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FALLACIES AND IMPLICATIONS OF CONVENTIONAL SATURATION FLOW MODEL OF QUEUE DISCHARGE BEHAVIOR AT SIGNALIZED INTERSECTIONS

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Abstract: Current methodologies used in many countries for capacity and level-of-service analysis of signalized intersections are based on the concept of saturation flow. They assume that, after green onset, the discharge rate of queuing vehicles will quickly reach its saturation flow rate after four or five vehicles have entered the intersection, and that the saturation flow will be sustained until shortly after the signal change interval begins. Recent studies, however, show that this saturation flow model often cannot realistically represent the observed queue discharge behaviors. Therefore continued use of the traditional model will have serious implications. This paper uses field data collected in Taiwan and the United States to analyze first the discrepancies between the observed queue discharge behaviors and the saturation flow model. It then discusses the implications of continued use of saturation flow for capacity estimation.

Key Words: Highway capacity analysis, Queue discharge behaviors, Saturation flow, Signalized intersection

1. INTRODUCTION

Current methodologies used in Australia (Akcelik 1982), Canada (Teply, S., *et al*. 1995), Great Britain (Kimber, R. M., *et al.* 1986), Sweden (Petersen and Imre 1977), Taiwan (Institute of Transportation 2001), and the United States (Transportation Research Board 2000) for capacity and level-of-service analysis of signalized intersections are based on the concept of saturation flow. As shown in Figure 1, saturation flow is considered to be a steady maximum rate of queue discharge after the green light is turned on. It is traditionally assumed that, after green onset, the discharge rate of queuing vehicles will quickly reach its saturation flow after four or five vehicles have entered the intersection, and that the saturation flow will be sustained until shortly after the signal change interval begins. Based on this assumed behavior, the U.S. *Highway Capacity Manual* (Transportation Research Board 2000) suggests that saturation flow be determined as the average discharge rate of the queuing vehicles after the fourth queuing vehicle enters the intersection.

The aforementioned saturation flow model makes it simple to estimate the capacity of a lane or lane group at a signalized intersection. Recent field data, however, has raised questions about the validity of the model. Most of these data were collected as part of an ongoing effort by the Institute of Transportation (IOT) in Taiwan to revise Taiwan's *Highway Capacity*

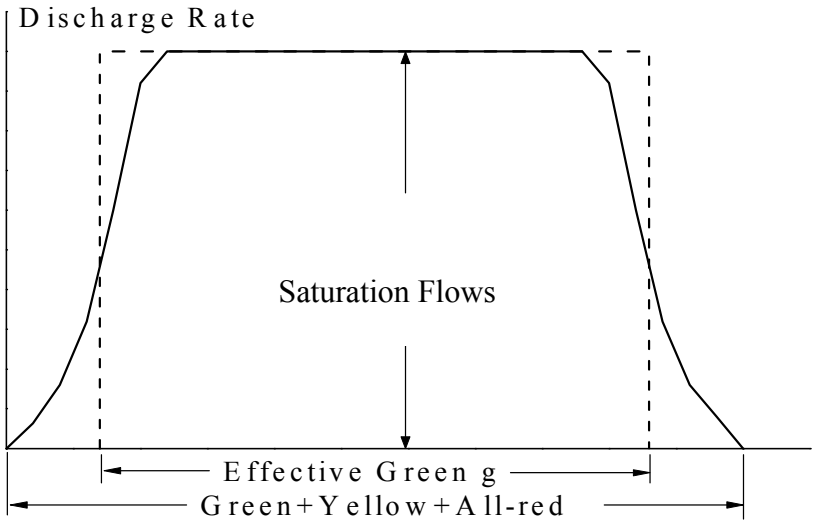

Figure 1. Traditional Concept of Saturation Flow

Manual (Institute of Transportation 2001). The first hint that the traditional saturation flow model is not a valid representation of the traffic characteristics in Taiwan emerged in 2002after the queue discharge characteristics in three straight-through lanes were analyzed. Instead of reaching quickly a steady maximum after green onset as indicated by the saturation flow model, the queue discharge rates observed in all these lanes kept rising long after the green light was turned on (Tseng and Lin 2003). Data collected later from additional straight-through lanes revealed the same phenomenon (Lin, *et al*., 2004). More recently, IOT's data on left-turn and right-turn queue discharges also showed that the traditional saturation flow model is a poor representation of the actual queue discharge characteristics. In the United States, an earlier study of a left-turn lane showed that the queue discharge rate continued to rise even after fifteen vehicles had been discharged (Li and Prevedouros 2002). To determine whether the queue discharge characteristics observed in Hawaii also exist elsewhere in the United States, one of the authors of this paper conducted a study in the summer of 2004 on three straight-through lanes on Long Island, New York. The field data showed again that the observed queue discharge characteristics were quite different from the behavior assumed in the traditional model.

