A study into urban roadworks with shuttle-lane operation

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A STUDY INTO URBAN ROADWORKS WITH SHUTTLE - LANE OPERATION

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ABSTRACT

In urban areas where roadworks are required, single lane shuttle operation is applied, especially where there is limited road space. There are operational problems relating to the site such as site geometry, visibility, length of roadworks zone, position of signs with other traffic control devices and signal timing. Other problems are mainly related to drivers' behaviour and their compliance with traffic controls on site.

The reduced road width caused by the works will interrupt the free flow of traffic and it can also add to the risks to road users. In addition, shuttle operation may introduce long queues and increase delays especially during peak periods.

There is a need to identify those parameters and behaviours which might influence traffic performance in terms of safety and capacity. An investigation of four roadworks sites in urban roadworks within the Greater Manchester area was undertaken for this purpose. Parameters included in the examination were position of the STOP sign, signal timing, weather conditions, time headway, vehicle speed and percentages of heavy goods vehicles (HGV) in the traffic stream. Statistical analysis and comparisons between sites were conducted. Other factors related to the operation of the shuttle-lane were provided based on site observations.

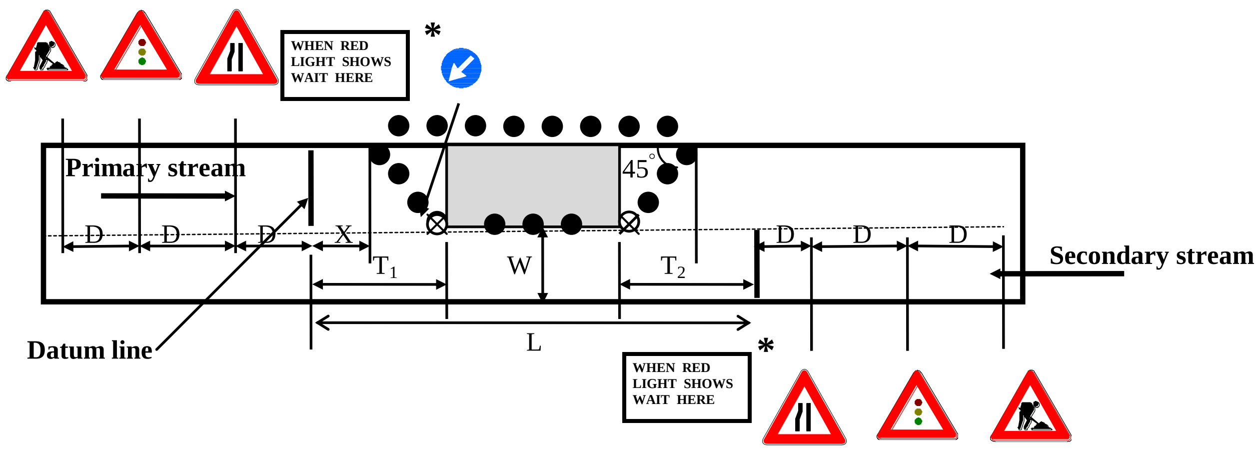
INTRODUCTION

Maintenance works on single carriageway roads in urban areas are normally carried out by closing one lane in one direction leaving the other lane for use by both directions. This type of operation is referred to as single lane shuttle operation. This one-way traffic operation requires an 'all-red' interval of sufficient duration for traffic to clear the shuttle-lane at the minimum operating speed in the work area (OECD, 1989).

During shuttle operation under congested conditions, severe delays and long queues may reach to the point of causing blockage to side roads in the vicinity of the roadworks site due to reduction in road capacity. Drivers' behaviour towards, for example, non-compliance with traffic signs and signals may be increased by road congestion during temporary roadworks which may lead drivers to taking unnecessary risks. This in turn could have adverse effects on both capacity and safety.

Choosing the type of control for shuttle operation depends on many factors such as traffic flow, length of controlled area, visibility through the work area and duration of work (Freeman Fox and Associates, 1973). The capacity (maximum two-way flow) of the shuttle-lane controlled by temporary traffic signal depends on site length. Capacity can reach 1600 veh/hr for site lengths of up to 50 metres and about 1100 veh/hr for 300 metres site lengths (Summersgill, 1981).

