Traffic Signal Coordination: A Review of Familiar Practices and Research Needs for Developing Countries

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Abstract: Coordinated operation of traffic signals plays a key role in performance of an urban traffic corridor. It caters multifold benefits such as minimization of delay, reduction in number of stops, congestion, fuel consumption, and improvement of air quality. The present work attempts to present a detailed review of available literature to identify the key aspects of a coordinated traffic signal system covering a wide range of issues, such as warrants to adopt signal coordination, design approaches, state of the art design tools, and performance evaluation of the system. Several key aspects such as vehicle progression, effect of flow state, design objective, etc. are found to have significant impact on a coordinated traffic signal system. The present work also highlights the key research areas for coordinated traffic signal system in the backdrop of developing countries such as India.

Keywords: Traffic signal coordination, Urban mobility, Developing countries, Research gaps

1. INTRODUCTION

Traffic signals are the integral parts of an urban transport network. It provides orderly movement of conflicting vehicles at traffic nodes, thus plays a key role in urban mobility. Inefficient operation of traffic signals results into several externalities such as congestion, vehicle emission, fuel consumption, and air/noise pollution. As an example, fuel consumption due to the idling of vehicles at signalized intersections caused an economic loss of about 9944.5 million INR per annum in Delhi, India (Parida and Gangopadhyay 2008). Improved signal control measures such as retiming, activation, progression, actuated control, etc. result in enhanced capacity, and are found analogous to physical augmentation of existing roadway facility (Harwood, 1987; Park and Chen, 2010). Over the years, suitability of various strategies for operating the traffic signals is tested over various roadway and traffic scenarios. Traffic signals may be operated in an isolated or coordinated way. As the name suggests, isolated traffic signals are operated without considering the interaction of adjacent traffic signals. In contrary, the coordinated operation of traffic signals considers their mutual interaction. At a corridor or network level, signal coordination is essential for area traffic control (Robertson 1969). Signal coordination promotes smooth flow of traffic along a corridor, and thus reduces delay, bring down the number of stops and harmonizes the speed profile (Koonce et al. 2008; Park and Chen 2010; Beak et al. 2017). It is also found effective to mitigate congestion, and reduce vehicle emission and fuel consumption (US DoT 2010, Li and Tarko 2011, FGSV, 2003). For closely spaced traffic signals, their coordinated operation is found beneficial over their isolated operation (Koonce et al., 2008; Li and Tarko, 2011). Therefore, from the mobility point of view, signal coordination is often desirable

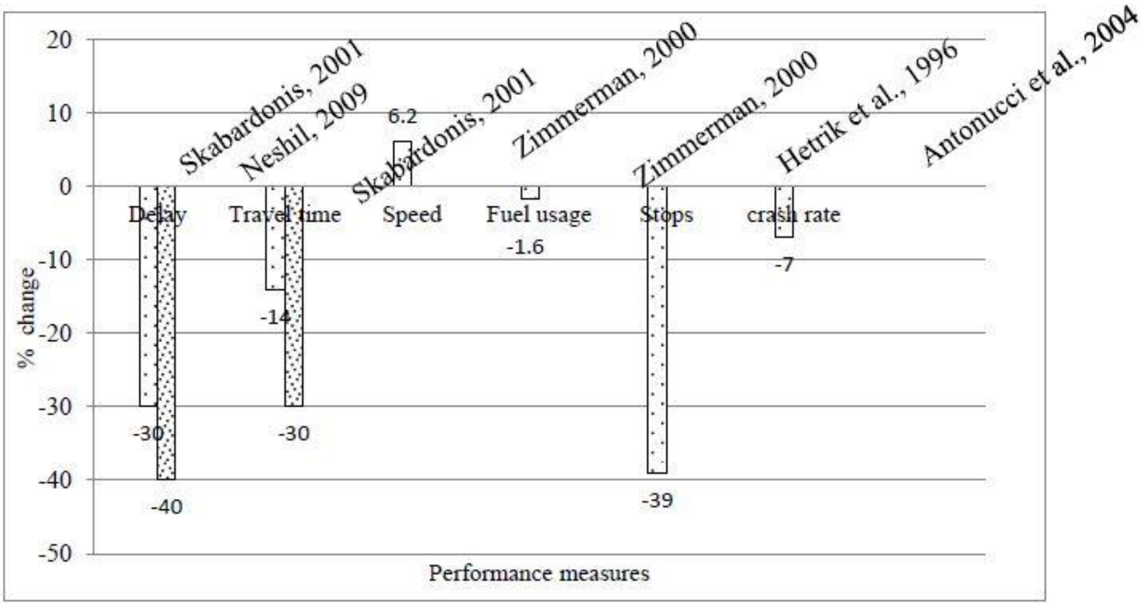


Figure 1: Result of performance evaluation for coordinated signal system under different measures