Significant discrepancies between the saturation flow model and the observed queue discharge characteristics could have profound implications. Capacity estimates are used in many countries to estimate delays, fuel consumptions, and air pollution related to the operation of signalized intersections. In Taiwan and the United States, delay estimates are also used to determine the levels of service of signalized intersections. And the estimated levels of service, in turn, become the basis for making planning, design, and operating decisions concerning signalized intersections. Small errors in capacity estimates can be magnified in the decision-making process. This paper uses the field data collected in Taiwan and Long Island, New York to analyze first such discrepancies. It then discusses the implications of continued use of saturation flow for capacity estimation and points out a new direction for further research.

2. DATABASE

The IOT data were collected from 20 traffic lanes at signalized intersections. Fourteen of these lanes provided for straight-through movements. Five other lanes were for left-turn vehicles and one lane was for right-turn vehicles. The Long Island data were collected from three straight-through lanes located on New York State Route 110. Table 1 summarizes the site characteristics.

Table 1. Site Characteristics of Study Lanes

Lane ID	Country	Area Type	Directional Movement	Lane width (m)	Speed limit (km/h)
S1	Taiwan	Suburban	Straight through	3.0	70
S2	Taiwan	Suburban	Straight through	3.2	60
S3	Taiwan	Suburban	Straight through	2.6	60
S4	Taiwan	Suburban	Straight through	3.2	60
S5	Taiwan	Suburban	Straight through	3.5	60
S6	Taiwan	Suburban	Straight through	3.5	70
S7	Taiwan	Suburban	Straight through	3.1	60
S8	Taiwan	Rural	Straight through	3.6	80
S9	Taiwan	Suburban	Straight through	3.5	70
S10	Taiwan	Suburban	Straight through	3.2	70
S11	Taiwan	Suburban	Straight through	3.2	70
S12	Taiwan	Urban	Straight through	3.0	50
S13	Taiwan	Urban	Straight through	3.1	50
S14	Taiwan	Urban	Straight through	3.0	50
S15	U.S.	Suburban	Straight through	3.5	80
S16	U.S	Suburban	Straight through	3.5	80
S17	U.S	Suburban	Straight through	3.5	80
L1	Taiwan	Suburban	Left-turn	3.4	50
L2	Taiwan	Suburban	Left-turn	3.4	60
L3	Taiwan	Urban	Left-turn	3.1	50
L4	Taiwan	Suburban	Left-turn	3.2	50
L5	Taiwan	Suburban	Left-turn	3.5	60
R1	Taiwan	Suburban	Right-turn	3.5	60

Each of the study lane accommodated cars, vans, and various types of trucks. Trucks accounted for a small proportion of the observed vehicles. To avoid the random effects of trucks on queue discharge, large trucks and all the vehicles behind a truck in the same queue were ignored. For convenience, all small vehicles considered in this study are referred to herein as passenger cars. Following the common practice in North America, a vehicle was considered to be in a queue if that vehicle was either stopped behind the stop line or approaching a queuing vehicle ahead slowly and was already less than about one car length from the rear end of the vehicle ahead. This practice takes into consideration the fact that, after green onset, some vehicles that cross the stop line in a densely packed platoon may not have come to a complete stop when joining the queuing vehicles ahead. The data collection involved the measurement of the time when the rear wheel of each queuing vehicle crossed

the stop line after green onset. Given that four successive vehicles were respectively discharged at t1, t2, t3, and t4, the total time needed to discharge the last three vehicles was determined as t4 minus t1. This time was then used to determine the discharge rate of that group of three queuing vehicles. Including all three lanes on Long Island but excluding four lanes that have relatively short queues, Table 2 through Table 5 summarize the means and the standard deviations of the discharge rates of successive queue position groups in the study lanes.

The IOT data were collected with an electronic stopwatch that could store 60 records. The measurement error for each group of three vehicles could be as much as 0.2 s. Since it usually took more than 5 s to discharge three queuing vehicles, the percentage errors in the mean discharge rates were about 4% or less. The Long Island data were reduced from surveillance videotapes on a frame-by-frame basis. The measurement errors were smaller.

Table 2. Average Discharge Rates (vph)/Standard Deviations (vph)/ Sample Sizes of Three-Vehicle Groups in Lanes S1 ~S5

Queue Positions	Lane ID				
	S1	S2	S3	S4	S5
1~3	1139/172/142	1299/189/114	1153/188/114	1213/212/106	1310/237/180
4~6	1584/254/135	1782/310/112	1678/313/111	1810/269/ 99	1807/298/133
7~9	1722/275/135	1821/326/105	1822/305/101	1907/354/ 94	1829/342/ 84
10~12	1767/317/126	1915/369/ 89	1896/338/ 92	2003/405/ 89	1849/412/ 54
13~15	1827/319/110	2059/405/ 64	1926/468/ 79	2082/444/ 80	1882/379/ 32
16~18	1839/415/ 91	1875/376/ 52	2007/394/ 59	2028/339/ 59	2041/313/ 15
19~21	1878/407/ 60	2014/425/ 40	2022/353/ 27	2035/413/ 45	1779/264/ 5
22~24	1721/386/ 22	2089/395/ 23	2266/406/ 12	2191/444/ 31	2539/246/ 2
25~27	1901/299/ 14	2175/492/ 11	2107/967/ 2	2094/415/ 18	2552/572/ 2
28~30	2183/646/ 9	2222/314/ 4		1912/618/ 13	