This paper aims to examine the main parameters which could influence shuttle operation in terms of maximising capacity and improving safety. Figure 1 shows a typical site layout with the locations of the main traffic control devices as normally used in practice.



D= 25m for average speed of 30 mph (50 km/h) (not to scale)

L= site length in metres

T_1= position of the stop sign from the traffic signals

T_2= 20m depending on site condition

X= 5 to15m depending on width of obstruction

W= width of shuttle-lane

* stop sign

● reflective cones

⊗ traffic signals

Figure 1. Typical site layout and location of signs

Field surveys from different sites on urban roadworks were carried out to examine the following (for further details on site selection and data collection see Samoail, 1997):

⇒ time headways for both primary and secondary streams,

⇒ perception time of drivers on crossing the stop line after being in a queue and time required for queue dispersion, and

⇒ observations of driver’s non-compliance with traffic signs and signals.

DATA COLLECTION METHODOLOGY

Data were collected for at least two hours from four roadworks sites in urban areas as shown in Table 1. This was done during day light for both peak and off-peak periods. Traffic counts, time headways and signal timings were recorded using video cameras from these sites which were controlled by temporary traffic signals. Half of these signals were vehicle actuated (VA) and the other half were fixed timing (FT). The data were then abstracted from video playbacks using an event recorder. Average speed data were obtained manually by measuring the time taken by a sample of vehicles to travel between the two temporary traffic signals using a stopwatch. Queue lengths, physical characteristics and drivers behaviour were observed on sites.

Table 1. Summary of the physical characteristics for the four sites

Parameters	Site 1		Site 2		Site 3		Site 4	
	P	S	P	S	P	S	P	S
Date of Survey	Sunday 28/2/96		Friday 26/7/96		Monday 16/9/96		Thursday 24/4/97	
Period of Survey (Hours)	13:35 - 15:45		15:18 - 18:13		11:10 - 14:10		8:32 - 10:47	
Weather	Snowy		Cloudy		Sunny		Sunny	
Site Length (metre)	72.00		33.80		137.20		46.00	
Width of Hazard (metre)	4.30	4.30	5.30	5.30	3.40	3.40	5.20	5.20
W (metre)	3.90	3.90	5.30	5.30	3.60	3.60	5.20	5.20
T_1 (metre)	12.00		10.00		16.70		26.00	

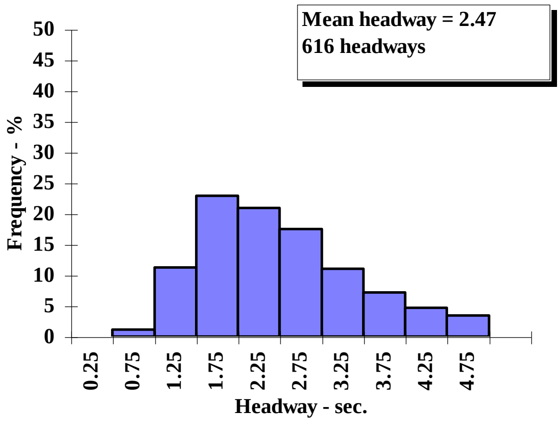
Note: P = primary stream, S = secondary stream

ANALYSIS OF DATA

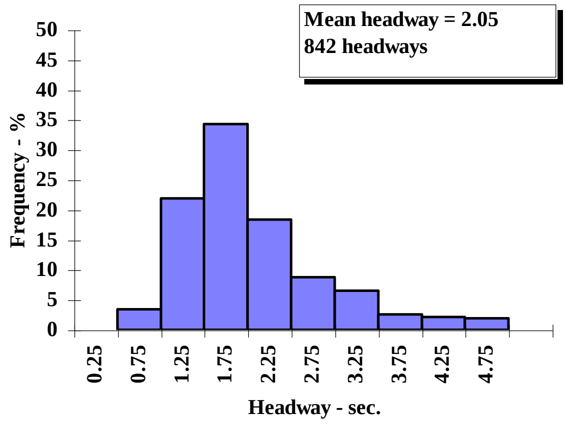
Time Headway

Time headway may be defined as the time difference between the arrival of successive vehicles at a reference point. Time headway data were obtained by recording the time between two successive vehicles in the same direction on touching the datum line. The time headway data was compiled for each direction, primary and secondary stream, separately. This allowed comparisons between both distributions and their respective means.