Several studies reported the realized benefits from signal coordination using various performance measures. These findings are summarized in Figure 1. As indicated in Figure 1, signal coordination is found effective to improve speed, and reduce vehicle delay, travel time, vehicle stops and fuel usage. Due to the reduction in vehicle stops, a number of rear-end collisions also reduces under signal coordination (Stevanovic et al., 2011). It may be indicated by a decrease in vehicle crash rate due to signal coordination as shown in Figure 1. Signal coordination is also found effective for reducing red light violations (Fambro et al., 1995; Shinar et al., 2004), and thereby lowering the vehicle crash possibility at signalized intersections. In recent years, (2012-16), due to city wide implementation of signal coordination in Toronto (Canada), the annual reduction in delay (in hours) and vehicle stops (in numbers) were found as 8.7% and 10.1% respectively (Gov. of Toronto, Canada, 2016). This signal coordination program resulted in an improvement of average corridor speed by 3.9% and reduction of fuel consumption and vehicle emission by 4.8% and 4.7% respectively. Life cycle benefit/cost ratios for the signal coordination program were found as 62:1 in the city of Toronto. In Virginia, USA, benefit/cost ratio of a coordinated actuated system was estimated as 461.3:1 for 10 years (Park and Chen 2010).

Shunkari (2004), Nisbet and Hammond (2011) reported the benefit/cost ratio of signal coordination programs in the city of Texas and Washington as 53:1 and 9:1 respectively. De Coensel et al. (2012) reported a reduction in air pollutant (viz. CO_2, NOX, and PM10) in the range of 10% to 40% and sound pressure by 1 dBA near the traffic signals due to signal coordination along a simulated one-way arterial corridor. Reduction in air pollutants and sound pressure at traffic signals were also reported by Unal et al. (2003), Desarnaulds et al. (2004), and Zhang et al. (2009). Interestingly, when closely spaced intersections cater to a high volume of through traffic, signal coordination works better than the actuated traffic control system (Koonce et al. 2008). Apart from these multi fold benefits in the context of mobility; signal coordination has also been studied for the interest of pedestrians. During the off-peak periods, pedestrians can take the advantage of signal coordination. In down town areas, both the pedestrians as well as the vehicle occupants can be benefitted from the coordination of signals (Virkler et al., 1998).

The benefits from signal coordination depend upon the area-specific traffic management policy and associated objectives. Users‟ experience and their perception about the operation of coordinated traffic signals are also important to assess the effectiveness of a coordinated traffic signal system (Koonce et al. 2008). Therefore, depending on area-specific stakeholders, the effectiveness of signal coordination may be realized in several ways. For example, in downtown areas, pedestrian traffic may be preferred over vehicular traffic; a community may seek a reduction in vehicle emission/traffic noise or else; vehicle throughput along a corridor may be focused over vehicle delay (Koonce et al. 2008). Apart from these policy level objectives, from traffic engineers‟ perspective, several other factors influence the operation of a coordinated traffic signal system. These may be identified as,

(a) Roadway characteristics, ,i.e. link/intersection layout, signal spacing, abutting land-use type, level of side-friction, frequency and type of access to adjacent property, functional classification of a corridor, its geometric layout, etc. (Harwood 1987);
(b) Prevailing state of traffic parameters, i.e. distribution of approach volumes, etc. (Chang, 1985), vehicle mix, volume to capacity ratio, link speed; vehicle arrival pattern at downstream stop-lines, queue spillback towards a upstream stop-line (Gattmen et. al.), and
(c) The site-specific traffic management and control measures, i.e. speed limit, controlled/ uncontrolled driveways, etc., out-dated offsets (Park and Chen, 2010)

