceeding the critical headway.

The methodology also makes specific adjustments to take into account the effect of heavy vehicles, the grade encountered and the presence of a three-leg intersection as shown in Equations 2 and 3.

$$t_{c,x} = t_{c,base} + t_{c,HV} PHV + t_{c,G} G - t_{3,LT} \tag{2}$$

$$t_{f,x} = t_{f,base} + t_{f,HV} PHV \tag{3}$$

where:

$t_{c,x}$: critical headway for movement "*x*" (s),

$t_{f,x}$: follow-up headway for movement "x" (s),
$t_{c,base}$: base critical headway for movement "*x*" (s),
$t_{f,base}$: base follow-up headway (s),
$t_{c,HV}$: adjustment factor for heavy vehicles (1.0 for major streets with one lane in each direction, 2.0 for major streets with two or three lanes in each direction) (s),
$t_{f,HV}$: adjustment factor for heavy vehicles (0.9 for major streets with one lane in each direction, 1.0 for major streets with two or three lanes in each direction) (s),
PHV: proportion of heavy vehicles for movement (expressed as a decimal)
$t_{c,G}$: adjustment factor for grade (0.1 for movements 9 and 12; 0.2 for movements 7, 8, 10 y 11) (s),
G : grade percent (%),
$t_{3,LT}$: adjustment factor for intersection geometry (0.7 for minor street left turn movement at three-leg intersections, "0" otherwise) (s).

2.2. Critical gap and follow-up time joint estimation procedure

There are two basic groups of methodologies for estimating critical headway and follow-up time, either jointly or independently. Some (Siegloch, 1973) determine both headways simultaneously using regression techniques while others (Troutbeck, 1992) use a probabilistic process such as maximum likelihood and calculate the expected mean value of the critical headway. The follow-up time is determined separately as the minor street headways' mean of vehicles in queue entering the conflict flow during the same accepted headway.

The Siegloch´s method for estimating critical and follow up headways jointly is simple and quite reliable but only applicable under saturation flow conditions, that is to say continuous queue on the minor street. Based on regression analysis of the number of vehicles using a specific headway size versus headway size, both the critical headway and the follow-up time can be determined. The procedure includes the following steps: (1) Register for each interval size "t", the number of vehicles "i" entering that headway, (2) calculate the average headway size E (t), for each of the headways accepted by only "n" vehicles, (3) adjust a linear regression between gap size E (t) average values (as dependent variable) and the number of vehicles entering during this average interval size, "n "(independent variable), (4) the regression line's slope is the estimated value for the follow-up time (t_f), accounting for the time added when passing from vehicle "i" to "i +1", (5) the critical acceptance headway is calculated as the intercept (t_o) plus half of the follow-up time, since no vehicle enters for smaller values. The regression method cannot be implemented without a continuous queue. In such case a probabilistic method is applicable.

2.3. Critical headway and follow-up time independent estimation procedure

When the minor street shows undersaturated conditions, which is the most frequent case, linear regression cannot be applied because mainstream intervals are not fully used. As a result, other methods must be employed. In order to estimate follow-up time a simple average of minor street headways should be used. This procedure is analogous with that used for saturation flow rate calculation at signalized intersections (TRB, 2010).

For critical headway estimation, the maximum likelihood methodology has shown to be accurate. In this case a probabilistic distribution function is adjusted for the accepted headway and maximum rejected headway of individual drivers. Troutbeck (1992) describes this procedure. It is clearly stated that the value of the critical headway lies between the maximum rejected headway and the actually accepted headway. If the accepted value were smaller than the maximum rejected headway, then the driver is not focused and the rejected value must be discarded. Due to procedure requirements, a probabilistic distribution must be assumed for driver population critical headways. In most cases a log-normal distribution is reasonable and has been adopted in many studies due to its right-sided skewness and lack of negative values.

