VOL. 8, NO. 1, JANUARY 2013 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

www.arpnjournals.com

EFFECT OF DIRECTIONAL FLOW RATIO ON SIGNALISED INTERSECTION CONTROL STRATEGIES

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ABSTRACT

A signalised intersection phasing is a multiple arm movements whereas staging is a single arm movement of vehicles at the onset of inter-green light. The purpose of the paper is to determine the extent to which highway traffic directional ratio can be accountable for the effectiveness of signal timing. Phasing and staging would be treated as mutual exclusive movements under varying directional traffic loading. Based on the hypothesis that percentage of directional split would influence traffic signal optimum performance and associated delays, directional split impact studies were carried out in Skudai town, Johor, Malaysia. Major roadway delays for traffic flows with 20/80; 30/70; 40/60; 50/50 incremental directional ratio were analyzed. Results show that phasing signal settings are best suited to 40/60 and 50/50 directional flow and staging for 20/80 and 30/70 directional traffic flow. The paper concluded that optimized signal setting based on phasing would be more effective in circumstances where the likelihood of 50/50 and 40/60 directional split are more likely. The same cannot be said of 70/30 or 80/20 directional split.

Keywords: signalized intersection, phase, stage, delay, waiting time, level of service.

1. INTRODUCTION

Many studies have been carried out on signal timing and the relevance of delays and queues at intersections. Most signalized intersections in Malaysia operate fixed time signal setting [1]. With actuated signal, induction loops buried in the roadway stop-line, video, infrared or microwave detection system automatically adjusts timings relative to prevailing degree of saturation. An intersection without such detection system operates on fixed times (static). In any case, the paper is concerned with the effects that directional flow ratio will have on intersection control strategies. It can be argued that the choice of signal timing sequence is a function of traffic directional distribution as well as other competing demand and land use activities in the vicinity. The objectives are to determine cycle time, delays, and level of service for both phase and stage movements.

Based on the hypothesis that percentage of directional split would influence traffic signal optimal performance and associated delays, the remainder of the paper are divided into four sections. The immediate section is on literature review of key traffic signal parameters, while section 3 focuses on data collection. Analysis and findings are discussed in section 4 and conclusions drawn in section 5.

2. LITERATURE REVIEW

Traffic signal has its share of misfortune at the onset of development, from the first explosion 1868 to that of 1932 in England. Notwithstanding traffic signals at highway intersections are now fully computerized and automated. Traffic signals are the traffic controlling devices which used to enhance or promote orderly movements of vehicles or pedestrian to avoid excessive delay to traffic. They operate in a cyclic manner made up of stages with each stage permitting non-conflicting vehicle movements. Stages can be fused into many combinations of phases so that gains can be made in the individual inter-greens and efficiency realized.

There are two common strategies in order to run a signalized intersection; *Staging* and *Phasing* modes. The paper argues that the choice of strategy to be deployed hinges firmly on road directional flow ratio. Road traffic directional flow ratio can be 50/50; 60/40; 70/30; 80/20; 90/10 or 100/0(one way). In urban and city center areas, 50/50 and 60/40 directional splits are common whereas in sub-urban and maybe70/30 and 80/20 directional split in rural areas.

In the paper, *staging* is taken as vehicle discharge per arm regardless of regardless of the direction of the movements of the vehicle. Whereas phasing means movement of vehicles from more than one arm with the overall objective of minimizing conflicts and avoidable delays. Staging and phasing as applied in the paper is illustrated below in Table-1.

Table-1. Stages (Left hand drive).

Stage	Staging	Phasing
1	ALL (S, R, L)	NS; SS; NL; SL
2	ALL (S, R, L)	NR; SR
3	ALL (S, R, L)	ES; WS; EL; WL
4	ALL (S, R, L)	WR; ER

Note: S = straight; L = Left; R = right

2.1. Cycle time of signalized intersections

Control strategy is usually achieved by vehicle actuation, integral time switch as well as directional flow ratio. Since signal settings are based on fixed proportional distribution of effective green per cycle time, it follows