Table 3. Average Discharge Rates (vph)/Standard Deviations (vph)/ Sample Sizes of Three-Vehicle Groups in Lanes S8~S12

Queue Positions	Lane ID				
	S8	S9	S10	S11	S12
1~3	1290/270/212	1186/242/178	1189/177/133	1107/170/178	1202/207/181
4~6	1663/330/174	1862/310/178	1701/274/128	1697/293/174	1668/268/178
7~9	1819/411/134	1931/356/171	1791/378/115	1791/343/167	1742/290/145
10~12	1839/449/ 93	2002/415/151	1794/366/ 92	1868/374/141	1838/368/118
13~15	1969/410/ 55	2053/444/124	1950/488/ 62	1891/350/100	1934/332/ 80
16~18	1926/412/ 43	2174/425/101	1992/585/ 30	1921/306/ 57	1887/380/ 49
19~21	2023/538/ 30	2241/406/ 79	2083/550/ 10	1916/428/ 25	1949/327/ 28
22~24	2033/370/ 23	2320/370/ 59	1949/554/ 3	2304/770/ 6	2000/318/ 18
25~27	2277/346/ 16	2413/385/ 39			1879/304/ 6
28~30	1910/197/ 7	2482/449/ 31			1886/154/ 4

Table 4. Average Discharge Rates (vph)/Standard Deviations (vph)/ Sample Sizes of Three-Vehicle Groups in Lanes S11 Through S13~S17

Queue Positions	Lane ID				
	S13	S14	S15	S16	S17
1~3	1087/170/147	1236/218/156	1369/290/211	1411/175/178	1411/168/91
4~6	1607/262/146	1677/252/154	1877/327/163	1842/290/178	1967/391/89
7~9	1742/303/141	1908/289/147	2026/399/119	1865/382/153	2130/438/87
10~12	1821/314/127	1939/296/126	2138/426/ 68	1975/448/ 97	2277/407/82
13~15	1852/327/112	2021/292/ 90	2208/400/ 42	2108/498/ 41	2201/416/68
16~18	1898/327/ 80	2129/305/ 57	2126/482/ 15	2261379/ 11	2505/679/38
19~21	1846/261/ 48	2119/281/ 38	2536/460/ 8		
22~24	1918/314/ 28	2239/256/ 21			
25~27	2075/207/ 13	2164/258/ 5			
28~30	2339/120/ 6	2324/176/ 2			

Table 5. Average Discharge Rates (vph)/Standard Deviations (vph)/ Sample Sizes of Three-Vehicle Groups in Left-Turn and Right-Turn Lanes

Queue Positions	ID				
	L1	L3	L4	L5	R1
1~3	1252/218/181	1117/184/161	1315/225/169	1110/246/99	1200/206/97
4~6	1433/203/180	1591/249/161	1685/207/163	1188/216/91	1555/235/92
7~9	1440/209/171	1726/309/139	1818/236/148	1223/190/80	1647/223/82
10~12	1473/258/124	1744/352/ 91	1898/313/127	1296/199/62	1727/209/65
13~15	1548/277/ 63	1824/346/ 61	2042/327/ 93	1311/233/40	1687/252/54
16~18	1424/160/ 7	1871/305/ 41	2101/294/ 51	1318/197/11	1802/248/36
19~21	1542/133/ 4	1807/319/ 25	2140/324/ 23		1997/242/ 9
22~24		1844/242/ 9	2106/200/ 9		2221/231/ 2
25~27		1901/145/ 3	2147/313/ 2		
28~30					

3. FALLACIES OF THE SATURATION FLOW MODEL

To provide a better perspective of the characteristics of queue discharge, the data are presented graphically in Figures 2 through 4. Based on the weighted average of the discharge rates of the study lanes for successive queue position groups, Figure 5 shows the aggregated discharge rates respectively for the straight-through lanes in the U.S.A., the straight-through lanes in Taiwan, and the left-turn lanes in Taiwan. By comparing this figure with Figure 1, it is obvious that the observed queue discharge characteristics are quite different from the conventional concept. In particular, the queue discharge rate in a lane rarely reaches an easily identifiable steady maximum. In general, the queue discharge rate tends to keep rising long after the green onset regardless of the directional movement of vehicles and the country where an intersection is located.