Considering the following notation:

a_i = logarithm of the accepted headway by the i^{th} driver
r_i = logarithm of the maximum rejected headway by the i^{th} driver
μ y σ^2 = individual driver's critical headways logarithmic distribution mean and variance (considering a log-normal distribution); and
f( ) y F( ) = probability density function and cumulative function for the normal distribution

The probability that a critical headway of a driver lies between r_i y a_i is $F(a_i) - F(r_i)$. For n drivers the likelihood for accepted headways and maximum rejected headways (a_i, r_i) is given by Equation 4

$$\prod_{i=1}^{i=n}\left[F(a_i) - F(r_i)\right] \tag{4}$$

and the logarithm of that probability is:

$$L = \sum_{i=1}^{i=n} \ln\left[F(a_i) - F(r_i)\right] \tag{5}$$

The maximum likelihood estimators μ and σ^2 that maximize L, making its partial derivatives null with respect to μ and σ^2 ($\delta L/\delta\mu = 0$ and $\delta L/\delta\sigma^2 = 0$), are the solutions to Equations 6 and 7.

$$\frac{\partial L}{\partial \mu} = \sum_{i=1}^{i=n} \frac{\frac{\partial F(a_i)}{\partial \mu} - \frac{\partial F(r_i)}{\partial \mu}}{F(a_i) - F(r_i)} = 0 \tag{6}$$

$$\frac{\partial L}{\partial \sigma^2} = \sum_{i=1}^{i=n} \frac{\frac{\partial F(a_i)}{\partial \sigma^2} - \frac{\partial F(r_i)}{\partial \sigma^2}}{F(a_i) - F(r_i)} = 0 \tag{7}$$

For the selected distribution it can be proven that:

$$\frac{\partial F(x)}{\partial \mu} = -f(x) \tag{8}$$

$$\frac{\partial F(x)}{\partial \sigma^2} = -\frac{x-\mu}{2\sigma^2} f(x) \tag{9}$$

Replacing the former, Equations 10 and 11 are derived. These equations must be solved simultaneously. Parameter μ^* is an estimate of μ, while f(x) and F(x) are functions of the mean and the variance estimators.

$$\sum_{i=1}^{i=n} \frac{f(r_i) - f(a_i)}{F(a_i) - F(r_i)} = 0 \tag{10}$$

$$\sum_{i=1}^{i=n} \frac{(r_i - \mu^*) f(r_i) - (a_i - \mu^*) f(a_i)}{F(a_i) - F(r_i)} = 0 \tag{11}$$

Thus the mean $E(t_c)$ and the variance $Var(t_c)$ of the critical headway distribution are given by the parameters of the log-normal distribution described by Equations 12 and 13 respectively.

$$E(t_c) = e^{[\mu + 050\sigma^2]} \tag{12}$$

$$Var(t_c) = E(t_c)^2 e^{[\sigma^2]} \tag{13}$$

The value of the critical headway $E(t_c)$ of driver population encountered is the one used in gap acceptance methodologies. This value should be smaller than the accepted headways mean

2.4. Capacity models based on headway acceptance theory

Two types of models are used when analyzing capacity at unsignalized intersections. Since this type of intersection gives no indication of when to cross, the minor street driver must decide when it is safe to enter one of the mainstream's headways. This process is the basis of the acceptance model.

All headway capacity models for TWSC intersections are derived from a simplified queuing model where only two crossing traffic streams are considered (Luttinen, 2003); with priority traffic given by the major street volume and non priority stream given by the minor street volume. Vehicles on the major stream suffer no delay while crossing the intersection while vehicles on the minor stream experience delay if the headway for the major street is less than t_c seconds.

The mathematical derivative of capacity c_n for the minor street is as follows: let g(t) be the number of minor street vehicles that can enter a major street headway of t seconds. The expected number of these t duration headways in an hour is $3600.v_p.f(t)$ where f(t) is the major headway probability density function, and v_p is the volume on the major street in vehicles per second. Therefore, capacity provided by headways of t duration is $3600.v_p.f(t).g(t)$ per hour. In order to obtain total capacity expressed in vehicles per second it must be integrated over the whole range of major street headways t as shown in Equation 14.

$$c_n = v_p \int_0^t f(t) g(t) dt \quad (14)$$

where:
c_n : maximum traffic volume that can depart from the stop line in the minor stream in vps units.
v_p : major stream volume in vps units.
f(t) : density function of major street headways
g(t) : minor street number of vehicles that can enter a major headway of size t.

It is accepted that a negative exponential distribution function can be used for major street headways in the acceptance model. Moreover, capacity in a simple case of two traffic streams can be evaluated using elemental probability theory models under the following assumptions:

- Constant t_c y t_f values (homogeneus and consistent driver population),
- Negative exponential distribution for priority stream headways
- Constant traffic flows for each traffic stream

There are two different formulations for the term g(t) that derive in two different families of capacity equations. The first one assumes a stepwise constant function while the other one adopts a continuous linear function for g(t).