Apart from these case-specific findings, in several parts of the world, investigations on signal coordination have revealed its effectiveness to bridge the gap between demand and supply of urban road transport systems (Day et al., 2010). Based on above discussion, major focus areas of investigations on signal coordination may be summarized as,

(i) Investigating the effectiveness of signal coordination for urban traffic management,
(ii) Identifying the benefits from various stakeholders‟ point of view,
(iii) Assessing the policy level objectives adopted by a jurisdiction, and users‟ perception about the system, and last but not least
(iv) Understanding the mutual interaction of roadway, traffic and control conditions on vehicle progression and its further implication on coordinated operation of traffic signals.

Giving due consideration to these key aspects, it is necessary to develop the nuggets of wisdom for successful implantation of signal coordination for a given roadway scenario. Majority of the past investigations were primarily carried out in the backdrop of developed countries where lane based traffic system with (nearly) homogenous vehicular stream are predominant. In contrast to that, for developing countries such as India, the benefits of signal coordination are not found appreciable and, therefore, rarely it has been adopted as a measure to improve urban mobility. This is due to the fact that learning and wisdoms on several elementary aspects of signal coordination are yet to be validated in the context of developing countries. In this regard, identification of the elementary issues and formulation of their way outs are still awaited.

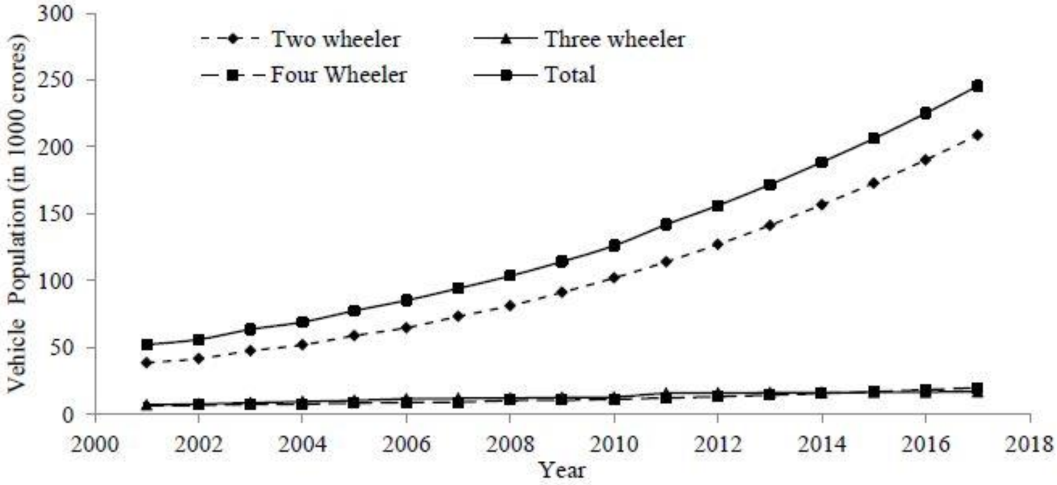
Therefore, in the present work, a detailed review of the familiar practices of traffic signal coordination is presented. In light of this review, research needs in the context of developing countries are identified. The manuscript is organized in five sections. Section 1 provides a brief discussion on past studies highlighting the significance of traffic signal coordination in the context of urban mobility and sustainable urban transportation system. In Section 2, characteristics of urban roadway system, prevailing in developing countries such as India, are briefly described. State of the art practices in coordinated operation of traffic signals are presented in Section 3. Key research areas for developing countries are identified in Section 4. Section 5 provides the conclusion of the study.

2. ROADWAY SYSTEM IN DEVELOPING COUNTRIES

Traffic congestion is a major concern in developing countries such as India. Most of the urban agglomerations in India are experiencing a severe demand-supply imbalance in the road transport system. Physical constraints in expanding right of way (ROW); impedance due to interaction of roadway system and abutting land-use (reasoned by direct access/regress from abutting land-uses), encroachment of designated right of way (due to unauthorized livelihood activities within a ROW), haphazard roadside activity (such as parking maneuverings, pedestrians spillage over carriageway, on-street/ street side hawking etc.) are to name a few, deterring the urban mobility. In addition to these deterrents, issues such as poor enforcement of traffic regulatory measures; lack of focus to strategize the micro-level improvements (such as making provisions for channelizing turning movements at signalized nodes, augmenting capacity of on-street transit stops, facilitating off-street transit stops for lowering the roadway impedance, etc.), limited cooperation from communities to dissolve the local traffic deterrents (such as encroachment due to the small-scale market activities), and many more issues are inhibiting urban mobility in India. Moreover, the roadway system in urban India is evidently different from other parts of the world. It is primarily characterized by non-lane based mixed traffic environment.