ISSN 1819-6608

ARPN Journal of Engineering and Applied Sciences

that effectiveness and efficiency may be achieved with direction flow ratio. Where directional flow ratio is tied to cycle time optimum intersection performance may be achieved. Cycle time itself is dependent on saturation flow for proportional distribution of signal timings [2]. A small change in the saturation flow value may result in a relatively large change in the calculated cycle time and the duration of the necessary green intervals [3]. 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. There are several methods to determine the cycle time, for example, The British Transport and Road Research Laboratory (TRRL) as shown in Road research technical paper 39 is the most common used cycle time equation. C_o or cycle time is determined using:

$$C_o = \frac{1.5L+5}{1-\sum y\max} \tag{1}$$

Where

L = total lost time per cycle; Y = sum of y values

The effective green time for a given movement or phase is calculated as:

$$g = G+Y+AR- t_L \tag{2}$$

Where

g = effective green (s)
G = actual green (s)
Y = amber (s)
R = red (s)
t_L = total lost time per cycle (s)

2.2. Delay and waiting time

The main parameter used to optimize traffic signal is delay, keep in mind that once traffic signal is introduced at an intersection, priority flow enjoy exclusively by vehicles on the major road is lost. Delays in intersections could be stopped time delay, when vehicle are stopped before the traffic light, approach delay which is the lost time during accelerating and decelerating of vehicles and also travel time delay which is the difference in time the vehicle clears the intersection at desired speed without stopping and when it stopped before a red light and then cross the intersection. There are several models that can be used to compute delay in intersections, and again the most common one is developed by TRRL which is given below. In this equation first terms accounts for uniform delay assuming uniform arrivals and second term accounts for incremental or random delay.

$$d = \left\{\frac{0.5C\,(1-u)^2}{1-ux}\right\} + \left\{900Tx\left[(x-1) + \sqrt{(x-1)^2 + m\frac{x-x_0}{QT}}\right]\right\} \tag{3}$$

Where

G = effective green
C = cycle time
u = (g/C)
T = flow
Q = capacity
m, n = calibration parameters, and
x_0 = the degree of saturation

There is no need to develop a new delay model. Delay at fixed-time controlled intersections has been a study of subject for many years. Several mathematical expressions have been derived to represent the so-called random delay component, the delay caused by the stochastic character of arriving traffic. Heydecker *et al.*, [4] presented a linear expression of delay as:

$$E(W) = \frac{t_r}{t_c\left(1-\frac{q}{s}\right)}\left[\frac{E(Q_0)}{q}x\frac{t_r+1}{2}\right] \tag{4}$$

Where

E (W) = expectation value of delay (s)
t_c = signal cycle
t_r = length of the effective red time
q = arrival flow rate
s = saturation flow
E (Q_0) = expectation value of overflow queue length

The assumption used by some authors that the queue should be represented by a step function appears to be superfluous. The stepwise character of the delay is transformed to a smooth character of the expected delay, linearly increasing in the red-phase and the first part of the green phase [5]. The expectation value of the queue in the green phase shows a nonlinear character as soon as the tail of the probability distribution comes close to zero. This phenomenon causes the overflow delay. However, Webster [6] presented delay model where the first term is analytical derivation of uniform delay, while the second term is a characterization of stochastic delay derived analytically assuming Poisson distribution. The last term has been introduced to reduce the discrepancy with results observed from simulation data. The formula has been simplified as:

$$d = \frac{9}{10}\frac{[C(1-\sim)^2}{[2(1-\sim x)} + \frac{x^2]}{2q(1-x)]} \tag{5}$$

Where

d = average delay per vehicle
c = cycle length, sec
q = flow, vehicles/sec
~ = effective green proportion of the cycle (g/c)
x = degree of saturation, (q/s)

Highway Capacity Manual [7] measures traffic performance of a signalised intersection by computing the expected delay per vehicle and decomposing it into 3 terms:

$$W = W_1 PF + W_2 + W_3 \tag{6}$$

Where

W_1 = uniform stopped delay/vehicle (s/veh)
W_2 = incremental stopped delay (s/veh)
W_3 = initial queue
PF = progression factor to account for signal coordination

The first two delay components are given by the following formulas:

$$W_1 = 0.5t_c \frac{\left(1-\left[\frac{t_g}{t_c}\right]\right)^2}{1-min(1,x)\frac{t_g}{t_c}} \tag{7}$$

$$W_2 = 900T\left\{(x-1)^2 + \sqrt{(x-1)^2} + \frac{\sqrt{8kI_f x}}{cT}\right\} \tag{8}$$