Harder's capacity formulation can be obtained solving the integral of Equation 14, considering the negative exponential distribution for f(t) given in Equation 15 and a constant stepwise function for g(t) given in Equation 16, deriving in Equation 17.

$$f(t) = \lambda e^{-\lambda t} \quad (15)$$

$$g(t) = \sum_{n=0}^{\infty} n P_n(t) \quad (16)$$

where:
λ : major street volume in (vps)

$P_n(t)$: probability that n minor stream vehicles enter a major stream headway of t duration in seconds

$$P_n(t) = \begin{cases} 1, & t_c + (n-1)t_f < t < t_c + nt_f \\ 0, & for\ other\ t\ values \end{cases}$$

$$C_n = v_p \frac{e^{-v_p t_c}}{1 - e^{-v_p t_f}} \tag{17}$$

Considering a negative exponential distribution for f(t) and a linear function for g(t) derives in Siegloch's capacity formulation (Siegloch, 1973) from Equation 18.

$$C_n = \frac{1}{tf} e^{-v_p t_o} \tag{18}$$

where:

$$t_0 = t_c - \frac{t_f}{2}$$

Both approaches for g(t) produce useful capacity formulae were the resulting differences are small and are usually ignored for practical applications (Kyte et al., 1996).

2.5. Capacity model based on regression analysis

Empirical capacity models are based on regression techniques (NCHRP, 2007). The model establishes a regression between major street volumes and minor stream entry volumes. To derive the relationship, observations of traffic operations at the intersection need to be made during periods of oversaturation on the minor street approach. The total observation time must be divided into periods of constant duration (e.g., one minute). During these one minute intervals, both priority volumes and minor street volumes are counted and converted to hourly volumes. Normally these data points are scattered over a wide range. Then an exponential regression, Equation 19, can be fit to the data (Kimber, 1989) (Kyte, 1996). It is essential that at least one vehicle remains in queue during each interval.

$$C_{p,x} = Ae^{-Bv_{c,x}} \tag{19}$$

Equation 19 establishes a functional relationship between capacity and conflicting flow when parameters A and B are previously calibrated.

Harder's model adopted in the HCM can be modified and derived into Siegloch's exponential regression model (Kyte et al, 1996) used in the new roundabout methodology (NCHRP, 2007) and incorporated in the HCM2010.

$$C_{p,x} = \frac{3600}{t_{f,x}} e^{-\left(\frac{t_{c,x} - t_{f,x}/2}{3600}\right) v_{c,x}} \tag{20}$$

With A and B parameter values presented in Equations 21 and 22 respectively.

$$A = \frac{3600}{t_{f,x}} \tag{21}$$

$$B = \frac{t_{c,x} - t_{f,x}/2}{3600} \tag{22}$$

Knowing parameter values A and B, It is possible to derive critical and follow up headways t_c and t_f from Equations 21 and 22

3. LOCAL STUDIES

3.1. Four leg intersection

Local studies were conducted at five intersections. Field measurements were performed registering important events such as the time when vehicles reached the stop line and major street times while crossing a virtual section until subject minor stream vehicle accepted a specific headway. The time the first main stream vehicle following the minor street subject vehicle left the stop line was registered as well. Some intersections were videotaped to be able to control critical and follow up headways estimation by using another methodology.

Collected data was analyzed using a worksheet in order to calculate rejected headways, accepted headway and follow-up time if it existed. Critical gap values cannot be obtained directly from raw data. To be able to use the maximum likelihood method for critical gap estimation, accepted and rejected gaps must be known and the maximum rejected gap selected among them. It is important to correctly define gap events that reflect driver's behavior at the intersection. The passage time of any vehicle that conflicts directly with the subject vehicle can be defined as a begin gap event. If it is a lag, the begin gap event is the arrival time of the subject vehicle at the stop line (first in queue time). An end gap/lag event is produced by the major street vehicle that conflicts with the subject vehicle. What actually determines a driver's decision whether to enter or not the intersection is the time left until the next major vehicle arrives at the intersection.

Unlike critical gap, follow-up time is measured directly from collected data. Follow up time can also be considered as the saturation headway of minor street vehicles. A follow-up time is observed only under the following conditions: a) The following vehicle has been queued, i.e. that when the vehicle arrives at the intersection there is already at least one vehicle waiting in front; and b) both vehicles (i.e., the lead vehicle and the following vehicle) use the same gap in the conflicting stream. When a following time is observed it is calculated based on the exit queue times of the two vehicles using the same headway.

Mean and variance for critical headways (calculated with the maximum likelihood estimator) and follow-up times are reported in Table 1 for intersections of two and four main stream lanes.