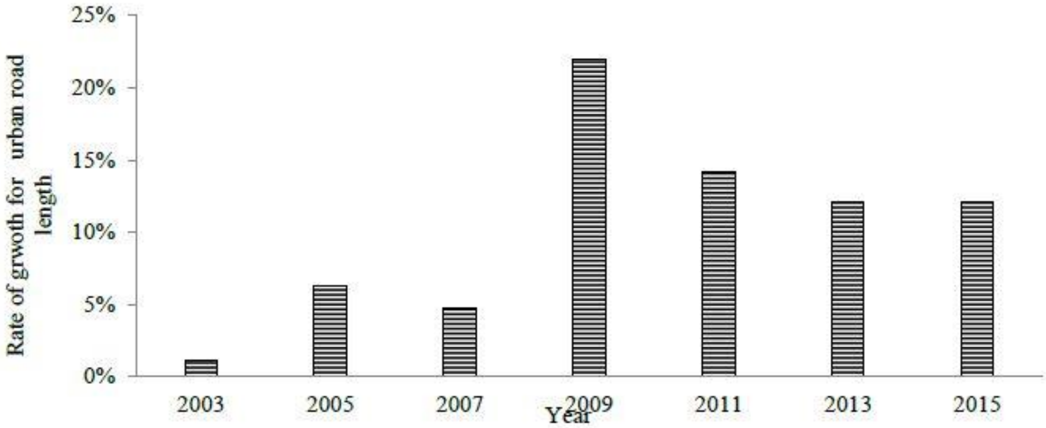
In non-lane based mixed traffic environment, based on space availability, vehicles are found moving across the entire width of a carriageway. Unrestricted movement of vehicles thus makes lane concept invalid (Arasan and Koshy, 2005). Driving behavior of vehicles is equally influenced by lateral and longitudinal spacing (Arasan and Arkatkar, 2010). A large span of variations in physical configuration (i.e., wheelbase, axle width, ride height, etc.) and vehicle performance characteristics (viz. acceleration/ deceleration behavior) are found over different vehicle classes. Such variations result in a wide range of desired speed in a mixed vehicular stream (Mehar et al., 2014; Dey et al., 2006). Apart these, unique characteristics of vehicle maneuverings in non-lane based mixed traffic environment are observed over vehicle seeping, tailgating, swerving, traveling abreast, etc. (Manjunathan et al., 2013; Munigety and Mathew, 2016). In the scenario of an intersection level traffic operation, an opportunity for such vehicle maneuverings results in dense and uneven arrangements of vehicles in a queue. Such queuing pattern causes uneven profiles of vehicle discharge from a signalized stop-line (Sharma et al., 2009). Moreover, vehicles with limited maneuverability induce a significant level of internal friction to other vehicles in a traffic stream (Arasan and Koshy, 2005).

In recent decades, India has also experienced rapid urbanization and economic development, and therefore a steep rise in travel demand in general. In due courses, increase in disposable income reflects a paradigm shift in vehicle ownership in urban India, particularly in the segment of personalized mode of transport (cars and two-wheelers). High growth in vehicle population, such as annually 10 per cent during last decade (2001-2011) has resulted into doubling of the country's vehicle fleet by every 6 to 7 years (MORTH GoI (1), 2012). Increase in population of different vehicle types, predominantly used in urban India, is shown in Figure 2. Figure 2 indicates an exponential growth in total vehicle population over last two decades which eventually generated a high volume of traffic on urban corridors. But expansion of road infrastructure did not follow a similar pace. The noteworthy growth in motor vehicle population has outstripped the modest growth in the roads network of 3.3% (MORTH GoI (2), 2012). Bi-annual rate of increase in urban road length in India is shown in Figure 3. Figure 3 indicates an uneven trend of increase in urban road length from the year 2001 to 2015. Such differential paces of growth between vehicle population and road infrastructure have resulted into a sheer imbalance between demand and supply of urban transport in India.



