A Traffic Management Scheme to ease Vehicle & Pedestrian Movements in the Kottawa Town

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Abstract: Among the suburban towns in Sri Lanka Kottawa can be considered as one that regularly faces many traffic related problems. It is observed that most of those who enter the town, face problems daily. In the case of drivers, their time is often wasted due to traffic congestion and in the case of pedestrians their safety is at risk due to the absence of a good traffic management system. Kottawa experiences severe traffic congestion because many motorists and pedestrians fail to follow the prevalent rules and because there is no proper traffic management scheme. There are three junctions in the town. Most of the motorists have to travel at least through two of these junctions. Therefore it could be seen that a considerable amount of vehicles frequently travel across the three junctions during peak hours. Southern Expressway users too travel through the Kottawa town since Makumbura interchange is situated next to Kottawa. At present, it is seen that most motorists including those who use the Southern Expressway waste their time at the Kottawa town. Stage 1 of the Outer Circular Highway is now in operation and the other two stages will also be commissioned soon. Since they too will be starting from the Makumbura interchange, traffic in the town will increase further.

At the time of this study in early 2014, there were many illegal constructions and there were street vendors conducting their daily businesses by the roadsides. These activities have reduced the capacity of the surrounding road network. The situation at Kottawa is further worsened due to roadside parking, the large numbers of pedestrian crossings provided in the Colombo-Awissawella (A4) Road and the existence of two unplanned bus parking locations at both sides of the A4 Road.

In order to find solutions to these problems, the capacities of the roads were evaluated by compiling data related to vehicular flow, pedestrian movements and other supportive information of that area as at present and for the year 2034 (i.e. 20 years from now). From the data analysis, suitable solutions to improve the pedestrian and vehicle movements and vehicle parking facilities in the Kottawa town along with measures to improve safety are provided. Finally a suitable layout plan was introduced for the study area.

Keywords: Kottawa, Traffic Congestion, Pedestrian Safety

1. Introduction

Kottawa town is currently experiencing severe traffic congestion and many people who enter the town, appear to face problems such as waste of their time due to the traffic congestion and unsafe pedestrian movements. At the time of collecting the data for this study, the area under study had no proper traffic control system in operation. It was felt that a properly designed and signalized traffic control system might help to improve the road efficiency. Therefore in the absence of a proper traffic management scheme, this town gives rise to traffic blocks caused by the failure on the part of motorists and pedestrians to follow prevailing rules. The roads meeting at Kottawa are Colombo Awissawella Road (A4), Piliyandala Road (B239), Athurugiriya Road

(B45), Battaramulla Road (B47) and a short stretch of a one-way road. As seen in Figure 1, there are three junctions in the Kottawa town. The Colombo-Awissawella Road (A4) and the Athurugiriya Road (B45) meet at one junction (3-way junction). The Colombo-Awissawella Road (A4), Piliyandala Road (B239) and a stretch of the one-way road meet at the second

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junction (4-way junction). The Battaramulla Road (B47), Athurugiriya Road (B45) and a stretch of the one-way road are meeting at the third junction which is also a four-way junction. Therefore, a considerable number of vehicles frequently travel across the three junctions during peak hours and also at certain other times. Some obvious problems seen at the Kottawa town area are the illegal constructions, street vendors conducting their businesses by roadsides causing obstruction, roadside parking for want of sufficient parking areas, pedestrians jay-walking on carriageways, the absence of proper bus parking arrangements etc. The aim of this study is to design a junction improvement plan for the Kottawa town to ensure a smoother traffic flow and to facilitate safer vehicular and pedestrian movements catering to the additional vehicular traffic that will result from the extension of the Southern Expressway and the Outer Circular Highway through the Makumbura interchange.

Those who use the Southern Expressway have to travel through the Kottawa town to reach the Makumbura interchange. This makes it difficult to avoid traffic congestion at Kottawa. Stage 1 of the Outer Circular Highway is currently in operation and once the other two stages are completed, they will add more traffic to the Makumbura interchange which will further worsen the situation. Through this study it is expected to offer solutions to improve pedestrian movements, vehicle movements, and vehicle parking facilities in the Kottawa town and improve overall safety which will help the road users to have greater mobility, shorter travel times and easy accessibility to the town.

Figure 1 – Map of the Study Area

2. Methodology

2.1 Data Collection

The data that is already available was collected from the following organizations:

- Planning Division of the Road Development Authority,
- Outer Circular Highway Office, Nugegoda
- Kottawa Police Station
- Homagama Police Station

Field and traffic surveys were conducted as follows to gather data related to the current traffic and pedestrian movements:

- i. Field measurements of the width and length of different roads
- ii. Turning movement surveys at all three junctions
- iii. Spot speed study
- iv. Moving observer method exercise for calculating average delays
- v. Pedestrian counts at the side walks
- vi. Pedestrian counts at the cross walks
- vii. Parking supply & demand survey

2.2 Capacity Analysis

Capacity and Level of Service (LOS) are related to each other. The Capacity is defined as the maximum number of vehicles/passengers per unit time, that can be accommodated under given conditions with a reasonable expectation of occurrence. The Level of Service indicates how good the traffic situation of a given facility. Thus it gives a qualitative measure of traffic, where as the capacity analysis gives a quantitative measure of traffic of a given facility.