Table 1 - Mean and variance for critical and follow up headways

Minor street movement	Critical headway (s) Two lanes		Critical headway (s) Four lanes		Follow-up time (s)	
	Córdoba	Variance	Córdoba	Variance	Córdoba	Variance
Right Turn	4.2	7.7	5.1	9.9	2.3	2.2
Through	6.0	10.2	6.4	9.9	1.8	1.2
Left turn	6.7	13.2	6.9	15	3.2	1.3

Critical gaps and follow up times result in lower values than the HCM2010. Local drivers are more aggressive, therefore capacity estimates will be higher. On the other hand operation at the intersection will be more dangerous. The confidence interval of the mean critical headway and follow up time does not include HCM2010 values for the 90% confidence interval.

3.2. Three leg intersection

In this specific study, the case of a three leg stop controlled intersection sited in the urban area of Cordoba city was examined. The HCM requires the base critical gap to be adjusted when T-intersections are present. Only minor left turns are allowed at the intersection. It should be noted that a left turn maneuver from a minor stream is considered the most difficult maneuver at TWSC intersections (TRB, 2010). It is more complicated than the right turn maneuver as drivers have to find concurring gaps in the major traffic on both the near and the far side of the road, let alone the longer distance that needs to be traveled. Thus, left turn maneuvers are generally believed to require longer gaps than other movements.

A sample size of six periods between twenty and sixty minutes totalizing five videotaped hours was collected during the study. Around 300 minor street vehicles, exposed to over 2000 individual gaps, were recorded. Several characteristics were collected such as presence of a queue behind the lead vehicle, measures for the accepted and rejected gaps, follow-up times related to the minor street vehicle at the stop line and main stream vehicles' passing times. Further data reduction extracted traffic flow parameters such as mainstream flow rates and headways.

3.2.1. Critical headway and follow up time estimates based on regression methodology

According to Siegloch's methodology, critical headway and follow-up time estimates can be obtained jointly using regression techniques. A sample of 71 cases was analyzed for the left turn movement at the three leg intersection. Base conditions are met regarding passenger cars, no grade present, one lane per movement, no signals nearby, etc.

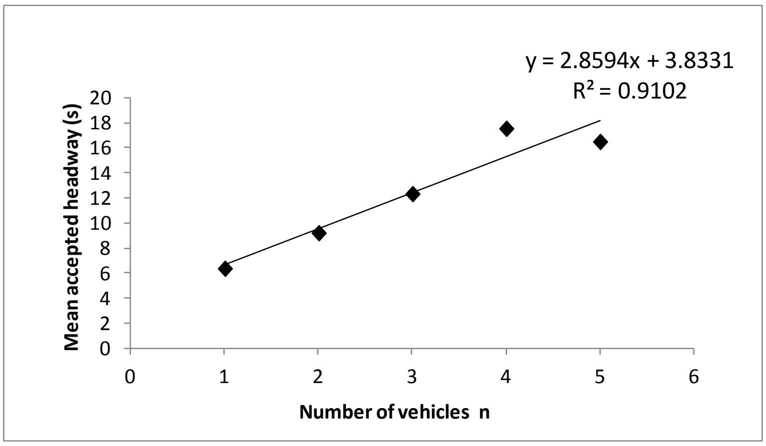

Figure 1 - Regression line for average accepted headway and number of vehicles

Figure 1 plots average accepted headway vs. number of minor stream vehicles entering during that headway on the major stream. The slope of the curve is the follow-up time (t_f): 2,8594s. The critical headway can be obtained as the intercept plus half of the follow-up time (t_c): 5,26s (3,8331s + 2,8594s/2).

3.2.2. Critical headways estimates based on maximum likelihood methodology

The maximum rejected gap, the accepted gap and the follow up time were measured for each period based on the procedures previously discussed. The measurements were collected from a total of six videotaped periods and analyzed separately. Only one period was discarded because base conditions on the main stream were not complied. According to the maximum likelihood method, the critical headway value for the three leg intersection was 4.77±1.35 s (n=308 cases). The observed follow-up time average was 2.80±0.86 s (n=225 cases). These values were adopted as local.

These values are statistically different from the ones proposed in the HCM2010. In no case HCM2010 values are included within the 95% confidence intervals (see Table 2).