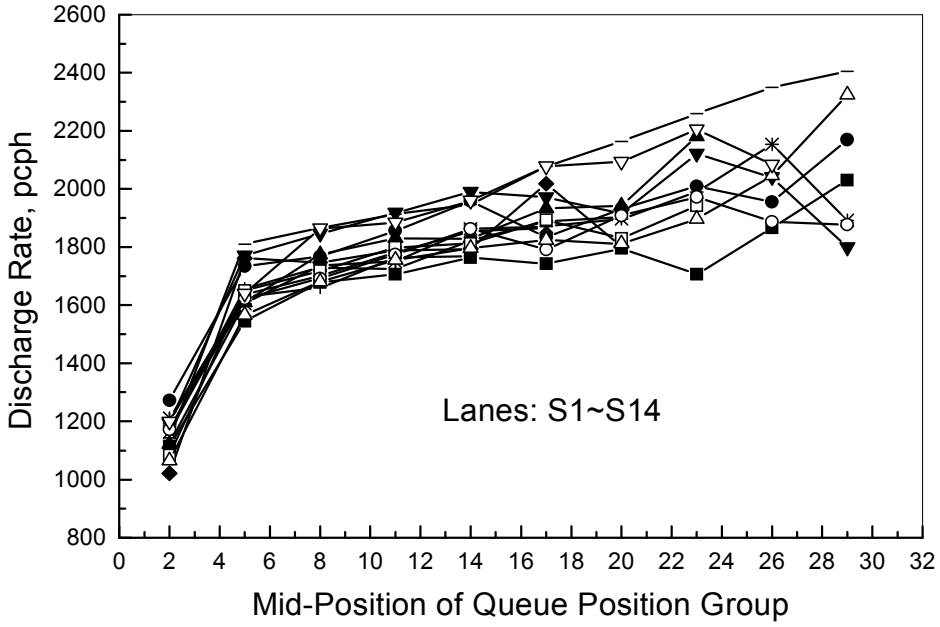

Figure 2. Relationships Between Queue Position and Discharge Rate of Straight-through Lanes in Taiwan

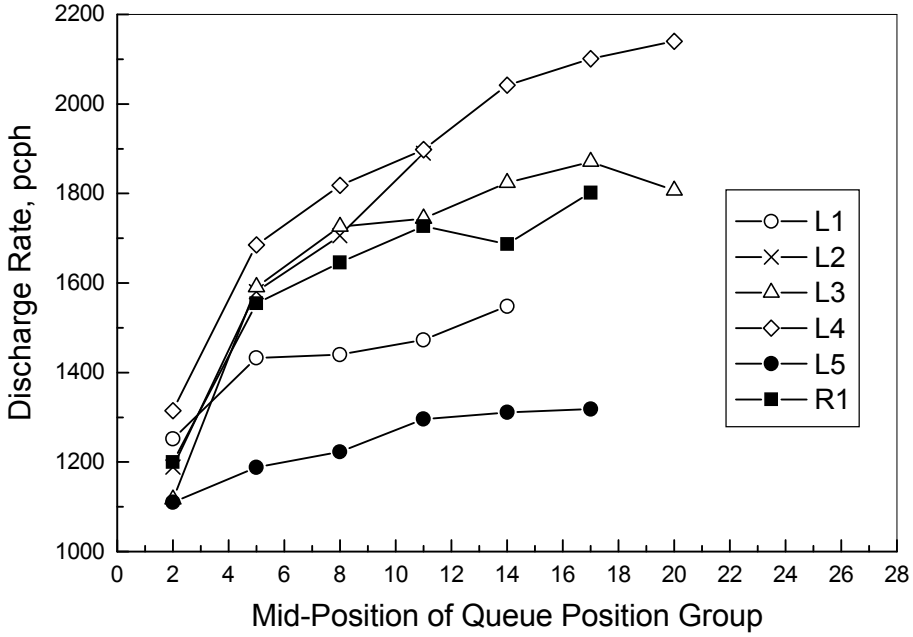

Figure 3. Relationships Between Queue Position and Discharge Rate of Left-turn and Right-turn Lanes in Taiwan

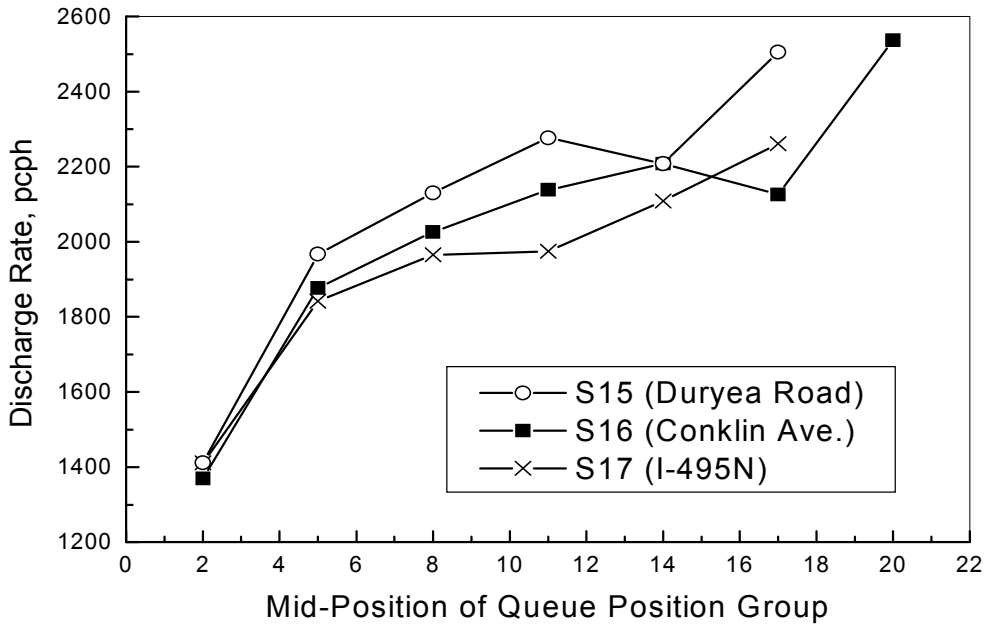

Figure 4. Relationships Between Queue Position and Discharge Rate of Straight-through Lanes in U.S.A.