Source: mospi.gov.in

Figure 2: Population of Two wheeler, shared Three wheeler, and Four wheeler over last two decades.



Source: mospi.gov.in

Figure 3: Bi annual rate of increase in urban road length from 2001 to 2015

This demand-supply imbalance along with frequent occurrences of incidents and accidents have aggravated the onset of peak-period congestion and reshaped the travel pattern in urban India such as trip makers" choice of route, mode of travel, etc. As a result, compared to the recently developed planned cities, majority of the age old cities have been facing acute demand-supply mismatch. This resulted in limited roadway width, high level of congestion, bottleneck, increase in travel time and several other relevant problems, such as vehicle emission, local air pollution, and consumption of petroleum fuels which are now at aggravated stage. The situation is further exacerbated by the impractical design of roadway features to accommodate different vehicular modes, with varied level of desired speeds, within the same right of way (ROW). Therefore, in recent decades, roadway congestion has severely constrained the urban mobility, and eventually, it has become a federal concern. It is steadily getting worse each year converting a peak hour into a more extended peak period, and thereby, increasing the travel delay experienced by urban commuters. The present scenario of peak period congestion in four metropolitan cities in India (e.g., Delhi, Mumbai, Bangalore, and Kolkata) and several cities in Eastern Asia is shown in Figure 4. As indicated in Figure 4, % additional time to travel in peak period in Indian cities is nearly 1 to 2 times higher than the average. For these four cities, congestion cost is around 922 billion US Dollar per year (Chin et al., 2018).

Source: Chin et al. 2018

Figure 4: Peak period congestion in a few selected cities in India and Eastern Asia

In general, intervention in several facets is necessary to curb the demand-supply imbalance, and reduce the externalities in urban areas. These measures may be grouped into three categories, namely, (a) demand management measures, (b) supply management measures, and (c) improved traffic control measures. Over the years, in many parts of the world, a wide range of measures such as flexible work schedule, compressed work week, telecommuting, ridesharing, vanpooling, levying congestion cost, etc. were tested to manage the urban travel demand but such measures were found useful over the long time horizon (Rhodes and Shogren, 2006; Gov. of Montana USA, 2018). The effectiveness of such demand management measures was also found to vary depending on users" socio-economic profile, their acceptability of a proposed measure for a given urban scenario (Farzanah, 2005; Gov. of Montana USA, 2018). In contrary, supply management measures such as physical augmentation of existing facility, improvement in a mass transit system and its ridership; and development of new transport infrastructure were found suitable to bring down the demand-supply imbalance over the short time horizon (Rhodes and Shogren, 2006). However, in several parts of the world such as in developing countries, fiscal constraint, limited right of way (Luten et al., 2004), deficiency in network planning and land-use allocation, etc. impose severe challenges to the effective application of supply management measures. In place of that traffic control measures may be formulated to compensate for the deficiency in implementing supply management measures. Harwood (1987) emphasized that corridor/network specific traffic control strategy is critical for maximum utilization of available roadway capacity. In an urban road network, traffic signals have a significant role in providing safe and efficient operation of conflicting vehicular movements. Therefore, selection of suitable strategy(s) to operate the traffic signals (isolated or coordinated; fixed or actuated), and the development of signal timing plans are identified as the key focus areas of Arterial Management Program in North America (US DoT, 2010; Gov. of Toronto, Canada, 2016). This calls for better utilization of existing facility through efficient traffic control and management.

Based on the previous discussions on the (a) prevailing state of roadway and traffic conditions in developing countries such as India, and (b) the potential benefits from improved traffic control measures in reducing the demand supply imbalance in urban transportation system, a comprehensive investigation is very much pertinent to appraise the suitability of signal coordination in developing countries. In this context, the manuscript aims to present a detailed insight into the familiar practices of traffic signal coordination, as reported in available literature. Various aspects of signal coordination e.g. modeling of vehicular movement under signal coordination, design of coordination plan, and its operational aspects / issues are thoroughly covered in the manuscript. Further, the key areas of further research in the context of developing countries are also identified. Hereafter, three terms namely, traffic signal coordination, coordinated operation of traffic signals (COTS), and coordinated traffic signal system are used alternatively.