Where 1 is for fixed time, 0.5 for semi-actuated and somewhere between 0.04 and 0.5 for actuated. If is the filtering adjustment factor; accordingly, this formula assumes the queue length to be constant and finite if x <1 while it behaves according to the linear deterministic function for x >1. The third component is computed by specifying the parameters of the formula:

$$W_3 = \frac{1800\ Q\ (0)(1+u)t}{c.T} \tag{9}$$

2.3. Saturation flow and capacity

Saturation flow is the most important single parameter in the capacity analysis of signalized intersections. It is a measure of the maximum rate of flow and it is used extensively in junction design and control applications. Besides, saturation flow is the maximum constant departure rate of a queue from the stop line of an approach lane during the green period. Saturation flow can be defined as number of vehicles that would pass through the intersection during that hour is the saturation flow rate. In Malaysia Arahan Teknik Jalan 13/87 [1], saturation flow is defined as the maximum flow, expressed as equivalent passenger cars that can cross the stop line of the approach where there is a continuous green signal indication and a continuous queue of vehicles on the approach.

Based on US Federal Highway Administration (FHWA) [7], saturation flow is the equivalent hourly rate at which vehicles can traverse an intersection approach under prevailing conditions, assuming a constant green indication at all time and no loss time, in vehicles per hour or vehicles per hour per lane. Using one hour to define saturation flow is a bit confusing because saturation cannot be observed continuously for one hour at any signalized intersection. Saturation flow at signalized intersection is observed per effective green time in seconds; to suggest that hourly multiplier can be employed on such flows would be misleading. Geometric factors as well as other externalities do affect saturation flow. TRRL indicated that opposed and unopposed traffic streams rules be used when evaluating saturation flow [8]. In the paper, however, the headway simple technique was used since the sites have standalone signal settings where saturation:

$$s = 3600 / h \tag{10}$$

Where

s = saturation flow rate in veh/h,
h = saturation headway in s/veh, and
3600 = number of seconds per hour

2.4. Level of service at signalized intersection

According to the highway capacity manual (HCM 2000) level of service as a measure for qualitative service of roadway can be defined using delay incurred by motorist at signalized intersection. The average stopping delay per vehicle for 15 minute analysis period is the criteria for the selection of level of service. In the paper, the criteria established in HCM 2000 were used for the purpose of determining the effectiveness of road service for the signalized intersections under observation.

Table-1 below shows the HCM 2000 derivation of level of service. It should be noted that Level of service A to F is also described in qualitative measures.

Table-1. Level of service.

Level of service	**Delay per vehicle** (sec)
A	< 10
B	10 – 20
C	20 – 35
D	35 – 55
E	55 – 80
F	> 80

Source: HCM 2010

Capacity of signalized intersections is based on the concept of saturation flow and saturation flow rates. Saturation flow rate is defined as the maximum rate of flow that can pass through a given lane group under prevailing traffic and roadway conditions, assuming that the lane group had 100% of real time available as effective green time and is expressed in units of vehicles per hour of effective green time (vphg). The flow ratio is defined as the ratio of actual or projected flow rate for the lane group, v, to the saturation flow rate, s. The flow ratio is (v/s) i for lane group i. The capacity of the lane group is:

VOL. 8, NO. 1, JANUARY 2013 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

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$$c = s \times (g/C) \tag{11}$$

Where

c = capacity of lane group (veh/h)
s = saturation flow rate in (veh/h)
g = effective green time (s), and
C = cycle length (s)

3. DATA COLLECTION

The study deals primarily with delays associated with directional traffic flow ratio. In order to compute delay, degree of saturation and by extension saturation flows must be computed. By applying the headway technique illustrated below in Figure-1, saturation flow can be estimated using equation (10). Delay computed with equation (5) and effective green relied on equations (2 and 3) for estimation. Actual green and red were hand timed. Geometric information was culled from Google earth and directly measured for acceptance. Automatic counters were installed at the entry arms for six weeks during daylight and dry weather conditions. Although headway, speed, volume, vehicle types and gap acceptance information were readily supplied, the automatic counters were useful pointer to weekly variations in traffic volumes, surveys were carried at peak hours as indicated by the automatic counters. The data collection for the saturation flow were 100 samples for each approach arm where the time for the first three cars was recorded as T3 and the thirteenth car time was also recorded as T13 in other to calculate the lost time, headway and saturation flow.