Capacity checks were carried-out on the Kottawa road sections for the peak traffic flows during the current year and for the projected flows for the year 2034 (20 years ahead) as per the US Highway Capacity Manual [1].

3. Analysis of Data

3.1 Identification of the Peak Hour

From the initial traffic counts, peak hours were identified as; the morning peak hour from 7:30hrs-8:30hrs, daytime peak hour from 12:30hrs-13:30hrs, and evening peak hour from 17:30hrs-18:30 hrs. Therefore turning movement data was taken by adding one hour before and after the pre-identified peak hours.

3.2 Capacity Analysis for Year 2014

In the area under study, there were three junctions with eight legs as shown in Figure 2.

Figure 2 - Capacity Analysis Road Sections

Road sections P₁-P₁, P₂-P₂ and P₄-P₄ along the Colombo-Awissawella Road had been considered as a multi-lane, and undivided suburban road. Others were considered as two-way, two-lane, rural roads. Different factors had to be taken into account under different conditions.

3.3 Calculation of the Allowable Flow of a Multilane Highway

By considering the left hand side of the road section P₁-P₁ during the morning peak hour at LOS B:

$$SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_E \times f_P$$

$$MSF_B = 1080 \text{ pc/h/lane}$$

$$N = 2 \text{ lanes}$$

$$\text{Existing Road Width} = 18.3\text{m}$$

$$\text{Lane width} = \frac{18.3\text{m}}{4} = 4.57\text{m (i.e.15ft)}$$

$$f_w = 1.0$$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1)}$$

$$P_T = \frac{\text{Heavy Lorries in the two directions}}{\text{Total vehicle volume in the two directions}} = \frac{59}{3745} = 0.016$$

$$P_B = \frac{\text{Large Buses in the two directions}}{\text{Total vehicle volume of the two directions}} = \frac{220}{3745} = 0.059$$

E_T and E_B for a level terrain condition:

$$E_T = 1.7, \quad E_B = 1.5$$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1)}$$

$$= \frac{1}{\{1 + 0.016(1.7 - 1) + 0.059(1.5 - 1)\}}$$

$$= 0.96$$

$$f_E = 0.8, \quad f_P = 1.0$$

$$SF_i = MSF_i \times N \times f_w \times f_{HV} \times f_E \times f_P$$

$$SF_i = 1080 \text{ pc/hr/lane} \times 2 \text{ lanes} \times 1.0 \times 0.96 \times 0.8 \times 1.0 = 1658.88 \text{ pc/hr}$$

$$= 1659 \text{ pc/hr(allow)} > 1186 \text{ pc/hr (actual)}$$

The actual flow rate is satisfied for the year 2014. The width of the road was sufficient to accommodate peak hour traffic flow.

3.4 Calculation of the Allowable Flow of a Two Way-Two Lane-Rural Road

Considering the road section P₃-P₃ during the morning peak hour at LOS B:

$$MSF_i \text{ (pc/h)} = 2800 \times \left(\frac{V}{C} \right)_i \times f_d \times f_w \times f_{HV}$$

$$v/c = 0.24, \quad f_d = 0.89, \quad f_w = 1.0$$

$$P_T = 59/2173 = 0.03$$

$$P_B = 74/2173 = 0.03$$

$$E_T = 2.2, \quad E_B = 2.0$$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1)} = 0.938$$

$$MSF_B = 2800 \times 0.24 \times 0.89 \times 1.0 \times 0.938 = 561 \text{ pcu/hr}$$

Results for the two-way, two-lane road during the morning peak hour and evening peak hour were calculated for different LOSs.

3.5 Capacity Condition during 2014

Analyzed results of the allowable and actual flows of the multilane and two-way, two-lane roads during the evening peak hour and morning peak hour were used to verify whether the capacity exceeded in 2014. According to the results, it can be seen that the capacity was fulfilled for a Level of Service D during both the morning peak hour and the evening peak hour during the year 2014. However the difference between the actual flow and the allowable flow was small. Therefore the actual flow will reach the allowable flow within the next few years.

3.6 Calculation of the Number of Lanes required by the Year 2034

For a design life of 20 years from 2014, the anticipated traffic was forecasted for the year 2034. The following equation was used to determine the number of lanes that will be required in the future taking into account the 2034 flow:

$$N = SF_{(allow)i} / (MSF_i \times f_w \times f_{HV} \times f_E \times f_P)$$

Considering the morning peak hour flow of the year 2034 at LOS B on the left hand side of section P1-P1

Allowable flow $SF_{(allow)B}$ = Junction 1

$$= \text{Flow}_{(11+12)} = 3354+751 = 4105 \text{ veh/hr}$$

$$MSF_B = 1080 \text{ pc/hr/lane}$$

$$f_w = 1.0$$

$$P_T = 1446/11864 = 0.122$$

$$P_B = 603/11864 = 0.051$$

$$E_T = 1.7, E_B = 1.5$$

$$f_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_B(E_B - 1)} = 0.9$$

$$f_E = 0.9, f_P = 1.0$$

$$N = 4105 / (1080 \times 1.0 \times 0.9 \times 0.9 \times 1.0) = 5$$

Considering the highest number of lanes of LOS D of each road section, the following conclusion was made.

