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Headway Compression during Queue Discharge at Signalized Intersections under Mixed Traffic Condition

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Abstract

The evaluation of capacity at signalized intersection is an important component in the planning, design, operation and management of transportation system. Presently, the methodologies available for the estimation of capacity of signalized intersections are based on the concept of saturation flow(s). Saturation flow is the steady maximum queue discharge rate after the green onset. According to the U.S. Highway Capacity Manual, this steady maximum rate is generally achieved after the fourth queuing vehicle enters the intersection. For the present work, data were collected at different signalized intersections of NH-2 and NH-5 under mixed traffic conditions. It has been found in the present study that the queue discharge headways show an unmistakable pattern of gradual compression as queuing vehicles are discharged in succession. Consequently, the average discharge headways become stable from position ninth at NH-2 intersections and from position eighth at NH-5 intersections. Saturation headway (h_s) was calculated by averaging the headways of all the vehicles in the saturation flow region and it was 1.70 and 1.64 s for NH-2 and NH-5 respectively. The queue discharge characterizations of all the intersections of NH-2 were found to be similar to those observed at intersection of NH-5.

Keywords: *Saturation flow, signalized intersection, saturation flow region, queue discharge, saturation headway*

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INTRODUCTION

The performance of signalized intersections often requires the determination of capacity of approach lane or lane group. The traditional approach to capacity estimation is based on the concept of saturation flow [1–3]. The saturation flow rate(s) is the maximum flow rate on the approach lane or lane group that can go through the intersection under prevailing traffic and roadway conditions when 100% effective green time is available. The U. S. Highway Capacity Manual (HCM) indicates that the queue discharge typically reaches its saturation flow after the fourth queuing vehicle enters the intersection [2].

The fundamental element of a signalized intersection is the periodic stopping and starting of the traffic stream. When a signal turns green from red, first of the stopped vehicles (first vehicle of the queue) starts to move and cross the intersection, then the second vehicle in the queue and so on. As the queue of vehicles moves, the headway measurements with respect to a fixed point or line (say STOP line) are taken. If many queues of vehicles are observed at a given intersection and the average headway is determined, this average headway tends towards a constant value. Greenshields found that the average discharge headways become constant after the first three or four vehicles [4]. This finding, which has been verified in numerous studies [5, 6], is the basis of theory behind the Highway Capacity Manual (HCM) [7], a number of signal optimization models, and a number of traffic simulation models. The constant headway achieved is referred to as the saturation headway (h_s) because it is the average headway that can be achieved by a saturated, stable moving queue of vehicles passing through the signal. This saturation headway (h_s) can be used to determine the maximum number of vehicles that can be released during a specified green time and also to determine the saturation flow rate, $s = 3600/h_s$. For example, vehicles departing from a queue with an average headway of 2.2 s have a saturation flow rate of 3600/2.2 = 1636 vphpl (vehicles per hour per lane). The saturation flow rate(s) is an important parameter for estimating the performance of vehicular movement at signalized intersections. Saturation flow rate for a lane group is a direct function of vehicle speed and separation distance. On the basis of saturation flow, the capacity of a traffic lane or lane group is determined in HCM as follows:

$$c = s\frac{G+Y-L}{C} \tag{1}$$

where,

c = capacity of a lane or lane group (vehicles per hour, vph)

s = saturation flow (vph of effective green time)

C = cycle length(s)

G = green interval(s)

Y = signal change or intergreen interval(s)

L = loss time in a single phase resulting from start-up delays and signal change from green interval to yellow warning interval(s)

The above equation is a convenient tool for capacity estimation of a signalized intersection. Its reliability, however, depends on whether the queue discharge rate would quickly reach a steady or the discharge headway becomes steady after green onset. The present work was done to study the headway characteristics of vehicles during queue discharge under mixed traffic condition and to determine the saturation headway (h_s) and the region of saturation flow.