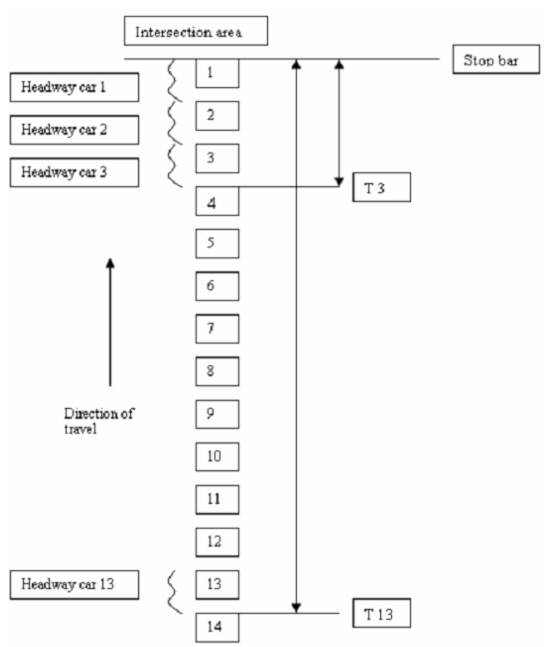

Figure-1. Typical set-up of headway survey.

Saturation headway is the headway of the vehicles in a "stable moving platoon" passing through an effective green light. A stable moving platoon is a group of vehicles that are traveling, but not really moving in relation to each other (i.e., all going the same speed). The headway of the first four vehicles leaving an intersection after a red light will have a higher value so the saturation headway will not be realized until the 4^{th} or 5^{th} queued vehicle leaves the intersection. So, if every vehicle requires a time equal to the saturation headway (h), in seconds, to be serviced at a signalized intersection, then the maximum number of vehicles that can be serviced in an hour of green is given by the equation (10). Headway (H) is defined as the time between two successive vehicles in a traffic lane as they pass a point (stop bar) on the roadway measured from front bumper to front bumper, in seconds to calculate the headway as following (H).

$$H = T13 - T3 / 10 \tag{12}$$

Where

T13= the time for first 13 cars to clear Stop Bar
T3= time for first 3 cars to clear Stop Bar

Saturation flow rate is computed for each of the lane groups established for the analysis. A saturation flow rate is determined from field measurement by the headway. Saturation flow rate (S) is calculated as following:

$$S = 3600/H$$

Loss time at the field, the time of first three cars (T3) will be recorded, next equation shows it:

$$\text{Loss time} = T3- (3\times H) \tag{13}$$

In order to determine the existing delay experienced by drivers, actual green time of the existing setting was needed. Therefore during each session actual green tome for each arm was measured, and an average value is considered as the actual green time.

Traffic signal parameters were collected at two sites; site 1 Johor Bahru (JB) to Skudai; site 2 is Skudai to JB. The sites are located along a major federal route in Taman Taratai, Skudai, Malaysia [9].

4. ANALYSIS AND FINDINGS

As shown below in Tables 2 and 3, traffic volumes for directional flow for the sites are classified according to the vehicle turning or straight movements. Traffic volumes are also placed in different vehicle types so that appropriate passenger car equivalent values could be used to convert vehicles per hour into *pce* per hour. From the Tables it can be seen that passenger cars (*pc*) are the dominant vehicle type. They account for 67 per cent of traffic flow from the north approach, 56 per cent from the south approach, 65 per cent from the east and 52 per cent from the west approach in Table-2.

VOL. 8, NO. 1, JANUARY 2013 ISSN 1819-6608
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Table-2. Traffic volume direction JB to Skudai.

Arm	App	PC	MC	LT	MT	HT	B/C	Total
N	↑	31	27	6	5	6	12	87
	→	709	186	76	25	24	45	1065
	↓	49	11	8	16	18	15	117
S	↑	82	7	18	2.5	0	18	127
	→	472	86	36	50	63	132	840
	↓	142	20	12	22	9	9	215
E	↑	74	28	10	12	18	6	149
	→	121	12	22	2	3	24	185
	↓	52	28	6	22	3	6	118
W	↑	81	23	12	25	6	12	159
	→	65	19	14	15	9	3	125
	↓	395	33	6	20	12	18	484

Note: PC = passenger car, MC = motorcycles, LT = light transit, MT = medium transit, HT = heavy truck, B/C = bus/coach

In Table-3, passenger cars account for 73 per cent of traffic flow from the north approach arm, 61 per cent from the south approach, 53 per cent from the east and 79 per cent from the west approach.

Table-3. Traffic volume direction Skudai to JB.