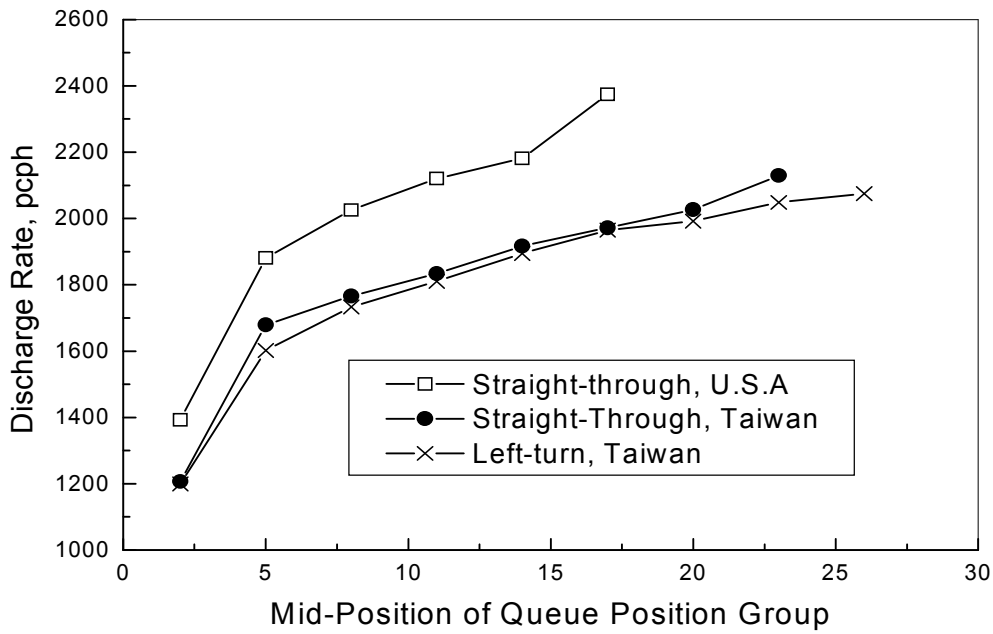

Figure 5. Aggregated Relationships Between Queue Position and Discharge Rate

If only those queue position groups that have at least 20 samples are used for comparison at a 5% level of significance, the difference between the queue discharge rate for queue positions 4~6 and that for queue positions 19~21 or 22~24 is statistically significant for all the study lanes analyzed. Figure 5 shows that, in aggregate, the discharge rate increases steadily. For straight-through lanes in Taiwan, the average discharge rate increases from 1,679 passenger cars/hour (pcph) in positions 4~6 to 2,129 pcph in positions 22~24. This represents an increase of 26.8%. For the left-turn lanes in Taiwan, the corresponding increase is from 1,602 pcph to 2,048 pcph (a 27.8% increase). The U.S. data show a 26.2% increase from 1,881 pcph in positions 4~6 to 2,374 pcph in positions 16~18.

4. IMPLICATIONS

Given that a steady maximum discharge rate exists, the area in Figure 1 under the assumed queue discharge curve represents the number of queuing vehicles that can be discharged in one signal cycle. This area can be approximated by a rectangle defined by the saturation flow and the effective green interval. Consequently, the capacity of a lane or lane group at a signalized intersection is traditionally estimated as follows:

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

where

c = capacity of a lane or lane group (vehicles/h or vph),
S = saturation flow (vph),
g = effective green,
C = cycle length (s),
G = green interval (s),
Y = signal change (or intergreen) interval (s), and
L = lost time (s),

For most applications of Equation 1, saturation flow is estimated rather than measured in the field. The procedure of estimating saturation flow can be tedious and, for opposed turning movements, complicated (Transportation Research Board 2000). In addition, it is also necessary to estimate the lost time due to starting delays and the transition of right-of-way from one signal phase to another. The fact that observed queue discharge characteristics differ from the traditional saturation flow model makes the correct use of Equation 1 more problematic. The related issues are discussed below.

4.1 Definition of Saturation Flow

Saturation flow is defined as a steady maximum queue discharge flow. But the field data described previously shows that queue discharge rate rarely reaches a steady maximum. Under the circumstances, saturation flow can be defined arbitrarily and this can cause inconsistent applications of Equation 1. Therefore the traditional definition of saturation flow is neither meaningful nor practical. For practical applications, an alternative is to define the saturation flow S in Equation 1 as the average queue discharge rate after a specified number of vehicles have entered the intersection. Refer to Figure 5. The discharge rate rises rapidly

between the first two queue position groups that cover the first six queuing vehicles. Beyond the second queue position group (positions 4, 5 and 6), the rate of increase in queue discharge rate becomes much milder and more or less steady. Therefore, one may use the average discharge rate after the sixth queuing vehicles to represent the saturation flow S in Equation 1. How the saturation flow S is defined and measured will affect the selection and the modeling of lost time L that is also needed in Equation 1. Therefore, it is essential for researchers and practicing professionals to adhere to the same definition.