Table 1 - Number of Lanes in each direction

Location	LOS	N per each direction
P1-P1	D	4
P2-P2	D	4
P3-P3	C	3
P4-P4	C	4
P5-P5	B	3
P6-P6	D	3
P7-P7	B	3
P8-P8	C	3

3.7 Roundabout Analysis

As a solution for the identified problems, a signalized roundabout was proposed at Junction-3 and its design was carried-out [2].

The typical turning movement diagram is shown in Appendix A -Figure A1. The left turn volume was neglected when calculating the entry volume of each lane as there was a separate lane provided for left turn vehicles.

For example, the entry volume of Colombo approach = 1676+149 = 1825 veh/hr

Circulating flow of Athurugiriya approach = 558+137+149 = 844 veh/hr

Entry volume and circulating flow are shown in Appendix A - Figure A2.

3.8 Capacity Analysis of the Roundabout at Junction 3

$$Q_{C_{max}} = \frac{n_e Q_c \exp \left[-\frac{Q_c T}{3600} \right]}{1 - \exp \left[-\frac{Q_c T_o}{3600} \right]}$$

$Q_{C_{max}}$ = Entry capacity (veh/hr)

n_e = Number of entry lanes

Q_c = Circulating flow (veh/hr)

T = Critical acceptance gap (4 sec)

T_o = Follow up headway (2 sec)

For the approach from Colombo:

$$\begin{aligned} Q_{C_{max}} &= (3 \times 524 \times 0.56) / (1-0.75) \\ &= 3521.28 \text{ veh/hr} \\ &= 3521 \text{ veh/hr} \end{aligned}$$

Results obtained are tabulated in Table 2.

Table 2 - Maximum Entry Capacity

Approach	n_e	Q_c	$e(-Q_c T / 3600)$	$e(-Q_c T_o / 3600)$	$Q_{C_{max}}$ (veh/hr)	Q_e (veh/hr)
Colombo	3	524	0.56	0.75	3521	1825
Borella	3	1825	0.13	0.36	1112	695
Athurugiriya	3	844	0.39	0.63	2669	2008
Homagama (not allow)	0	2145	0.09	0.3	0	0

The allowable maximum capacity was greater than the actual entry flow at all of the approaches. Therefore, the overall roundabout capacity will not be exceeded. Hence the pedestrian crossing may be allowed.

3.9 Delay Analysis of the Roundabout

The circulating flow and entry flows were used to calculate the queuing delay at each approach. Results are shown in Table 3.

Queuing delay for the Borella approach

$$\bar{w}_2 = \frac{\frac{1}{2 \times 4316.8029} + 0.07 \times 49.375 e^{(-2 \times 0.59)} [e^{(3 \times 0.59)} - 3 \times 0.59 - 1] / 0.59}{1 - 0.07 \times 49.375 [1 - e^{(-2 \times 0.59)}]}$$

$$\bar{w}_2 = 29.62 \text{ sec}$$

Table 3 - Calculation of the Queuing Delay

Approach	Qe (veh/hr)	Qc (veh/hr)	Delay (s)
Colombo	1825	524	4
Borella	695	1825	29.6
Athurugiriya	2008	844	20.5

3.10 Corner Kerb Radius of Junction 3

Figure 3 - Roundabout Elements

$$R_3 = R_1 - 2W + \sqrt{2}(R_1 - W)$$

Where

- R_3 = Corner kerb radius (m)
- R_1 = Inscribed circle radius (m)
- R_2 = Central island radius (m)
- W = Total approach width (m)

$$\begin{aligned} R_3 &= R_1 - 2W + \sqrt{2}(R_1 - W) \\ &= 15 - (2 \times 7.4) + \sqrt{2}(15 - 7.4) = 0.2 + 10.748 \\ &= 10.95 \text{ m} \end{aligned}$$

$$R_2 = 7.75 \text{ m}$$

3.11 Width of the Vehicle Path of Junction 3

$$Z = R_2 - W_2 + Z$$

- Where R_2 = Central island radius (m)
- W_2 = Total approach width (m)
- Z = $7.75 - 7.4 + 2.4$
= 2.75m

3.12 Turning Path Radius of the Island Arrangement

To determine the island radius, the average speed was assumed as 30 km/hr. Using the AUSTROADS guidelines [8], island radius was taken as 43.8 m

3.13 Delay due to Speed Variation

Spot speed data was collected and statistically analysed to identify the speed profiles at six selected points. Figures 4 to 7 show the speed profile variation with the distance between selected points in both directions (Colombo to Awissawella and Awissawella to Colombo). These speed profiles indicate the maximum and minimum speed variations in both directions. Figures 4 and 5 show morning peak variation, and Figures 6 and 7 show evening peak variations.