Headway Distribution. A maximum headway of 5 seconds was chosen for the analysis of the saturation flow for both primary and secondary streams for all sites. This was to ensure that vehicles were travelling in platoons. The total number of observed headways was 6317. Figure 2 shows typical headway distributions for the primary and secondary streams. Both are positively skewed. The secondary streams have higher percentage of time headways of less than 2 seconds compared with that of primary streams for all the sites (for more details see Samoail, 1997).



(a)



(b)

Figure 2. Typical headway distributions for time headways ≤ 5 seconds, (a) for primary stream and (b) for secondary stream

<u>Average Mean Time Headways</u>. Average mean headways between primary and secondary streams were compared to give an indication on their highest equivalent saturation flows. Mean time headways for each cycle were first calculated then an average value of these headways were obtained for all cycles for each site. The average mean headway for all the cycles for each direction was based on excluding the first vehicle from both directions to rule out the effect of site length. Results showed that the highest observed flows for the secondary stream were always greater than that of the primary stream for all sites as shown in Table 2.

Table 2. Typical comparison of μ_p and μ_s

		n	$\bar{x}$	σ^2	Z-Value	Critical	Saturation Flow (pcu/hr)
Site 1	P	88	2.58	0.2756			1410
	S	88	2.05	0.0748	8.44	1.96	1780
Site 2	P	129	2.42	0.2100			1630
	S	129	2.25	0.1595	3.39	1.96	1740
Site 3	P	108	2.73	0.3210			1400
	S	102	2.65	0.6250	0.84	1.96	1470
Site 4	P	101	2.13	0.0635			1750
	S	98	2.08	0.0655	1.38	1.96	1840

Note: P = primary stream, S = secondary stream, n = sample size (number of cycles during the survey); $\bar{x}$ = Average sample mean; σ^2 = sample estimate of population variance; Z = standardised normal distribution; μ_p = population average mean headway for primary stream; μ_s = population average mean headway for secondary stream.

Tests of significance were carried out, initially on the hypothesis that the average mean headway for all cycles in the primary stream did not differ from that in the secondary stream for all sites. Examination of primary and secondary streams average mean time headways at the 5% level of significance showed that the calculated values of Z (as shown in Table 2) were higher than the critical ones for Sites 1 and 2. The results could be attributed to the effect of the physical characteristics and geometric layout and the differences in mean speed and acceleration.

Vehicles travelling in the primary streams need more manoeuvring time to clear the taper (obstructed work) than those in the secondary stream. Also bends or curvatures in the path might result in increased average gaps between vehicles. The measured average speed through the shuttle-lane for the secondary stream was higher than that of the primary stream for all the sites. This tends to shorten the time gaps between vehicles which may result in a higher overall capacity.

<u>Headways Between Sites</u>. Another tests of significance were carried out to compare the average mean headway between different sites for both primary and secondary streams. Table 3 shows a typical comparison, for example, between Sites 1 and 4. A test of significance at the 5% level indicated that primary average mean time headway for Site 1 does differ from that of Site 4.

Table 3. Typical comparison of μ_{p1} and μ_{p4} between Sites 1 and 4

n_{p1}	n_{p4}	$\bar{x}_{p1}$	$\bar{x}_{p4}$	σ_{p1}^2	σ_{p4}^2	Z	Critical
88	101	2.58	2.13	0.2756	0.0635	7.34	1.96

Note: $\%HGV_{p1} = 2.22$, $\%HGV_{p4} = 4.05$

The differences between the two averages could be attributed to weather conditions, position of the warning stop sign from the signals (which is more than double the distance for Site 4 compared with that of Site 1) and the percentages of HGV's. To examine which of these elements had a major impact on the difference between the two averages, another test was carried out by taking two samples (each sample with {n} number of cycles) from both sites with similar or identical percentages of HGV's. Although the sample sizes were small, a test of significance indicated that the primary averages mean time headway for Site 1 did differ significantly from that of Site 4 at the 5% level of significance, as shown in Table 4. This could be attributed to the position of the warning stop signs at the primary stream (T_1). If T_1 is too small, it may create manoeuvring difficulties for vehicles in the primary streams (especially for HGVs) before entering into the shuttle-lane. However, a longer distance may reduce the effect of the tapered section. This may bring the operational conditions of the primary streams to be similar to those of the secondary streams. This needs further examination to optimise the most appropriate positioning which can provide higher capacity.