3. STATE OF THE ART PRACTICES IN COORDINATED OPERATION OF TRAFFIC SIGNALS

A coordinated traffic signal system provides a smooth progression of vehicles along a traffic corridor or throughout a network of major corridors (NTOC 2005, Homburger et al. 2001). While designing a coordinated traffic signal system, signal timing plans are developed to minimize vehicle stops along a corridor. For fixed time traffic controllers, Time of Day (TOD) signal coordination plans are developed to cater large variations in approach volumes. In the era, when computers were rarely used for engineering application, a TOD plan was usually developed by manipulating a signal cycle length in time-space diagrams (Marsh, B.W. 1927; Petterman, J.L. 1947). The green bands were usually modelled using pins and thread on a drawing board, and cycle lengths were represented by paper strips. This was the practice which emerged around 100 years back, in the 1920s (Day et al., 2009). Since then several other design techniques/methods namely algorithm for maximizing platoon bandwidth (Morgan and Little 1964), RRL combination method (Hiller 1965/66), TRANSYT (Robertson 1969), PASSER (Messer et al. 1973), MAXBAND (Little et al. 1981), MUTIBAND (Gartner et al. 1990) and SYNCHRO (Husch D. 1998) were developed for optimizing a corridor/ network level signal coordination plan. In recent times, posterities of PASSER such as PASSER II (Chang and Messer 1991), PASSER III (Venglar et al., 1998), PASSER IV (Chowdhury and Messar 1993) and PASSER V (Chowdhury et al. 2002) become popular for developing a network level signal coordination plan. Hereafter, the terms such as TOD plan, signal timing plan, signal coordination plan, and arterial signal coordination are used alternatively.