Figure 4 - Speed Profile of Awissawella - Colombo Direction at 7:30-8:30hrs

Figure 5 - Speed Profile of Colombo - Awissawella Direction at 7:30-8:30hrs

Figure 6 - Speed Profile of Awissawella - Colombo Direction at 18:30-19:30hrs

Figure 7 - Speed Profile of Colombo - Awissawella Direction at 18:30-19:30hrs

It can be seen that the speed first decreased from point 1 to point 2 and increased thereafter under steady flow in both directions. The maximum speed was 34 km/h in Awissawella-Colombo direction in the morning and 26 km/h in the opposite direction in the morning at Point 2. The lowest maximum speed in the Colombo-Awissawella direction was in the morning.

3.14 Delay experienced by Expressway Users

The time taken to travel between three identified points P1, P2 & P3 in the Awissawella-Colombo direction was noted and the travel times between the points P4, P5 & P6 in the opposite direction were recorded using a car. The average of the five trips in one direction during the morning peak hour, day time peak hour, evening peak hour and also during two off peak hours was computed. The distances between any two points were also recorded. The average journey times so computed are shown in Table 4.

Table 4 - Travel Time during Peak Hours

Peak Hours	From Awissawella to Colombo(seconds)	From Colombo to Awissawella(sec onds)		
	Average Time			
	P1 toP2	P2 to P3	P4 to P5	P5 to P6
7:30 – 8:30hrs	139.0	68.0	97.7	71.3
12:30 – 13:30hrs	66.3	49.8	61.8	48.2
18:30 – 19:30hrs	115.4	66.0	196.6	71.3
Distance (km)	0.3	0.4	0.4	0.3

From Table 4, it is seen that the motorists take a longer time to travel from P4 to P5 (coming from the direction of Colombo) during evening peak hour and that during morning peak hour a longer time is taken to travel from P1 to P2 in the opposite direction (coming from the direction of Awissawella).

Table 5 - Travel Time during Off Peak Hours

Average Time	From Awissawella to Colombo(seconds)	From Colombo to Awissawella(seconds)			
	P1 to P2	P2 to P3	P4 to P5	P5 to P6	
10:30 - 11:30hrs	56.6	61.2	57.6	48.0	

16:30 - 17:30hrs	58.8	62.0	144.0	50.6
Distance (km)	0.3	0.4	0.4	0.3

During the evening off peak hour, it takes a longer time to travel from Point 4 to Point 5 coming from the direction of Colombo. It appeared that motorists spend a longer time to pass Point 4-5 section and Point 1-2 section. Therefore in these sections, delays occurred and a solution will be needed to minimize travel time.

3.15 Distribution of Accidents

Data under the categories of Fatal, Grievous and Minor (or Light) injuries and related to accidents that occurred during the last ten years is shown graphically in Figure 8. Accident data was used to identify black spots and in the design layout. Steps were taken to eliminate negative effects of blackspots.

Figure 8 - Accident Details (Year 2003 to 2013)

3.16 Black Spot Identification

Locations where three or more accidents had occurred during the last ten years were considered as blackspots [3]. The aim was to reduce the number of accidents at these identified blackspots.

The accident records was used to study in detail the causes of accidents; types of vehicles involved, time of the accidents, collision types, types of injuries or damages etc. The collision diagram developed for the past ten years considering this data is shown in Appendix B-Figure B1.

3.17 Pedestrian Analysis - Side Walk Analysis

It is observed that most pedestrians walk along the right or the left side of the footpath of Awissawella - Colombo (A04) approach. The total number of pedestrians walking past (in both directions) at a certain point was counted at 15 minute intervals during peak time.

Table 6 - Peak Pedestrian Count

Time	Right side	Left side
Morning Peak Time	Nos	Nos
6:45 – 7:00hrs	162	172
7:00 – 7:15hrs	170	175
7:15 – 7:30hrs	175	180
7:30 – 7:45hrs	168	175
7:45 – 8:00hrs	160	170
Evening Peak Time	Right side	Left side
16:30 – 16:45hrs	110	108
16:45 – 17:00hrs	122	115
17:00 – 17:15hrs	145	138
17:15 – 17:30hrs	140	125
17:30 – 17:45hrs	138	120

3.18 Level of Service to Pedestrians in 2014

The total walkway width is 2.0m (including electricity poles and waste bins).

Assumptions:

- 15 min peak flow rate is 180 p/15-min on the left side
- 15 min peak flow rate is 175p/15-min on the right side
- Obstacles -Electricity poles, waste bins etc.

Table 7 – Results of Pedestrian Level of Service Results

Determine width adjustment	$W_0 = 0.8 + 0.9 = 1.7$
Determine effective width W_E	$W_E = W_T - W_0$ $= 2.0 - 1.7 = 0.3\text{m}$
Find v_p (use Equation 3.6)	Left side $V_p = V_{15} / (15 \times W_E)$ $180 / (15 \times 0.3)$ 40 p/min/m
	Right side $V_p = V_{15} / (15 \times W_E)$ $175 / (15 \times 0.3)$ 38.9 p/min/m
Determine LOS for the average condition	Left side=LOS D Right side=LOS D

At LOS D, a very slow pedestrian movement touching each other is possible. Long-term durations at this density will not be comfortable.

3.19 Forecast of the Side Walk Pedestrian Numbers for 2034

Pedestrian growth rate in the Colombo District was taken as 0.35% based on the statistics of the last census done.