Table 4. Typical comparison of μ_{p1} and μ_{p4} between Sites 1 and 4

n_{p1}	n_{p4}	$\bar{x}_{p1}$	$\bar{x}_{p4}$	σ_{p1}^2	σ_{p4}^2	t	Critical
13	21	2.37	2.19	0.0493	0.0504	2.24	2.06

Note: $\%HGV_{p1} = 9.86$, $\%HGV_{p4} = 9.89$

<u>Other Parameters</u>. Another parameter which might influence driver's behaviour when entering the shuttle-lane is road surface conditions. On a wet surface, observations show a general increase in the time headway between vehicles which indicates that drivers enter the shuttle-lane more cautiously. This may cause a general decrease in the capacity of the shuttle-lane.

Perception Time

The effect of the time headways between vehicles in the primary stream on crossing the stop line after queuing at the traffic signals was also examined for the four sites. The number of vehicles passing through in each cycle depends to a large extent on these headways. Average values for these headways as obtained from the four roadwork sites are shown in Table 5. For comparison, typical values as reported by Briggs (1997) for signalised intersections are also included.

Table 5. Average time headways on moving off from traffic lights

Average time headway (sec)	1 Site 1	2 Site 2	3 Site 3	4 Site 4	5 Average of the four roadwork sites	6 Typical values at signalised intersections (Briggs, 1977)
for 1st vehicle	2.81	2.82	3.44	2.34	2.85	3.8
between 1st and 2nd vehicle	3.29	2.73	3.09	2.45	2.88	2.56
between 2nd and 3rd vehicle	2.69	2.52	2.57	2.43	2.55	2.25
between 3rd and 4th vehicle	2.28	2.29	2.54	2.29	2.35	2.10
between 4th and 5th vehicle	2.35	2.21	2.39	2.17	2.28	1.98
between 5th and 6th vehicle	2.35	2.21	2.39	2.05	2.25	1.93
Total	15.77	14.78	16.42	13.73	15.16	14.62

In general, there are differences in the average time headways between the four sites (Column 5) and those obtained at intersections (Column 6). The results showed that for the 6th vehicle in the queue these average headways gradually reduced to 2.25 seconds (Column 5) at roadwork sites compared with 1.93 seconds (Column 6) at traffic signals. The total average headway for all six vehicles for the four sites (15.16 seconds) is slightly higher than the total average at traffic signals (14.62 seconds). Average headways for the 1st vehicles are longer at traffic signals compared with those at roadwork sites. This increase in perception time may be attributed to drivers at intersections have to take extra care when entering the intersection by ensuring that their path is clear from approaching traffic from other arms compared with the shuttle-lane which has only one opposing direction.

Queue Dispersion

In designing temporary traffic signals for any shuttle-lane, it is essential to know the extent queues are likely to occur and the required time to discharge them. A queue can be defined as the condition when vehicles are either stationary at the selected point or are moving past that point at less than some pre-determined critical speed. The critical speed is considered to be in the range of 5 to 12 mph (or 2.2 to 5.3 m/sec) for most situations as reported and applied by Ham (1967).

Relationship Between Average Journey Time and Site Lengths. Queues were observed before the start of the green period in each cycle and the number of queued vehicles was counted in each direction. A linear regression analysis was undertaken to represent the average travel time through the site length. This would help assessing the required time for queue dispersion. To achieve this, four points have been calculated from the four sites representing different site lengths as shown in Figure 3. Each point represents the average time required by the 1st vehicles from both directions to clear the site length as shown in equation "(1)".

$$\mathbf{J_T = 0.1405\ L + 1.2112} \quad \text{.....................................} \quad \mathbf{(1)}$$

Where:

J_T = average travel time for the 1st vehicles to clear the shuttle length (seconds)

L = site length in metres

This equation can be applied to calculate the average speeds within the shuttle-lane for the four sites. These speeds were ranging between 20 and 25 km/h depending on site length (higher speeds for longer site lengths).