EARLY OBSERVATIONS

The saturation flow is used as the basis for the determination of traffic signal timing and evaluation of intersection performance. One of the problems in estimating saturation flow from observed headway data is accurately defining the saturation flow region of the queue of vehicles. Branston [8] studied the factors affecting the capacity of signalized intersections and found the differences in saturation flows between peak and off-peak periods, as well as between day-time and darkness. He recommended three parallel straight lines as a good representation of the observed variations in saturation flow at different times of the day. Stokes *et al.* [9] used regression model for double left-turn vehicles to determine the saturation flow region of the queue. They opined that this model is also applicable to straight-through vehicles, although the headway values of left-turn vehicles and straight-through vehicles are not the same. Fujiwara *et al.* [10] investigated the effect of winter season on saturation flows at urban signalized intersections. They suggested that the decrease in winter saturation flow rates is about 20% as compared to the observed non-winter saturation flow rates; at most the variation could be around 30% in comparison with the basic saturation flow rates. Mccoy and Heimann [11] conducted a study to investigate the effect of driveway traffic on saturation flow rates at signalized intersections. The authors concluded that the driveway traffic can reduce the saturation flow rates on signalized intersection approaches. Cohen [12] studied the application of car-following systems to queue discharge problems at signalized intersections. Rahman *et al.* [13] compared the saturation flow rates at signalized intersections in Yokohama and Dhaka. They found that the procedure followed in HCM overestimates saturation flow rates for some approaches and underestimates other approaches. They found that the headway values of passenger cars in Dhaka metropolis are lower than that of Yokohama. Lin and Thomas [14] studied headway compression during queue discharge at signalized intersections. They opined that the saturation flow does not reach stabilized maximum value after the fourth queuing vehicle enters the intersection. The maximum discharge rates were associated with the last few vehicles beyond queue position 15. Stokes [15] reviewed a study on the factors affecting signalized intersection capacity. A review of this study suggested that intersection capacity is affected by a number of physical and operational features at or in the vicinity of the intersections. The research suggested that past efforts were fragmented in terms of study methods, location characteristics, and technical objectives. No study was related to determination of saturation flow region except Stokes *et al.* [9]. According to the Highway Capacity Manual (HCM, 2000) the saturation flow region starts after the fourth queuing vehicle enters the intersection, although this may not be applicable to Asian cities because of differing road environment and operating characteristics.

METHODOLOGY AND DATA COLLECTION

For the purpose of the present study, data were collected at different signalized intersections of NH-2 (Delhi-Kolkata road) and NH-5 (Kolkata-Chennai road). The data were collected for the headway of vehicles at signalized intersections on the rural highways. The intersections were free from bus stops, parked vehicles, and pedestrians. All intersections were having 14.0 m wide carriageway and the pavement surface condition was fair. Data were collected for about 2–3 h on different weekdays on all the intersections. In all, more than 1000 cycles of traffic data were collected. Queuing vehicles refer to those vehicles that are stopped by the red light and those join the stationary queue of vehicles. Due care was also taken to eliminate the data of vehicles that did not stop before the stop line. The platoons of traffic within which vehicles did not stop before entering an intersection, and platoons in which vehicular movements were impeded by pedestrians, cross traffic, or turning vehicles were not considered. In other words, only platoons of traffic containing uninterrupted, straight-through vehicles stopped before entering an intersection were considered as valid cases for the study. As the queue of vehicles moves, the headway measurements with respect to a fixed point or line (say STOP line) are taken as follows:

- The first headway is the time lapse between the initiation of green signal and the time that the rear wheel (car-following laws assumes that the drivers follow the rear bumper of the leader, not the front bumper [16] of the first vehicle in the queue cross the stop line].
- The second headway is the time lapse between the time that the first vehicle's rear wheels cross the stop line and the time that the second vehicle's rear wheels cross the stop line.
- Subsequent headways are similarly measured.

The headway data were collected with the help of stopwatch with an accuracy of 0.01 s. The discharge headway data were collected for the vehicles in each queue position. The left side straight-through lane was chosen for the study.

ANALYSIS OF DATA

The collected data were analyzed to find out the saturation flow region of the queued vehicles. Before finding the queue position from which saturation flow region started, it is necessary to determine the average headway for the vehicles in each queue position. Table 1 shows the headway statistics of collected data on signalized intersections of NH-2 and NH-5.