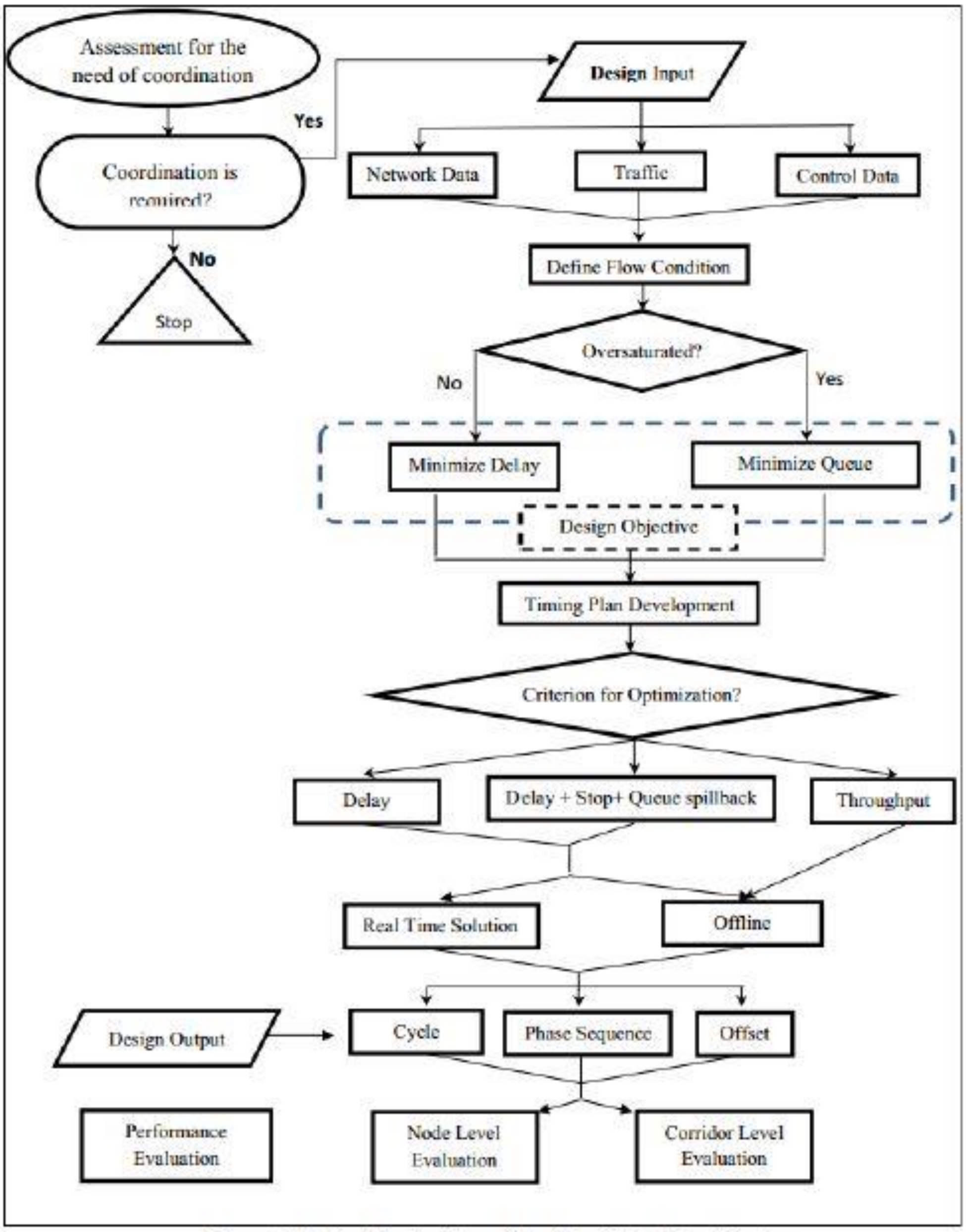


Figure 5: Typical Design Steps of the Coordinate Signal System

During the evolution of these design techniques/methods, major efforts were given to address the complexities which arise from the mutual interaction of roadway, traffic and control conditions. Giving due consideration of these complexities, design of a coordinated traffic signal system may be framed up as a multi-step process. Based on the literature review, a design framework for coordinated traffic signal system is proposed here (shown in Figure 5). As indicated in Figure 5, design steps may be broadly identified as a) Assessment for the need of signal coordination, b) Selection of design approach, c) Selection of the design tool, d) Development of signal timing plan, and e) Performance evaluation. A detailed discussion of these design steps is presented through sub-sections 3.1 to 3.5.

3.1. Primary Assessment for the Need of Signal Coordination

Prior to design, it is required to assess the suitability of coordinating a pair of traffic signals in lieu of their isolated operation. Prevailing state of system parameters (e.g., roadway, traffic and control parameters) are usually investigated to assess the need of signal coordination, and it is repeated over each traffic period of interest (Bonneson et al. 2009, Koonce et al. 2008). Several evaluation measures are proposed by researchers to assess the need for coordinating traffic signals. A brief discussion on these measures are presented in the following paragraph.

One of the early research works stated that „any two or more signals which are less than one-half mile apart or within a cycle length (which may be more than one-half on a high-speed approach) should be interconnected‟(US DoT 1975). In another study, Yagoda et al. (1973) proposed a unitless parameter, „Coupling Index‟, to measure the relative need for coupling or „coordination‟ among the number of intersections. This index is computed as a ratio of hourly link volume to link length. Larger the value of this index, it will be more beneficial to coordinate the adjacent traffic signals. However, no threshold value for this index was reported below which coordination should not be attempted. To resolve this limitation, Orcutt (1993) proposed the following guidelines to evaluate the need for signal coordination. A low value of Coupling Index such as lesser than 0.3 unlikely to benefit from coordination; a value in between 0.3 and 0.5 likely to benefit if access point activity along the link is low, and turn bays are provided on major-street at each signalized intersection; a value higher than 0.5 indicates benefits are to be occurred. To apply these guidelines, link length is needed to be measured in feet. Experiences in using coupling index indicate that segments shorter than 2600 ft (i.e.792 m) will typically be coordinated, and those longer than 5300 ft (i.e.1615 m) should not be coordinated (Bonneson et al. 2009). However, Koonce et al. (2008) recommended that when intersections are still within ¾ mile (i.e. 1.2 km) of each other, they should be coordinated. For spacing greater than ¾ mile the need for coordinating a pair of nodes was suggested based on the review of approach volume and vehicle platoon behaviour. Hook and Albers (1999) modified the „Coupling Index‟ parameter by introducing a distance term in squared form (based on Newton‟s law of gravitation), thus giving more emphasis to link volume than link length to adopt signal coordination. Christopher and Kiddle (1979) suggested signal spacing and ratio of link travel time to cycle length as two major aspects for measuring the quality of vehicle progression. They found that good progression could be achieved when the signal spacing was fairly uniform, and the value for the ratio of travel time to cycle length was in the range of 0.40 to 0.60. Chang (1985) also referred „Ideal Progression Spacing‟ as link travel time in between one-third to one- half cycle length multiplied by the design speed. Consideration of cycle time indicated the due significance of total intersection approach volume and conflicting volumes, in addition to through volume for adopting signal coordination. Signals separated by a distance of a mile or more may be coordinated if the variation of traffic along the links (to be coordinated) is less (FHWA 1975). Gordon and Tighe (2005) advised interconnecting adjacent traffic signals when the distance is less than approximately 70 times the desired average speed in ft/sec (m/s). Other measures were also developed to decide the need for coordinating two adjacent intersections. These measures included “Linking Factor” as used by Computation in Winston Salem, NC, and “Offset Benefit” as described in NCHRP Report 3-18 (3) (Henry 2005). These measures primarily considered the mutual interaction of link length and approach volume across the links.