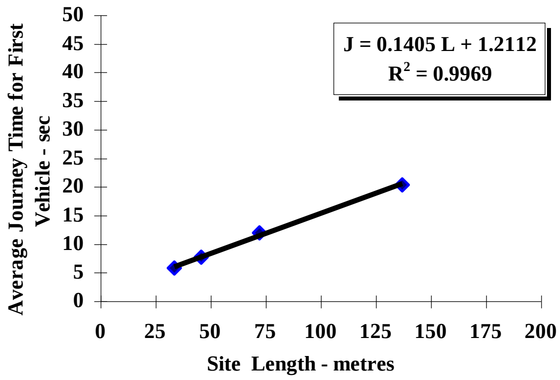


Figure 3. Regression line of average journey times through four sites

Queue Dispersion for Secondary Streams. Data on queue dispersion were only available from the secondary streams for Sites 2 and 4 because of site restrictions and the position of cameras used for data collection. After obtaining the time headways in each cycle for the secondary streams at Sites 2 and 4, it was possible to calculate the travel time required to reach the datum lines by the queued vehicles. These results are as shown in Figure 4 together with the regression lines representing the relationships as given in equations "(2) and (3)". The highest observed queues for Sites 2 and 4 were 11 and 15 vehicles, respectively.

$$\mathbf{J_2 = 2.0478\ Q_2 + 4.5049} \quad \text{.......................................} \quad \mathbf{(2)}$$

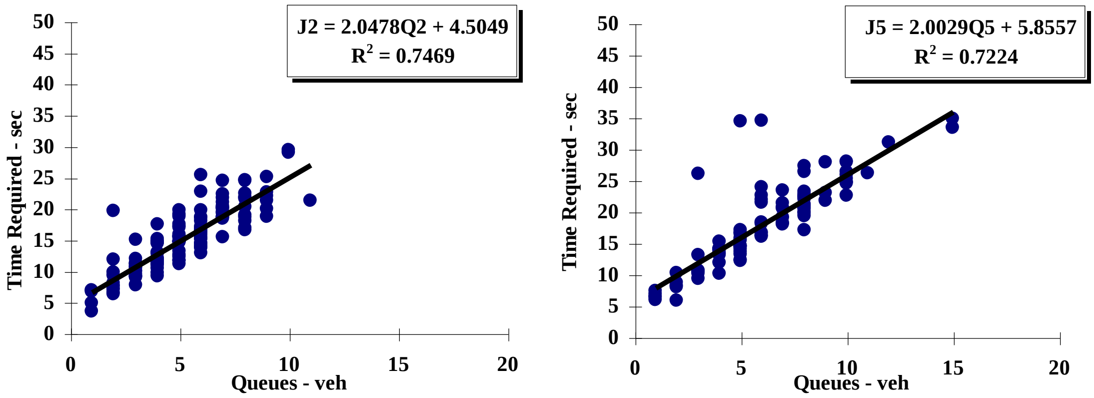
$$\mathbf{J_4 = 2.0029\ Q_4 + 5.8557} \quad \text{.......................................} \quad \mathbf{(3)}$$

Where,

J_2 , J_4 = time in seconds required for queue dispersion at Sites 2 and 4 respectively

Q_2 , Q_4 = queues at Sites 2 and 4 respectively

Based on equations "(2) and (3)", the time required to disperse queues can be calculated for Sites 2 and 4 as highlighted in Columns 4 and 5, Table 6. For illustration purposes, queues of sizes 1, 5, 10 and 15 vehicles were selected as shown in Column 3.



(a) (b)

Figure 4. Time required to disperse queues for the secondary stream, (a) Site 2 and (b) Site 4

Equations "(2) and (3)" were also used to calculate the time required to disperse queues for the different site lengths. In order to obtain this, use was made of equation "(1)" which gave the relationship between the average journey time for the leading vehicle in a queue (1st vehicle) for the different site lengths. This was calculated and shown in Column 6.