Arm	App	PC	MC	LT	MT	HT	B/C	Total
N	↑	71	15	8	24	12	28	158
	→	664	87	56	30	36	36	909
	↓	153	27	36	10	12	24	262
S	↑	144	48	24	0	0	24	240
	→	744	186	48	10	60	168	1216
	↓	180	15	8	0	0	12	215
E	↑	52	12	16	0	0	24	104
	→	120	66	16	10	0	12	224
	↓	232	69	16	20	24	0	361
W	↑	328	9	56	0	0	0	393
	→	152	9	0	30	0	0	191
	↓	200	18	20	0	0	0	238

Note: PC = passenger car, MC = motorcycles, LT = light transit, MT = medium transit, HT = heavy truck, B/C = bus/coach

Saturation flow, headway and loss time for directional flow were computed and shown below in Tables 4 and 5. For site 1, the average headway is 1.9s, saturation flow is 1895pce/hr and loss time per arm is approximately 2s. For site 2, the average headway is 1.92s, saturation flow is 1875pce/hr and loss time per arm is approximately 2.04s. Cycle time was computed using equation (1). Cycle time for the existing traffic condition where the directional demand flow ratio is 70/30 is shown in Table-6.

Table-4. Saturation flow rate and loss time.

No.	Direction JB to Skudai				
	T3	T13	H (s)	S (pce/hr)	Loss time (s)
1	7.40	22.63	1.52	2364	2.83
2	7.16	25.23	1.81	1992	1.74
3	9.23	32.34	2.31	1557	2.30
4	8.11	28.58	2.05	1759	1.97
5	7.02	25.00	1.80	2002	1.63
6	9.01	28.58	1.96	1839	3.14
7	8.57	28.80	2.02	1779	2.50
8	7.26	28.24	2.10	1715	0.97
9	5.94	23.01	1.71	2108	0.82
10	9.41	28.92	1.95	1845	3.557
11	7.11	23.01	1.59	2264	2.34
12	8.47	27.65	1.92	1876	2.72
13	7.31	24.70	1.74	2070	2.09
14	8.11	27.07	1.90	1898	2.42
15	9.62	25.99	1.64	2199	4.71
16	8.61	33.02	2.44	1475	1.29
17	7.51	25.42	1.79	2010	2.14
18	6.41	26.11	1.97	1827	0.50
19	7.09	24.21	1.71	2102	1.95
20	7.14	22.92	1.58	2281	2.41
21	8.30	29.28	2.10	1715	1.84
22	8.43	32.14	2.37	1518	1.32
23	7.18	24.56	1.74	2071	1.97
24	9.62	29.28	1.97	1831	3.72
25	5.26	22.07	1.68	2142	0.22
26	7.22	24.56	1.73	2076	2.02
27	6.24	25.62	1.94	1857	0.43
28	7.28	26.70	1.94	1854	1.45
29	7.32	28.92	2.16	1666	0.84
30	7.31	25.99	1.87	1927	1.71
Ave	**7.7**	**27**	**1.90**	**1895**	**2.0**

VOL. 8, NO. 1, JANUARY 2013 ISSN 1819-6608
ARPN Journal of Engineering and Applied Sciences

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Table-5. Saturation flow rate and loss time.

No.	Direction Skudai to JB				
	T3	**T13**	**H** (s)	**S** (pce/hr)	**Loss time** (s)
1	7.20	24.63	1.74	2065	1.97
2	8.16	25.23	1.71	2109	3.04
3	8.23	33.34	2.51	1434	0.70
4	8.11	28.58	2.05	1759	1.97
5	7.02	25.00	1.80	2002	1.63
6	9.01	28.58	1.96	1839	3.14
7	8.57	28.8	2.02	1779	2.50
8	7.26	28.24	2.10	1715	0.97
9	9.94	23.01	1.31	2754	6.02
10	8.41	26.92	1.85	1944	2.86
11	7.11	23.01	1.59	2264	2.34
12	8.47	27.65	1.92	1876	2.72
13	7.31	24.70	1.74	2070	2.09
14	8.11	27.07	1.90	1898	2.42
15	8.52	29.99	2.15	1677	2.08
16	8.61	33.02	2.44	1475	1.29
17	7.51	25.42	1.79	2010	2.14
18	6.41	26.11	1.97	1827	0.50
19	7.09	24.21	1.71	2102	1.95
20	7.14	26.92	1.98	1820	1.21
21	8.30	29.28	2.10	1715	1.84
22	8.43	32.14	2.37	1518	1.32
23	7.18	24.56	1.74	2071	1.97
24	9.62	29.28	1.97	1831	3.72
25	7.26	27.07	1.98	1817	1.32
26	7.22	24.56	1.73	2076	2.02
27	6.24	25.62	1.94	1857	0.43
28	7.28	26.70	1.94	1854	1.45
29	7.32	28.92	2.16	1666	0.84
30	7.61	27.99	2.04	1766	1.50
Ave	**7.8**	**27**	**1.92**	**1875**	**2.04**