4.2 Determination of Lost Time

Regardless of how the saturation flow S is defined in Equation 1, the lost time L that should be used is one that would yield accurate estimate of the capacity of a lane or lane group. In other words, the lost time should be determined from the following relationship:

$$Q = \frac{S(G + Y - L)}{3600} \tag{2}$$

where Q represents the average maximum observed number of queuing vehicles that can be discharged per signal phase.

Based on Equation 2, the lost time should be:

$$L = G + Y - \frac{3600Q}{S} \tag{3}$$

To provide an insight into the nature of the required lost time, the field data were used in Equation 3 to calibrate the values of lost time L under the following conditions:

(1) Based on the *U.S. HCM*, the saturation flow S is determined as the average discharge rate of the queuing vehicles behind the fourth vehicle.
(2) Green interval varies and signal change interval is 4.5 s.
(3) On average, 2 cars in a queue enter the intersection during signal change interval.

The calibrated values of lost time are shown in Figure 6. Each curve in this figure represents the variation of lost time for each study lane as the green interval is changed. Figure 6 shows that the correct lost time to use varies with the green interval being analyzed and lane location. For a given lane, the variation of lost time with green interval can exceed 3 s. The variation from one location to another can be as much as 6 s. There are currently no reliable models to relate lost time to green interval and to account for the effects of lane location. In fact, transportation professionals often have to use default lost time for capacity estimation. The *U.S. HCM* suggests that, in the absence of reliable information, a default lost time of 4 s be used. The use of default lost time can lead to significant errors in subsequent estimation of lane capacity and vehicle delays, as discussed below.

4.3 Estimation Error

It has been noted that current methodologies in estimating saturation flow has a standard error of about 10% (Tarko and Tracz 2000). This implies that even there are no other errors

in using Equation 1, the capacities estimated from Equation 1 will on average have an error of about 10%. The use of default lost time, or any other lost time that is not calibrated from field data, can increase the estimation error. For example, if the values of the calibrated lost time shown in Figure 6 are replaced by a default lost time of 4 s as suggested in the *U.S. HCM,* the capacities of the study lanes as estimated from Equation 1 would have errors as shown in Figure 7. These errors, in conjunction with the errors in estimating saturation flow, can cause the errors in estimating capacities to far exceed 10%.

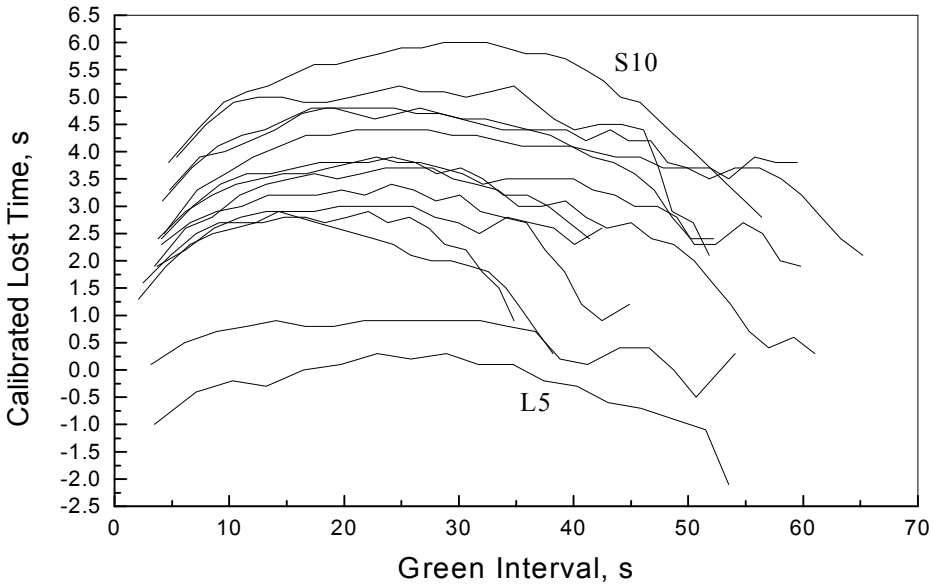

Figure 6. Lost Time Calibrated According to U.S. HCM's Definition of Saturation Flow