Number of pedestrians after a period of n years
= Number of pedestrians in 2014 $\times (1 + \text{Growth percent})^n$

It was presumed that the maximum number of pedestrians on the left hand side is 180 and that it is 175 on the right hand side.

Left hand side = $180 (1 + 0.0035)^{20} = 193.02$
= 193 ped / 15 min

Right hand side = $175 (1 + 0.0035)^{20} = 187.66$
= 188 ped / 15 min

3.20 Calculation of Walkway Width for the Year 2034

The pedestrian level of service was taken as C and it was presumed that there are light poles and trees.

Considering LOS C on the LHS

$$W_E = W_T - W_0$$

$$W_E = [(V_{15}) / (15 \times V_p)]$$

$$V_p = 24 \text{ p/min/m (used Appendix-A, Table-A.13)}$$

$$V_{15} = 193 \text{ p/15-min}$$

$$W_E = [(193) / (15 \times 24)] = 0.54$$

$$W_0 = 0.8 + 0.6 = 1.4$$

$$W_T = W_E + W_0$$

$$= 0.54 + 1.4 = 1.94\text{m}$$

Right hand side $W_T = 1.92\text{m}$ when computed as above.

3.21 Cross Walk Analysis

There were four zebra crossings in the town as shown in the Figure 9. Pedestrians walking in both directions were counted at each crossing at fifteen minute intervals on a week day. Data was collected from 7:00 – 9:00hrs, 12:30–14:30hrs, and from 17:00–19:00 hrs.

Figure 9 – Locations of Pedestrian Crossings

The number of pedestrians walking past each crossing during morning, day time and evening peak hour is plotted in Figure10.

Figure 10 - Average Pedestrian Profile at the Four Crossings

3.22 Identification of Peak Pedestrian Count at the four Zebra Crossings

In the morning, the number of pedestrians walking across the crossings was counted at fifteen minute intervals from 7:00 – 9:00hrs and from 17:00 – 19:00 hrs in the evening. The results are tabulated below.

Table 8 - Pedestrian Count in the Mornings

Time	PC1+ P C2	PC3+ PC4	PC5+ PC6	PC7+ PC8
7:00- 8:00hrs	199	1321	299	483
7:15- 8:15hrs	200	1327	334	459
7:30- 8:30hrs	273	1582	603	434
7:45- 8:45hrs	239	1618	646	458
8:00- 9:00hrs	212	1521	634	485

Table 9 - Pedestrian Count in the Evenings

Time	PC1+ PC2	PC3+ PC4	PC5+ PC6	PC7+ PC8
17:00- 18:00hrs	239	2585	578	<u>836</u>
17:15- 18:15hrs	218	2891	593	811
17:30- 18:30hrs	230	<u>3254</u>	639	738
17:45- 18:45hrs	269	3232	<u>665</u>	639
18:00- 19:00hrs	<u>308</u>	2963	599	457

The highest pedestrian number is underlined (see Table 9) by comparing the morning and evening counts at each crossing. These highest

values were used to analyse each pedestrian crossing. Results are graphically shown below.

3.23 Forecast of Crosswalk Pedestrians for 2034

Based on the statistics, the pedestrian growth rate was taken as 0.35%. Results are shown in Table 10.

Table 10 - Expected Pedestrian Growth Rates in 2034

Section	Peak pedestrian number per hour	Growth pedestrian number per hour	Peak pedestrian number per minute
PC1+PC2	308	330	6
PC3+PC4	3254	3490	58
PC5+PC6	665	713	12
PC7+PC8	836	897	15

3.24 Calculation of the Cross Walk Width for 2034

A pedestrian LOS D was considered as satisfactory for the year 2034 at the peak pedestrian flow. Hence pedestrian space was taken as 2.4 ft²/person.

Design width (ft)=(Pedestrian space × Pedestrian per min× cycle time)/Crossing length

As the shifting of the bus stand to a new location has been proposed, it is assumed that the number of pedestrians at the pedestrian crossing at P3+P4 will be reduced. Existing pedestrian crossings P1+P2, P5+P6 and P7+P8 will be shifted to three signalized intersections. The new crossing arrangement is indicated in the proposed layout plan.

Table 11 - Pedestrian Cross-walk Design

Section	Pedestrian Space (ft ² /p)	Pedestrian per minute (p/min)	cycle time (min)	Crossing length (ft)	Design width (ft)	Design width (m)	Recommended width h(m)
PC1+PC2	2.4	6	2	96	2.8	0.85	2.5
PC3+PC4	2.4	15	2	96	7.5	2.29	2.5
PC5+PC6	2.4	12	2	72	8.0	2.44	2.5
PC7+PC8	2.4	5	2	96	2.5	0.76	2.5

Figure 11 - Proposed Crossing Locations

3.25 Vehicle Parking

There is a block of government owned land closer to Junction 2 in the Kottawa town and adjoining the Awissawella - Colombo Road and Piliyandala Road. This place is the most suitable location to provide for car parking. The extent of this block of land is about 121m x 84m.

3.26 Off-Street Parking Layout

Considering the fact that there is not much land available in the Kottawa town, a multi storied car parking layout with split level car parking is proposed [4], [5].