Table 2 – Confidence interval for critical headways and follow-up times

	Córdoba			HCM2010
	Mean (s)	Inferior bound (95%)	Superior bound (95%)	Mean (included in the interval)
Critical headway	4.77	4.62	4.92	6.4 (no)
Follow-up time	2.8	2.69	2.91	3.5 (no)

3.2.3. Capacity estimates based on headway acceptance theory

Replacing headway values adopted in the capacity models previously presented, Equations 23 and 24 can be derived for Harder's model and Siegloch's model, adopted by the HCM2010 and by the NHCRP 572 project, respectively.

$$c_{p,x} = v_{c,x} \frac{e^{-0.001325 v_{c,x}}}{1 - e^{-0.000778 v_{c,x}}} \tag{23}$$

$$c_{p,x} = 1286 e^{-0.000936 v_{c,x}} \tag{24}$$

Both formulations arrive at similar results as can be seen in Figure 2, while considerably lower capacities are obtained with the HCM2010 which are also plotted.

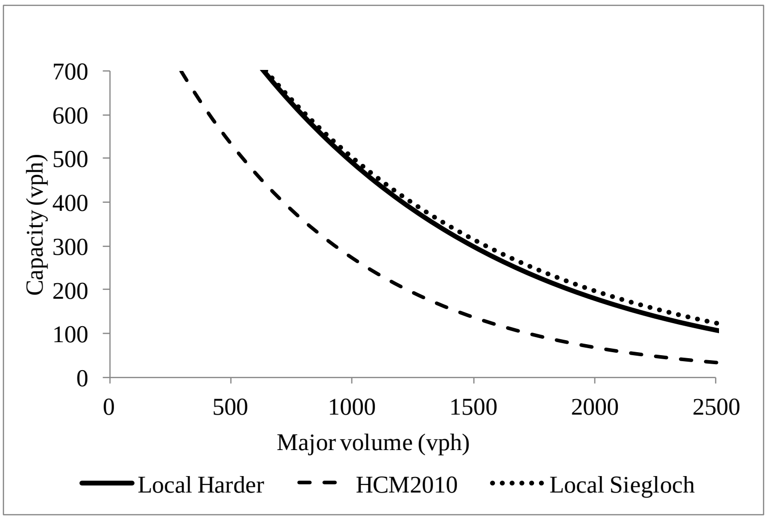

Figure 2 - Comparison of Harder's and Siegloch's calibrated formulations and HCM2010

Field capacity is defined as the true capacity measured in the field, and it serves as a basis for testing different capacity models. Model capacity is the capacity predicted by a theoretical model based on inputs of certain traffic and intersection characteristics (Kyte et al, 1996). Model testing process cannot be conducted unless field capacity is measured correctly since capacity model testing relies on these measurements.

Field capacity of a minor stream can be measured directly on the field under a continuous queue condition. The departure flow rate is unequivocally the actual field capacity of the minor street movement. At many intersections even a one-minute continuous queue situation may be a strange event. (Kyte et al., 1996).

Field capacity measures were conducted at the intersection that showed better adjusted values of theoretical capacity models when local headways were used. Figure 3 compares HCM2010, Harder's local capacity values and field capacity.

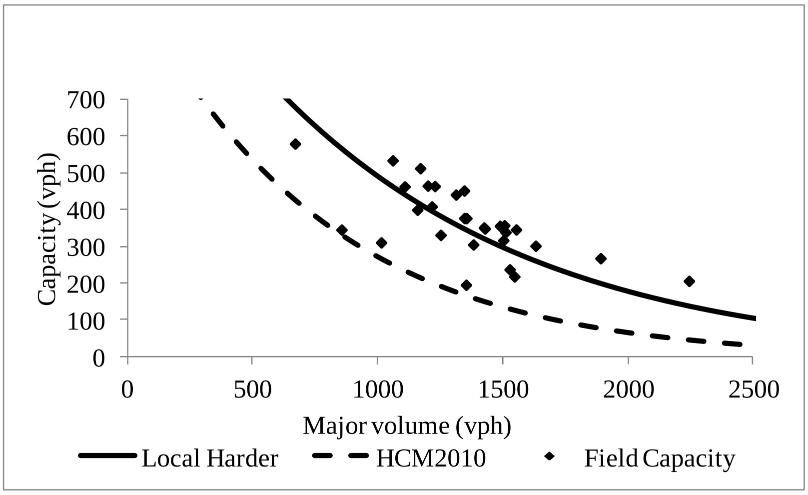

Figure 3 – Calibrated Harder's formulation, field capacity and HCM2010

3.2.4. Capacity estimates based on regression analysis

During a continuous queue period of twenty five minutes, the discharge rate of the minor street at the three leg intersection was measured in one-minute intervals. At the same time volumes at the main street were registered. Figure 4 plots both. An exponential function was adjusted calibrating A and B parameters in order to establish the capacity / conflicting volume relationship in Equation 25.