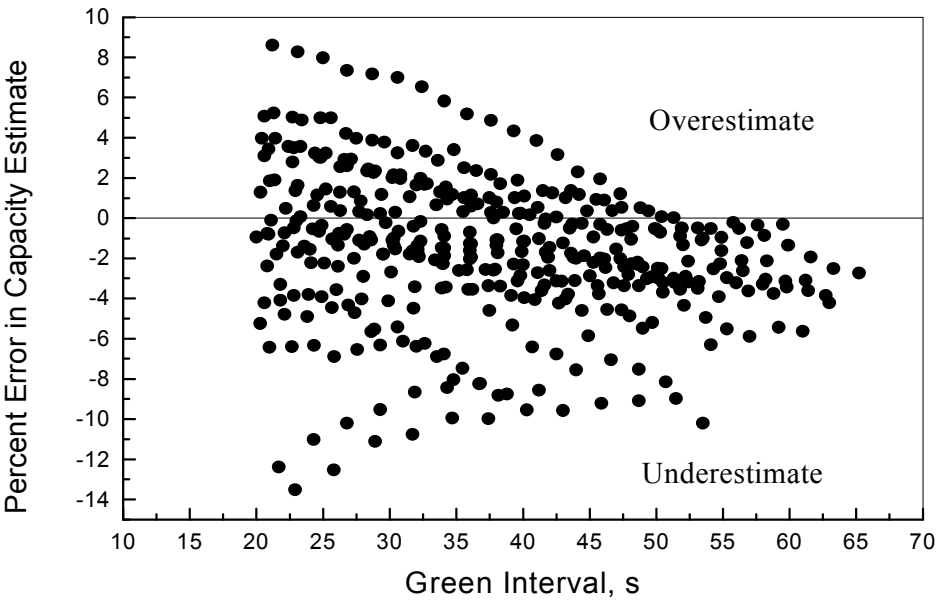

Figure 7. Capacity Estimation Errors Attributable to Use of 4 s Default Lost Time in U.S. HCM Procedure

Even more troublesome is that a small error in capacity estimate can propagate into a very large error in estimated delays. Figure 8 illustrates the seriousness of this problem when underestimated capacities are used in the following *U.S. HCM* model to estimate delays at isolated intersections that are controlled with pretimed signals:

$$d = \frac{0.5C(1-\frac{g}{C})^2}{1-\frac{g}{C}Min(1,x)} + 900[x-1+\sqrt{(x-1)^2+\frac{4x}{cT}}] \tag{4}$$

where

d = average control delay (s/veh),
C = cycle length (s),
g = effective green (s),
x = volume/capacity (v/c) ratio,
T = analysis period (h), and
c = capacity of lane or lane group being analyzed (vph).

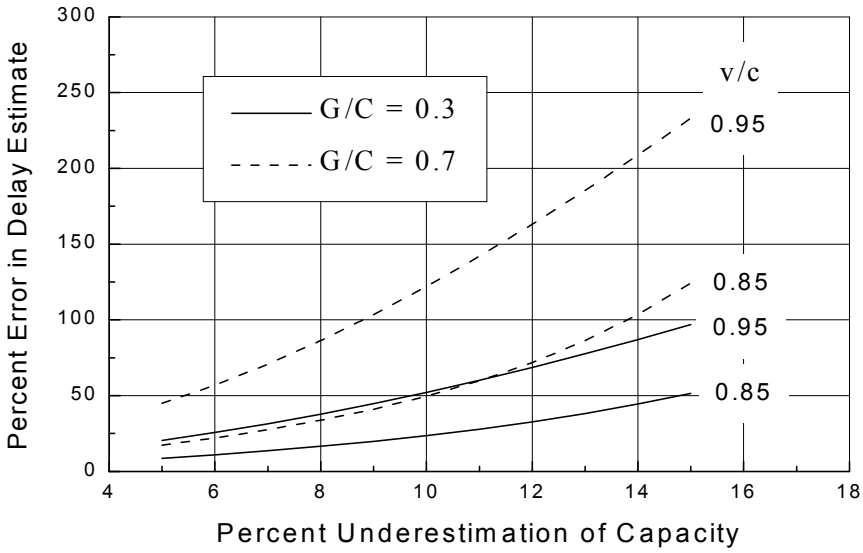

Figure 8. Impact of Error in Capacity Estimate on Delay Estimate

Figure 8 is based on the following conditions: (1) The intersection is controlled with a pretimed signal; (2) Vehicles arrive randomly; (3) The analysis period is 15 minutes and there are no queuing vehicles at the beginning of the analysis period; (4) The cycle length is 100 s; (5) The signal change interval is 5 s and the lost time is 4 s; (6) The volume/capacity (v/c) ratio is either 0.85 or 0.95; and (7) The saturation flow is 3,800 vph for two lanes. This figure shows that when capacities are underestimated by 10% to 15%, the delays estimated from Equation 4 would have errors in the range of 25% to 240%. As discussed previously, the combined error in estimating saturation flow rate and lost time is likely to cause an error of more than 10% in estimating capacities. Furthermore, it is not unusual for a lane to operate

with a v/c ratio of greater than 0.9 during peak hours. Therefore, the use of Equation 1 for capacity analysis will likely result in large errors in delay estimates. For these reasons, the drawbacks in using saturation flow and lost time for capacity analysis should discourage continued use of Equation 1.