Figure 12 - Floor arrangement of the Split Level Car Park

Based on the findings of the parking demand survey, a split level four storied car parking arrangement is proposed to cater to the forecasted traffic demand in the year 2034 (20 years from now), with each story consisting of adjacent parking levels separated by a half story height. Forty vehicles can be parked at the half story level and a total of 240 vehicles can be parked altogether in the multi-story car parking facility.

Proposed parking arrangement requirements are as follows:

Most appropriate parking angle is 90° and the level of service selected is 1 to provide for future parking demand.

Angle of parking - 90° , Level of service - 1, A - 3.2m

B - 3.2 m, C - 0.0 m, D - 5.4 m

Aisle width - 6.0 m, Entrance side distance - 12 m

Exit side distance - 9 m

Figure 13 - Circulation of Vehicles between Two Half Storied Levels of the Car Park

The proposed off-street three story car park arrangement is given in Figure 13.

3.27 Bus Bay and Bus Parking Area

The area occupied by the present bus stand has not been planned properly and the accident studies reveal that a large number of accidents occur in this area. This highlights the need to plan properly bus loading and unloading. Considering the land size and parking demand of buses, a properly designed bus bay would minimize the risk the pedestrians would face when they embark and disembark busses.

Following dimensions were used in the bus-bay design to suit the requirements [5].

Entrance taper length - 13.25 m

Entrance taper radius - 60 m

Deceleration taper length - 13.25 m

Deceleration taper radius - 60 m

Stopping area - 25m (for 2 buses)

Acceleration taper length - 5.4 m

Acceleration taper radius - 15 m

Exit taper length - 10.8 m

Exit taper radius - 30 m

Angle of parking- 90° , Level of service-1, A- 3.2m

B- 3.2 m, C- 0.0 m, D- 5.4 m

Aisle width- 6.0 m, Entrance side distance- 12 m

Exit side distance- 9 m

3.28 Proposed Traffic Signal System

A number of phases have been selected considering the traffic volume of each road at the intersection. When Kottawa town as a whole is considered, there is Colombo - Awissawella Road intersecting with roads leading to Piliyandala, Athurugiriya and Battaramulla. During morning and evening peak hours, traffic flows along the Colombo and Awissawella approaches are more than the flows of any other approach. The following three phases were selected as the most suitable phases for the Kottawa intersection to minimize delays and queue values after giving due

priority to the Colombo and Awissawella approaches.

Figure 14 - Proposed Traffic Signal System (three phases)

The forecasted values for the next twenty years were converted to passenger car units (PCU) and that data was used for the signal design and the following section shows the specimen calculations and the results. When the signal phasing and timing are fixed for peak flows in 2034, the timing could be adjusted rather easily to suit present flow conditions. Furthermore, this signal timing has to be adjusted from time to time as the flow conditions change or upgraded for a flow actuated system.

Specimen Calculations

Morning Peak Hour

The lane width is considered as 3.7m and site factor as 100% (between 85% for a poor site and 120% for a good site) with no gradient. For turning lanes, turning radius is to be taken as 15m.

Junction 1

(Note - Different directional flows of Junction 1 are given notations 11, 12, 13, 14 etc. for reference purposes)

Step 1 - Saturation Flow Calculation

Saturation Flow 11 = $525 \times 14.8 = 7770$ pcu

Saturation Flow 12 = 1920 pcu

Saturation Flow 13 = $3000 / (1 + (1.515/15)) = 2725$ pcu

Saturation Flow 14 = not allowed to move and flow diverted to 23 and 24 flows

Saturation Flow 15 = $525 \times 7.4 = 3885$ pcu

Saturation Flow 16 = $525 \times 14.8 = 7770$ pcu

Step 2 - Revised Saturation Flow Calculation

If shared lanes are used by the straight flow and right turning flow, turning percentages

used to calculate the revised saturation flows would be considered.

Revised Saturation Flow 11 = $7770 \times 104.07/100 = 8086$ pcu

Revised Saturation Flow 12 = 1920 pcu (no shared lanes)

Revised Saturation Flow 13 = 2725 pcu (no shared lanes)

Revised Saturation Flow 14 = not allowed to move and flow diverted to 23 and 24 s.

Revised Saturation Flow 15 = 4562 pcu (no shared lanes)

Revised Saturation Flow 16 = $7770 \times 117.42/100 = 9124$ pcu

Step 3 - Site Factor and Gradient

Saturation flow does not change with the site factor and gradient, because the site factor is taken as 100% and there is no gradient in the study area.

Step 4 - Design Flow

Adjusted final design flow for the year 2034 is used as the design flow for the pcu.

Design flow 11 = 4181 pcu

Design flow 12 = 814 pcu

Design flow 13 = 2707 pcu

Design flow 14 = diverted

Design flow 15 = 2329 pcu

Design flow 16 = 7699 pcu

Step 5- Revised Design Flow

In this step, only Flow 12 changes, because Flow 12 can be allowed to move during the two phases and the revised design flow is obtained from the design flow divided by 2.

Revised design flow 12 = $814/2 = 407$ pcu

Step 6 - Revised Final Design Flow

As in Step 2, turning percentages are used to calculate the revised design flows.