The initial headways are larger than the saturation headway. The first driver in the queue needs to observe and react to the signal change at the start of green time. After the observation, the driver accelerates through the intersection from stand-still which results in a relatively long first headway. The second driver performs the same process with an exception that the driver could react and start accelerating whilst the first vehicle began moving. This results in a shorter headway than the first, because the second driver had an extra length in which to accelerate. This process carries through with all following vehicles where each vehicle's headway will be slightly shorter than the preceding vehicle. This continues until a certain number of vehicles have crossed the intersection and start-up reaction and acceleration no longer have an effect on the headways. From this point headways will remain relatively constant until all vehicles in the queue have crossed the intersection or green time has ended. This constant headway is known as the saturation headway and can start to occur anywhere between the third and sixth vehicle in the queue [17].

To provide a better insight into the nature of queue discharge, the headway data given in Table 1 are represented graphically in Figures 1 and 2. Figures 1 and 2 show that contrary to the conventional findings, the average headway does not stabilize after the fourth queuing vehicle enters the intersection.

Table 1: *Headway Statistics at Intersections of NH-2 and NH-5.*

Queue position	**NH-2**			**NH-5**		
	Mean(s)	**S. D.(s)**	**Sample size**	**Mean(s)**	**S. D.(s)**	**Sample size**
1	3.96	1.33	562	3.65	1.37	475
2	3.14	1.02	562	2.61	1.14	475
3	2.73	0.93	562	2.38	0.97	475
4	2.45	0.77	531	2.31	0.69	475
5	2.24	0.66	501	2.29	0.88	463
6	2.21	0.71	458	2.19	0.67	463
7	2.12	0.66	411	2.01	0.62	449
8	1.95	0.63	348	1.71	0.73	378
9	1.73	0.54	277	1.65	0.66	370
10	1.78	0.49	209	1.68	0.73	354
11	1.71	0.49	145	1.78	0.53	345
12	1.67	0.52	95	1.65	0.69	275
13	1.65	0.37	71	1.61	0.54	203
14	1.74	0.34	42	1.69	0.56	178
15	1.69	0.27	19	1.69	0.48	105
16	1.70	0.26	9	1.71	0.31	73
17	1.65	0.18	3	1.55	0.56	61
18	--	--	--	1.51	0.43	35
19	--	--	--	1.56	0.28	19
20	--	--	--	1.52	0.35	7
Saturation headway*	1.70	--	--	1.64	--	--

**Saturation headway was calculated by averaging the headways of all the vehicles in the saturation flow region.*

In aggregate, the average queue discharge headways display an unmistakable pattern of gradual compression as queuing vehicles are discharged in succession. Field studies of one left-turn lane in Hawaii [16] and five straight-through lanes in Taiwan [18] have shown that the queue discharge headway tends to compress as the discharge of a queue at signalized intersection progresses. The average discharge headways become stable from position 9th (ninth) at NH-2 intersection and from position 8th (eighth) at NH-5 intersection. The average discharge headway or saturation headway is calculated by averaging the discharging headway of all vehicles in the saturation flow region as given in Eq. (2) and it is 1.70 s and 1.64 s for NH-2 and NH-5 respectively.

$$h_s = \frac{\sum_{j=n}^{l}(h_j)}{(l-n+1)} \qquad (2)$$

where,

h_s = saturation headway, s

h_j = discharge headway of j-th queued vehicle, s

n = position of queued vehicle from where saturation flow region started

l = last queued vehicle position.

Therefore, it can be said that under mixed traffic conditions, after the eighth or ninth vehicle in the queue, the region of saturation flow starts or the steady queue discharge rate starts. Contrary to the conventional concept of saturation flow, the discharge does not reach stabilized maximum after the fourth queuing vehicle enters the intersection. Field studies at three intersections on Long Island, New York, have shown that the maximum discharge rates were associated with the last few vehicles beyond queue position 15 [14].

The cumulative queue discharge headways at NH-2 and NH-5 are studied with the help of maximum likelihood method and regression analysis. The cumulative observed headways (H) were plotted to fit the data points to a straight line by maximum likelihood method.