Table 6. Time required to disperse queues for different site lengths

Col. 1	Col. 2	Col. 3	Col. 4	Col. 5	Col. 6	Col. 7
Site	Site length	Queue	Time required to disperse queues			
	(metres)	(No. of vehicles)	Equation 2, Site 2	Equation 3, Site 4	Equation 1, All Sites	Col. 4 minus Col. 5
1	72.00	1	11.92	11.51	11.33	0.41
		5	20.11	19.53		0.58
		10	30.35	29.54		0.81
		15	NA	39.56		-
2	33.80	1	**6.55**	6.14	5.96	0.41
		5	**14.74**	14.16	–	0.58
		10	**24.98**	24.17		0.81
		15	**NA**	34.19		-
3	137.20	1	21.08	20.67	20.49	0.41
		5	29.27	28.69	–	0.58
		10	39.51	38.70		0.81
		15	NA	48.72		-
4	46.00	1	8.26	**7.85**	7.67	0.41
		5	16.45	**15.87**	–	0.58
		10	26.69	**25.88**		0.81
		15	NA	**35.89**		-

Note: NA = Not available

Both equations "(2) and (3)"can be generalised and used to calculate the time required to disperse the queues for the different sites by calculating the differences in values shown in Column 6. These values could be either positive or negative depending upon the site lengths. For example, equation “(2)” for Site 2 can be used for Site 1 by shifting it up by a value equals to [11.33 minus 5.96] which represents the difference in average travel time between the two sites. The results are as shown in Column 4. Similarly equation “(3)” for Site 4 can be used for other sites and the results are given in Column 5.

The differences in the calculated times to disperse queues for the different site lengths given in Columns 4 and 5 were relatively small for the selected queues as shown in Column 7. Therefore, equation “(3)” together with equation “(1)” can be generalised for site lengths between 33 and 137 m and queues between 1 and 15 vehicles to calculate the time required to disperse queues. This information may be useful in choosing a suitable green period in signals setting for different site lengths with different expected queues.

Driver Behaviour and Other Site Observations

Driver’s Compliance. Observations from the four sites have shown that some drivers do not comply with traffic signals when the signals change to red. This violation occurred on a few occasions when drivers were seen to be travelling at high speeds. Violations also occurred on one of the sites because a heavy plant used by the contractor was obstructing the passage of the open lane, resulting in drivers failing to comply with the signals. This working activity influenced drivers’ behaviour and was causing chaos and congestion in the vicinity of the shuttle-lane.

On another occasion where congested conditions occurred, manual controls were used for a short duration in order to clear the shuttle-lane from any remaining vehicles. This type of control was found to be effective in improving the operation of the shuttle-lane within a relatively short period of time under severe congested conditions.

The proportion of the violations described earlier varied between 12 and 30% of the total number of vehicles which passed in the primary stream during the survey period. Most of these violations were influenced by the congested conditions which affected drivers’ behaviour especially during the morning peak hours. These violations could endanger the safety of road users. Further research into accident rates and traffic control devices is therefore necessary to examine the safety and efficiency of the roadworks section.

Signing at Roadworks. Observations have shown that the signs were either insufficient or not completely visible to drivers. Some signs were not secured properly (for example some were moved or knocked down by vandals). There were no marked stop lines in either directions in the position of the temporary signals. This is important from the view point of safety, visibility and manoeuvrability as recommended in the Traffic Signs Manual - Chapter 8 (1991).

Other Observations. It was also observed that parked vehicles near to traffic signals could affect the position of vehicles queuing in the primary stream. This might also create blockage to the oncoming traffic from the secondary stream. On one of the sites, it was found that the "all-red" period was not sufficient to clear the shuttle-lane from the remaining vehicles, while on another, violations were caused by the long cycle time and long "all-red" periods. These could have adverse effects on both safety and capacity of the shuttle-lane.

CONCLUSIONS AND RESULTS

1. Results revealed that equivalent saturation flows for secondary streams were relatively higher than that of primary streams.
2. On average, the 1st vehicle queuing in the primary stream in the shuttle-lane required shorter time to clear the stop line compared with that of traffic signals at intersections. This may be attributed to drivers at intersections being more aware of traffic approaching from other arms.
3. The procedure described in Table 6 may be used in choosing a suitable green period in signals setting for different site lengths with different expected queues.
4. Under congested conditions, manual controls were found to be effective in preventing the shuttle-lane from being blocked for longer periods.
5. The proportion of drivers' non-compliance with traffic signals increased under congested conditions and when unusual working activities were taking place within the roadworks site. There was some evidence of non-compliance with the standards for signing at roadworks.

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