Revised design flow 11 = $4181 \times 104.07/100 = 4351$ pcu

Revised design flow 12 = 407 pcu

Revised design flow 13 = 2707 pcu

Revised design flow 14 = diverted

Revised design flow 15 = 2329 pcu

Revised design flow 16 = $7699 \times 117.42/100 = 9039$ pcu

Step 7 - q/s Ratio Calculation

The revised final design flow divided by the revised saturation flow is the final result of the above calculations.

The q/s ratio (Y) values are reduced through the following modifications:

- At Junction 3, all approaches need 3 lanes to circulate around the roundabout.
- Flows 16, 24, 26, 28, 33, 36 and 39 can be allowed to travel during the whole cycle time if there is no opposing flow.
- Flow 12 can be allowed to move during Phase 1 and Phase 2. Design flows are then separated for the different phases and calculations done.

Step 8 –Practical “y” Values

When the three junctions are considered together, the actual design flows are found to be greater than the saturation flows given by the number of lanes and lane widths. As a result of this, in some flows, q/S ratio (y) is greater than 1. The result shows that it is not possible to discharge a whole number of vehicles within one signal cycle. During the selected cycle time, the volume of discharge can be calculated from the design peak hour volume. Then the "y" value has to be calculated again. Using the new "y" value, calculate Y and green time for each phase.

Step 9 – Cycle Time Selection and Green Time Calculation

When

$I = 5s$ for each phase,

$a = 3s$,

$l = 2s$ [starting lost time/stopping lost time]

Lost time $[L] = p(I - a + l)$

$L = 3 \times (5 - 3 + 2) = 12s$

Then the Optimum Cycle Time is given by the following equation:

$$C_0 = (1.5L + 5) / (1 - Y)$$

$$C_0 = (1.5 \times 12 + 5) / (1 - 0.82)$$

$$C_0 = 127.78s$$

The optimum cycle time can be taken from the $3/4 C_0 - 3/2 C_0$ range. The design cycle time will fall within 95.83s – 191.67s based on the C_0 value chosen. The green times for each phase are calculated by taking cycle time as 120s.

Effective green time = cycle time – total lost time per cycle

$$\text{Effective green time} = 120 - 12 = 108s$$

This total effective green time is used to find out the corresponding effective green time of each phase by using the q/s ratios.

Effective green time calculation

For Phase 1 = Total effective green time $\times y_1 / Y$

y_1 = Maximum q/s ratio of Phase 1

Y = Sum of maximum q/S ratios

$$\text{For Phase 1} = 108 \times (0.25/0.82) = 33s$$

Calculate the actual green time (k) for Phase 1 as

$$k_1 = 33 + 2 - 3 = 32s$$

Table 12 - Effective Green Time and Actual Green Time of each approach during morning peak hour

Phase	Discharge flow (q)	Saturation flow (s)	q/s ratio	Effective green time(s)	Actual green time(s)
1	2004	7995	0.25	33	32
2	592	3885	0.15	20	19
3	1136	2725	0.42	55	54

Step 10– Delay Calculation

Delay can be calculated as follows for Phase 1.

$$\lambda = g/c\lambda = 33/120$$

$$\lambda = 0.28$$

$$x = q / \lambda = 2004 / (0.28 \times 7995) = 0.895$$

By using above λ and x values A and B are obtained from the tables.

$$A = 0.346$$

$$B = 3.814$$

$$M = qc / 3600 M = (2004 \times 120) / 3600 = 66.80 \text{ pcu}$$

$$D = 3.916 \%$$

$$d = cA + (B / q) - D \text{ 3.18}$$

$$= \{cA + (B / q)\} \{1 - D/100\}$$

$$= \{(120 \times 0.346) + (3.814/2004)\} \{1 - (3.916/100)\}$$

$$d = 39.90 \text{ s/pcu}$$

Table 13 - Delay values for the Three Phases

Phase	$\lambda = g/c$	$x = q / \lambda s$	A	B	$M = qc/3600$ (pcu)	D (%)	d (s/pcu)
1	0.28	0.895	0.346	3.814	66.800	3.92	39.90
2	0.17	0.896	0.406	3.860	19.733	10.08	43.81
3	0.46	0.906	0.250	4.366	37.867	10.11	26.97

The minimum delay is observed in Phase 3 and this means that in Phase 3, the time for which the motorists have to wait before they can depart through the intersection is minimum. In Phase 2, the time that the motorists have to wait for travel through the intersection is maximum.

Step 11– Calculation of the Average Queue

$$R = c_0 - g$$

$$= 120 - 33 = 87s$$

From delay calculation,

$$d = 39.90 \text{ s/pcu}$$

$$q/3600 = 0.557 \text{ pcu/s}$$

$$N = q \{(R/2) + d\} = 0.557 \{(87/2) + 39.9\}$$

$$N = 46.454 \text{ pcu}$$

Table 14 - Average Queue Values

Phase	$R=c_0-g$ (s)	d (s/pcu)	$q/3600$ (pcu/s)	N (pcu)
1	87	39.90	0.557	46.454
2	100	43.81	0.164	15.385
3	65	26.97	0.316	18.793

At the beginning of the green period, the average queue is usually the maximum queue in the cycle. The length of queues depends on the composition of traffic. In the signal design, minimum queue is observed in Phase 1. Thus in Phase 1, motorists have to wait before they can depart from the intersection unlike the motorists of the other two phases [6], [7].