Table-6. Optimum stage setting at 70/30.

Stage	Staging	Co = 218s	Co = 120s
1	North (S, R, ~~L~~)	Gn_{eff} 65s	Gn_{eff} 37s
2	South (S, R, ~~L~~)	Gn_{eff} 51s	Gn_{eff} 29s
3	East (S, R, ~~L~~)	Gn_{eff} 22s	Gn_{eff} 12s
4	West (S, R, ~~L~~)	Gn_{eff} 63s	Gn_{eff} 35s

Note; S = straight; ~~L~~ = Left; R = right; Gn_{eff} = effective green

Directional demand traffic flow was adjusted to 60/40 and cycle time was recomputed with results shown below in Table-7. Once the directional flow was changed to 60/40, optimum cycle time was reduced from 218s to 90s; a cycle time reduction of about 59%.

Table-7. Optimum stage setting at 60/40.

Stage	Phasing	Co = 90s
1	NS; SS; ~~NL~~; ~~SL~~	Gn_{eff} 32s
2	NR; SR	Gn_{eff} 19s
3	ES; WS; ~~EL~~; ~~WL~~	Gn_{eff} 17s
4	WR; ER	Gn_{eff} 12s

Note; S = straight; ~~L~~ = Left; R = right; Gn_{eff} = Effective green

It is usual in practice to limit the maximum value of cycle time to 120s and in the paper the maximum value of 218s was applied. The paper assumed that the outcome will not be affected either way. Note that north and south approach is the major road with dominant traffic flow. It should also be noted that all left turn movements were allowed during all phases, since they do not consume any time on the signal setting and have less interruption to traffic flow. Delays and Level of service were computed for 60/40 phasing and 70/30 staging as well as 70/30 phasing and the results are shown in Tables 8 and 9. Level of service was not improved by changing from stage to phase movement where the directional flow ratio is the same. Therefore is can be suggested that improved delay performance may not be achieved by simply rearranging vehicle discharge movements only. For improvement in delay to be realized consideration must be given to directional traffic flow ratio. Where the directional traffic flow is 70/30 and above stage movement traffic control strategies may be more effective.

Table-8. Estimated delays in seconds JB-Skudai.

Approach	Phasing 70/30	Phasing 60/40	Staging 70/30
N	62 (F)	32 (C)	78 (C)
E	70 (F)	36 (D)	85 (F)
S	90 (F)	47 (D)	127 (F)
W	67 (F)	45 (D)	95 (D)

Note: symbol in parentheses are level of service

Table-9. Estimated delays in seconds Skudai-JB.

Approach	Phasing 70/30	Phasing 60/40	Staging 70/30
N	79 (F)	32 (C)	93 (F)
E	71 (F)	28 (C)	83 (F)
S	103 (F)	47 (D)	113 (F)
W	103 (F)	54 (D)	126 (F)

Note: symbol in parentheses are level of service

In sum up, the delay computed using the existing setting for intersection was substantial indicating that the signalised intersection is performing poorly and in dire need of recalibrating. From the empirical survey data and ensuing analysis, directional flows for the sites are in the region of 55/45 or if you like 60/40. The paper has shown that phasing would be better suited to the sites surveyed.

5. CONCLUSIONS

The aim of this study is to explore the impact of directional flow ratio on signalized intersection performance. Based on findings and discussion in the previous section, the paper concluded that:

- Delay is perhaps the most significant performance measurement.
- Cycle time for stage movement is higher than phase movement.
- Higher delays for road users may result from stage signal setting in circumstances where the directional flow ratio 60/40 or 50/50.
- Stage or phase movement strategies are based mainly on directional and necessarily on peak hour flow.
- The hypothesis that directional flow ratio has effect on signalized intersection performance is valid.

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