$$C = 1099.49e^{-0.000887 v_{c,x}} \qquad (25)$$

Values of t_c and t_f can be obtained from parameters A and B with Equations 26 and 27.

$$t_c = 3600B + \frac{t_f}{2} = 4.83s \qquad (26)$$

$$t_f = \frac{3600}{A} = 3.27s \qquad (27)$$

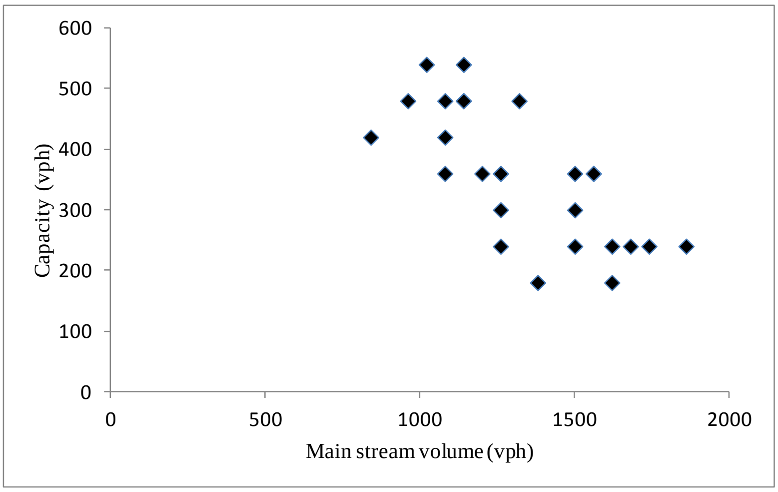

Figure 4 - Scatter plot: Capacity vs major street volumes

4. CONCLUSIONS AND RECOMENDATIONS

Research regarding capacity analysis at unsignalized intersections shows that the key parameters are critical headway (minimum time required between mainstream vehicles for the entry of a minor vehicle) and follow-up interval (interval between minor street vehicles queued, entering during the same critical headway on the main stream). These values are the best representatives of driver behavior influence on traffic conditions (Weinert, A., 1999). It should be clear that the values of critical and follow up headways by themselves are not important except to the extent that they feed those equations that enable capacity and delay estimation at the intersection.

Table 3 reports, for 4 leg intersections, critical and follow-up headways obtained by maximum likelihood methodology compared to those proposed in the HCM2010

Table 3 - Local headways and HCM2010 estimates. Four leg intersections.

Minor street movement	Critical headway (s) Two lanes		Critical headway (s) Four lanes		Follow-up time (s)	
	Córdoba	HCM2010	Córdoba	HCM2010	Córdoba	HCM2010
Right Turn	4.2	6.2	5.1	6.9	2.3	3.3
Through	6.0	6.5	6.4	6.5	1.8	4.0
Left turn	6.7	7.1	6.9	7.5	3.2	3.5

Table 4 reports, for 3 leg intersections, critical headways and follow-up times obtained by the application of different methodologies, in comparison to the HCM2010 values.

Table 4 - Local headways and HCM2010 estimates. Three leg intersections.

Minor street	Critical headway (s) Two lanes			Follow-up time (s)			Critical headway (s)	Follow-up time (s)
	MLM	REG	EXP	AVG	REG	EXP	HCM2010	
Left turn	4.77	5.26	4.83	2.8	2.86	3.27	6.4	3.5

MLM: Maximum likelihood methodology
REG: Linear regression for the estimation of critical headway and follow-up time jointly
EXP: Exponential regression for capacity and major stream volume. Headway estimates derived from A and B calibrated parameters of the exponential function
AVG: Simple average of minor street follow - up times

In every analyzed case, local critical and follow up headway estimates result in lower values than those proposed in HCM 2010. Therefore local drivers are more aggressive.

Considering their strong influence on capacity determination, the use of local values is recommended. Increased capacity, due to critical headway and follow-up time reductions, is proportionally greater as conflicting flow increases. While curves tend to approach the percent difference is greater.

Unsignalized intersections work efficiently when traffic volumes are low because delays are usually lower. If on the other hand traffic volumes increase and driver behavior is far more aggressive as local data shows, safety becomes a great issue. From this perspective it can be inferred that intersections operate more efficiently, but also more dangerously.

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