The maximum flow that can be passed with signal settings = $(g/C) \times S$

For Phase 1 = $33/120 \times 7995 = 2199$ pcu/h

For Phase 2 = $20/120 \times 3885 = 648$ pcu/h

For Phase 3 = $55/120 \times 2725 = 1249$ pcu/h

For the evening peak hour, calculations were done as for the morning peak hour calculations and 90s was selected as the cycle time for all the cases.

Table 15 - Final Results for Signal Timing

	Morning Time	Evening Time
Cycle Time	120s	90s
Effective Green Time of Phase 1	33s	36s
Effective Green Time of Phase 2	20s	22s
Effective Green Time of Phase 3	55s	20s

Figures 15 and 16, show the timing diagrams of the three phase traffic signal system with 120s and 90s as cycle times for the morning and the evening respectively.

Figure 15 -Timing Diagram for Morning **Figure 16 - Timing Diagram for Evening**

3.29 Channelization

Channelization simplifies the traffic flow by restricting the driver's choice, reducing conflict points, segregating conflicting flows, guiding traffic into suitable paths, and reducing the area of conflicts by controlling the intersecting traffic streams. Channelization protects pedestrians from turning vehicles by providing pedestrian refuges and discouraging prohibited or undesirable movements of motorists.

A centre median along the A4 Road with semi-circular end shapes was proposed for the Kottawa town to separate out the opposing traffic at the traffic signal and at the roundabout. The length of the centre median was taken to suit the site conditions.

Minimum width of centre median -1.2 m.

Radius of the semi-circular end -0.6 m.

The proposed centre median is indicated in the final layout plan given in Appendix C -Figure C1.

4. Conclusion

From the capacity analysis results, it was seen that a signalized roundabout at the third junction [i.e. the four-way junction where the Battaramulla Road (B47) and the Athurugiriya Road (B45) cross each other] and at the other two intersections was necessary and it was appropriate to have traffic signals at these places. Thus designs were carried out accordingly. Cross-walk and side walk designs were done to improve the pedestrian facilities. A suitable parking area was proposed separately for the vehicles and busses. In addition to this, bus bays for loading and unloading pedestrians, suitable channelization turning regulations, and hand railings were proposed. Finally, a suitable layout plan was drawn based on the results of the analysis and designs.

From the road capacity checks performed on different road sections in the Kottawa town, it was found that some road sections could not handle traffic flows during peak hours even as at now and that the other sections also would face the same situation in 20 years time from now (i.e. 2034), unless improvements are carried-out. Hence to cater to the 2034 traffic flows, the following numbers of lanes are proposed for the sections (see Table 16).

In addition to the traffic growth in the surrounding roads, additional vehicular traffic that will generate in future from the Southern Expressway expansion and Outer Circular Highway through the Makumbura interchange has been taken into account.

Table 16 – Number of Lanes of Road Sections

Road Section	Number of lanes in one direction
Colombo – Awissawella Road (A4)	4
Piliyandala Road	3
One-way Road	3
Athurugiriya Road	3
Battaramulla Road	3

To cater to the heavy traffic during peak hours, a traffic signal system linking the three junctions was designed. At Junction 3, a signalised roundabout with a center-island (7.75m radius) was proposed. A centre median along A4 with semi-circular end shapes was also proposed for the Kottawa town to separate the opposing traffic through the traffic signal and the roundabout.

The present bus stand has not been properly planned and it does not have passenger shelters and queuing facilities. Therefore passengers face difficulties when busses turn within the area forcing them to move away from their standing positions to give-way for the turning busses and this causes them great inconvenience. Hence to minimize this bus parking problem, it is proposed that there should be forty slots with parking facilities and two bus bays for the Awissawella route and Colombo route which can load concurrently two buses going in the same direction.

Currently there are many shops within the town area and customers park their vehicles in an undisciplined manner. According to accident reports, vehicles had collided with parked vehicles at identified parking places due to irregular parking. Some shops have provided unplanned parking areas and as a result of this, customers use sidewalks for parking their

vehicles. Vehicles and pedestrians face difficulties as a result of this practice. Hence a comprehensive parking demand survey was carried-out in the town area and considering the results of questionnaire surveys, a split level four storied car park was proposed, with each story with adjacent parking levels separated by a half story height. The car park that was designed was of 85.0 m x 33.6 m in size and it could accommodate 240 vehicles for parking at a time.

In addition to those solutions, to segregate pedestrians and vehicles and minimise j-walking pedestrians, hand rails in town area were proposed. In addition, to reduce conflicting movements of traffic flows, left turning regulation for Athurugiriya Road section at Junction 1 was proposed. The authors believe that these solutions would minimize the delays experienced by the expressway users in the Kottawa town and provide a safe and efficient movement for vehicles and pedestrians along the roads in the town.

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Appendix A

Appendix A - Figure A1 -A typical Turning Movement Diagram

Appendix A - Figure A2 -Entry and Circulating Flow Diagram

Appendix B

Appendix B – Figure B1 -Entry and Circulating Flow Diagram

Appendix C

Appendix C – Figure C1 – Final Layout