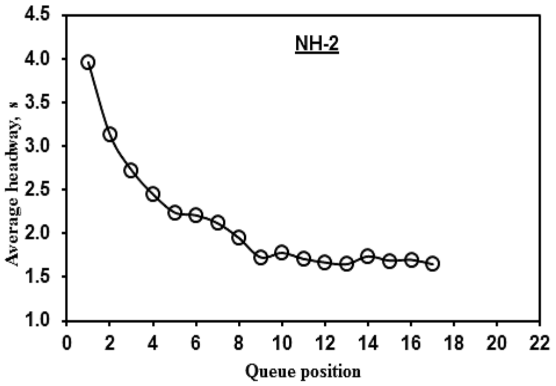

Fig. 1: *Characteristics of Queue Discharge Headways at NH-2 Intersection.*

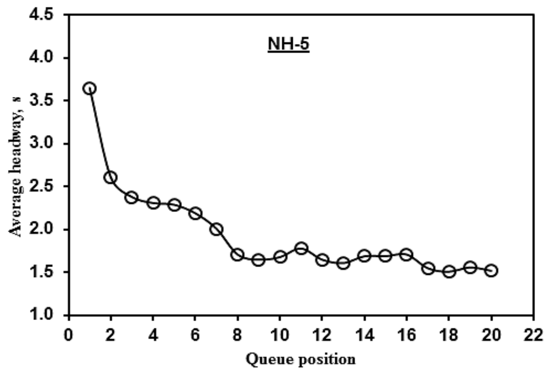

Fig. 2: *Characteristics of Queue Discharge Headways at NH-5 Intersection.*

Suppose we know the functional relationship between the y's and the x's as:
y = q(x, α, β,...)
where, α, β, ...are parameters
Maximum likelihood method gives us a method to determine α, β, ...from our observed data.
The functional relationship between the (cumulative headways) H's and the queue position Q's is as
H = *q*(Q, α, β)
Now the fitting of data points to a straight line:
q(Q, α, β) = α + β*Q
H = α + β*Q
Then the likelihood function is

$$L=\prod_{i=1}^{n} f(Q_i,\alpha,\beta)=\prod_{i=1}^{n}\frac{1}{\sigma_i\sqrt{2\pi}}e^{-\frac{(H_i-q(Q_i,\alpha,\beta))^2}{2\sigma_i^2}}=\prod_{i=1}^{n}\frac{1}{\sigma_i\sqrt{2\pi}}e^{-\frac{(H_i-\alpha-\beta Q_i)^2}{2\sigma_i^2}}$$

The value of α and β is determined by maximizing the likelihood function L:

$$\frac{\partial \ln L}{\partial \alpha}=\frac{\partial}{\partial \alpha}\sum_{i=1}^{n}\left[\ln\left(\frac{1}{\sigma_i\sqrt{2\pi}}\right)-\frac{(H_i-\alpha-\beta Q_i)^2}{2\sigma_i^2}\right]=\sum_{i=1}^{n}\left[-\frac{2(H_i-\alpha-\beta Q_i)(-1)}{2\sigma_i^2}\right]=0$$

$$\frac{\partial \ln L}{\partial \beta}=\frac{\partial}{\partial \beta}\sum_{i=1}^{n}\left[\ln\left(\frac{1}{\sigma_i\sqrt{2\pi}}\right)-\frac{(H_i-\alpha-\beta Q_i)^2}{2\sigma_i^2}\right]=\sum_{i=1}^{n}\left[-\frac{2(H_i-\alpha-\beta Q_i)(-Q_i)}{2\sigma_i^2}\right]=0$$

Assume all σ's are the same for simplicity:

$$\sum_{i=1}^{n}H_i-\sum_{i=1}^{n}\alpha-\sum_{i=1}^{n}\beta Q_i=0\;;\quad \sum_{i=1}^{n}H_iQ_i-\sum_{i=1}^{n}\alpha Q_i-\sum_{i=1}^{n}\beta Q_i^{\,2}=0$$

We now have two equations that are linear in the two unknowns, α and β

$$\sum_{i=1}^{n}H_i=n\alpha+\beta\sum_{i=1}^{n}Q_i\;;\quad \sum_{i=1}^{n}H_iQ_i=\alpha\sum_{i=1}^{n}Q_i+\beta\sum_{i=1}^{n}Q_i^2$$