5. RECOMMENDED METHOD FOR CAPACITY ESTIMATION

Fortunately, saturation flow is not an essential parameter for capacity estimation. A better approach is to estimate directly the number of queuing vehicles that can be discharged in the green interval and the signal change interval of each usable signal phase. Then the capacity of a lane or lane group can be determined as follows:

$$c = \frac{3600}{C}\sum_{i=1}^{m}(N_{gi} + N_{yi})F \tag{5}$$

where

c = capacity of a lane or lane group (vph),
C = cycle length (s),
i = i^{th} phase usable to the vehicles in the lane or lane group being analyzed,
m = number of phases available to the vehicles in the analysis lane or lane group,
N_{gi}= number of queuing vehicles that can be discharged in the green interval of signal phase i,
N_{yi}= number of queuing vehicles that can be discharged in the change interval of signal phase i, and
F = adjustment factor to account for variations not accounted for in estimating N_{gi} and N_{yi}.

The use of Equation 5 eliminates the need to use saturation flow and lost time and thus avoids the related estimation errors. Furthermore, the value of N_{gi} is strongly correlated with the green interval. Figure 9 shows an example of such a relationship based on the straight-through queue discharge characteristics at the rural and suburban intersections in Taiwan. The strong correlation between N_{gi} and the green interval implies that the modeling and estimation of N_{gi} can be accomplished with small errors. For example, for unopposed movements at the rural and suburban intersections listed in Table 1, N_{gi} can be estimated as:

$$N_{gi} = -4.97 + 0.443G + 0.001215G^2 + 1.176W \quad \text{For straight-through traffic} \tag{6}$$

$$N_{gi} = 1.4 + 0.426G - 1.186M \quad \text{For protected left turns} \tag{7}$$

where

N_g = average number of queuing passenger cars discharged in green interval per lane (pcph/lane),
G = green interval (s),
W = lane width (m), and
M = number of left-turn lanes

Equation 6 has a modest estimation error of 3.2% and Equation 7 has a slightly larger error of 5.6%. As for N_{yi}, the data collected in Taiwan show that its values are consistently between

2.0 and 2.2 cars per change interval. Therefore, it can be modeled and estimated accurately. It should be noted that Equations 6 and 7 are not full-fledged models. The IOT is in the process of developing an enhanced database for modeling N_{gi} and N_{yi}. The impact of heavy vehicles and other variables can be incorporated into the adjustment factor F in the form of $F=f_1f_2...f_n$, where f_i (i =1, 2, …, n) are individual adjustment factors. For example, if only the impact of heavy vehicles is to be accounted for, each heavy vehicle can be converted into a passenger car equivalent and the value of $F=f_1$ can be determined as $1/[1+p(E-1))$, where E is the passenger car equivalent and p is the proportion of heavy vehicles. For flat intersections in Taiwan, the passenger car equivalent of a heavy vehicle is about 2.1.

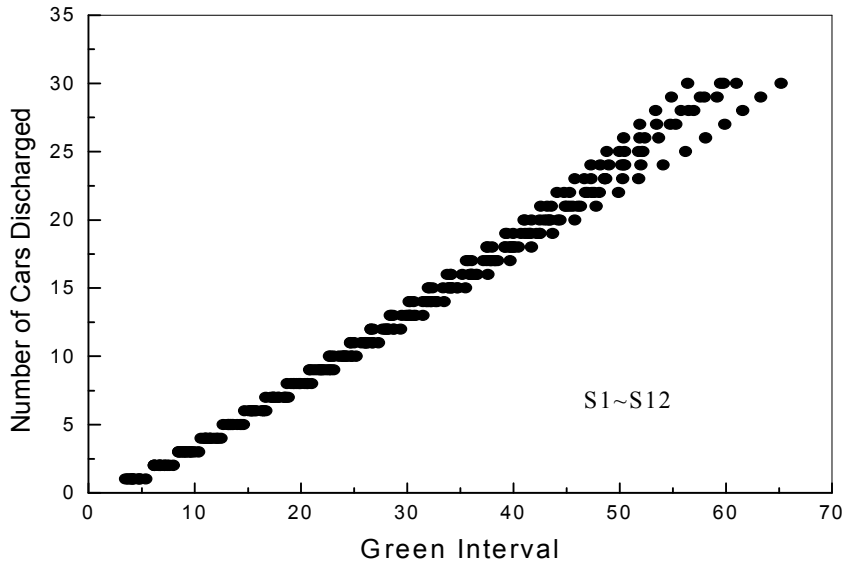

Figure 9. Number of Queuing Vehicles Discharged as a Function of Green Interval

6. CONCLUSIONS

The queue discharges in Taiwan and the United States exhibit similar characteristics; their discharge rates keep rising long after green onset. This contradicts the traditional saturation flow model that assumes queue discharge rate will quickly reach a steady maximum. Thus continued use of saturation flow for capacity analysis will create difficulties in defining, modeling, and estimating a meaningful saturation flow. In addition, the correct lost time that needs to be used in conjunction with a specified saturation flow varies widely with green interval and the lane location. This makes it difficult to model and estimate lost time. Consequently, traditional approach of capacity estimation is susceptible to large errors. Such errors can be significantly magnified when the estimated capacities are used to estimate vehicle delays. The errors in the estimated delays will in turn lead to poor decisions in the planning, design and operation of signalized intersection. A better alternative to the traditional approach is to estimate the number of queuing vehicles that can be discharged in each signal phase. The IOT in Taiwan is employing this approach to revise *Taiwan Area Highway Capacity Manual* (Institute of Transportation 2001).

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