Matrix From-
$$\begin{bmatrix}\sum_{i=1}^{n}H_i\\ \sum_{i=1}^{n}H_iQ_i\end{bmatrix}=\begin{bmatrix}n & \sum_{i=1}^{n}Q_i\\ \sum_{i=1}^{n}Q_i & \sum_{i=1}^{n}Q_i^2\end{bmatrix}\begin{bmatrix}\alpha\\ \beta\end{bmatrix}$$

$$\alpha=\frac{\sum_{i=1}^{n}H_i\sum_{i=1}^{n}Q_i^2-\sum_{i=1}^{n}H_iQ_i\sum_{i=1}^{n}Q_i}{n\sum_{i=1}^{n}Q_i^2-\left(\sum_{i=1}^{n}Q_i\right)^2}\quad\text{and}\quad \beta=\frac{n\sum_{i=1}^{n}Q_iH_i-\sum_{i=1}^{n}H_i\sum_{i=1}^{n}Q_i}{n\sum_{i=1}^{n}Q_i^2-\left(\sum_{i=1}^{n}Q_i\right)^2}$$

Table 2: *Values of α and β by Different Methods.*

Name of road	Maximum likelihood method		Regression analysis		
	α	β	α	β	(R^2 value)
NH-5	2.30	1.91	3.81	1.80	0.994
NH-2	4.23	1.94	4.23	1.94	0.991

Our model for the cumulative headway (H) is $H = \alpha + \beta*Q$, i.e., cumulative headway = (best estimated value of headway)*queue position + best estimated value of starting point. By using maximum likelihood method the best value of headway (β) was estimated for NH-5 and NH-2 and the values are given in Table 2.

The cumulative observed headways (H) were also plotted to fit the data points by regression analysis method and the data points (queue position, cumulative headway) were best fitted by a straight line as explained above, i.e., $H = \alpha + \beta*Q$. The values of α and β for NH-5 and NH-2 were determined by this method and are tabulated in Table 2.

For individual queue position of vehicles in NH-2 and NH-5, the observed headways were compared with the best estimated headways. In the intersection of NH-5, the observed headways were on the higher side of the best estimated headway (1.91 s by maximum likelihood method and 1.80 s by regression analysis) up to the position of seventh vehicle in the queue. From eighth position onwards, the observed headways were less than the best estimated headway indicating that the zone of stable flow starts from the eighth position of vehicle in the queue at NH-5 intersections. Similarly, in intersections of NH-2, from the ninth position onwards the observed headways for individual queue position were less than the best estimated headway indicating that the headways become stable from ninth position in the queue.

Although Figures 1 and 2 are useful to describe the general characteristics of queue discharge, the sample sizes for some queue positions are small. This smaller sample sizes for some queue positions may exaggerate the relationship between the average queue discharge headway and queue position as already explained in Figures 1 and 2. To provide a better understanding (perspective), successive two or three queue positions were grouped and their average queue discharge rates were calculated and compared. Figures 3 and 4 show the relationship between the average queue discharge rate and queue position group.

Figure 3 shows that at NH-2 intersection, the average discharge rate of each position group increases. The increase from one group to another is statistically significant. In particular, the average discharge rate for position 1 through 3 is 1099 vphpl whereas that for positions 13 through 15 is 2137 vphpl. This represents a 95% increase. Similar trend was also observed in case of intersection at NH-5 as shown in Figure 4. Figure 5 shows the queue discharge characteristics in NH-2 and NH-5. The queue discharge rate tends to keep rising long after the green onset.

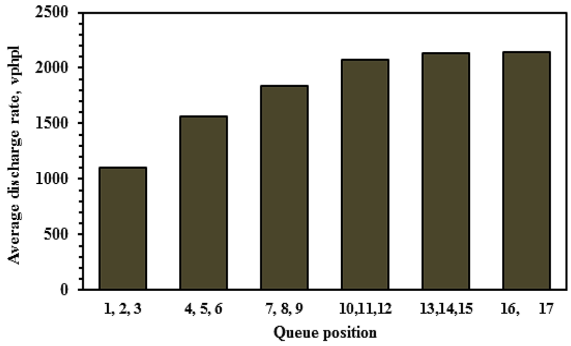

Fig. 3: *Average Queue Discharge Rate as Function of Queue Position Group at NH-2 Intersection.*

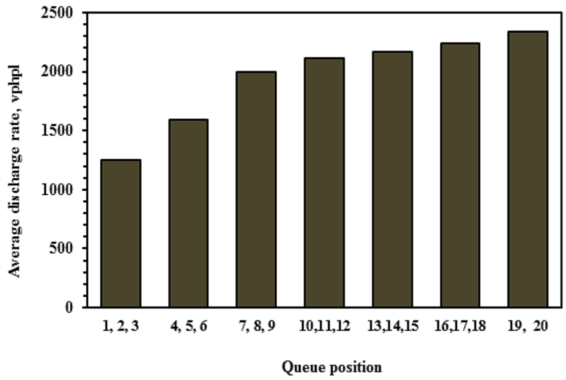

Fig. 4: *Average Queue Discharge Rate as Function of Queue Position Group at NH-5 Intersection.*

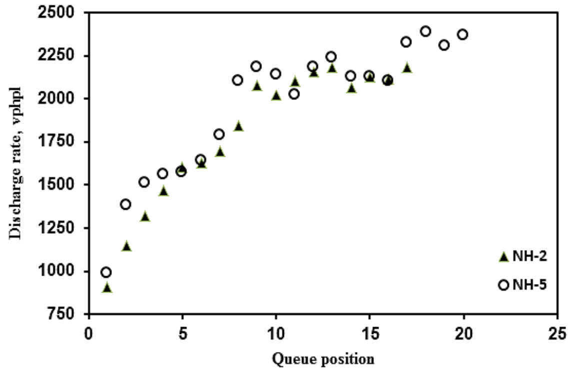

Fig. 5: *Queue Discharge Characteristics in NH-2 and NH-5.*

IMPLICATIONS

The headway compression during queue discharge at signalized intersection is common. The ideal saturation flow rate may not be achieved (observed) or sustained during each signal cycle. There are numerous situations where actual flow rates will not reach the average saturation flow rate on an approach including situations where demand is not able to reach the stop line, queues are less than five vehicles in a lane, or during cycles with a high proportion of heavy vehicles. To achieve optimal efficiency and maximize vehicular throughput at the signalized intersection, traffic flow must be sustained at or near saturation flow rate on each approach. In most HCM analyses, the value of saturation flow rate is a constant based on the parameters input by the user, but in reality, this is a value that varies depending on the cycle-by-cycle variation of situations and users. The traditional concept of saturation flow may not realistically represent the actual queue discharge characteristics. The HCM provides a standardized technique for measuring saturation flow rate. It is based on measuring the headway between vehicles departing from the stop line, limited to those vehicles between the fourth position in the queue and the end of the queue. For signal designing, it is often necessary to put emphasis on this parameter due to the high degree of fluctuation in this parameter from cycle to cycle. It is also difficult to identify the position of queued vehicles from where the saturation flow region starts or the steady maximum discharge rate, which would create two problems with the use of Eq. (1) for capacity estimation of a signalized intersection. First, it is difficult to define saturation flow from observed data. Second, the correct lost time (L) to be used in the equation becomes a direct function of vehicle speed and separation distance. These are in turn functions of a variety of parameters, including the number and width of lanes, lane use (e.g., exclusive versus shared lane use), grades, and factors that constrain vehicle movement such as presence or absence of conflicting vehicle and/or pedestrian traffic, on-street parking, and bus movements. As a result, lost time (L) varies by movement, time, and location of the intersections. For example, the correct lost time for one intersection and that for another can differ by more than 5 s [18]. This large variation makes the estimates obtained from Eq. (1) susceptible to large errors.

CONCLUSIONS

The queue discharge characteristics under mixed traffic conditions exhibit a general trend of gradual compression of headways as the queue discharge continues. Contrary to the traditional concept of saturation flow, the discharge rates do not become stable after the fourth queuing vehicle enters the intersection. The average discharge headways become stable from position 9th (ninth) at NH-2 intersection and from position 8th (eighth) at NH-5 intersection. The saturation headway is 1.70 s and 1.64 s for NH-2 and NH-5 respectively.

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