Several other aspects were also investigated by researchers while proposing the measures for assessing the suitability of coordinating traffic signals. Chang and Messer (1986) proposed Interaction Desirability Index (I) to assess the coordination requirement of each link of an isolated signalized intersection. The evaluation criterion considered the approach volume imbalance condition, and platoon dispersion effect at that intersection. Approach volume imbalance index considers the proportion of through volume coming from upstream intersection to total approach volume (which consists of turning volumes, mid-block generated volumes), which represent the relation between flow rates and platoon formation. The evaluation of ‘I’, took into account the number of departure lanes at upstream intersection and number of lanes available for arrival at downstream intersection. Value of ‘I’ ranging from 0.50 to 1 was suggested as appropriate to justify the requirement for coordinating the concerned intersections, whereas the value of ‘I’ between 0 and 0.25 was used to suggest no requirement of coordination. The range of values for ‘I’ from 0.25 to 0.5 requires further evaluation of other measures to decide the interconnection. However, those measures were not specified by Chang and Messer (1986). Hook and Albers (1999) also referred to two more parameters, namely (i) Strength of attraction and (ii) Coordinatability factor (used in Synchro) to assess the need of signal coordination. Vehicle platoon aspects, traffic volume, link speed, and link length were the common parameters which were considered for evaluation of these two factors. In addition to these, Coordinatability factor also considered natural cycle length (which ensures minimum delay at an intersection) and queuing of vehicles to determine the desirability of coordination. Threshold values for these two parameters to adopt signal coordination are given in Table 1.

Table 1: Threshold Values of Strength of Attraction and Coordinatability Factor

Parameter	Requirement of Coordination		
	Not desirable	Desirable	Critical
Strength of Attraction (*SA*)	<0.50	0.50 to 2	>2
Coordinatability Factor *(CF)*	<20	20 to 80	>80

Source: Hook & Albers (1999)

The evaluation measures for assessing the need of signal coordination, discussed in previous paragraphs, primarily deal with the time of day operation of traffic controllers and therefore, do not consider the stochastic variations in a traffic system. Whereas for real-time assessment, inherent randomness of a traffic system such as variation in vehicle arrival pattern at a d/s stop-line, phase events of traffic signals should be studied. Vehicle arrival data, obtained from loop detectors ahead of a d/s signal may be used for real-time assessment. In this context, the decision to coordinate or not to coordinate a downstream traffic signal varies over signals cycles, and accordingly, a downstream signal is operated (Day et al., 2011). Simulation tool such as VISSIM, CORSIM, SimTraffic may also be applied, together with a signal timing program such as SYNCHRO, TRANSYT 7F for assessing the coordination requirements, and a section boundary may be identified using a micro-simulation model (Gordon and Tighe 2005). Use of such tools at the planning stage may result in efficient corridor/ network level urban traffic management.

Discussion:

A comparative analysis highlighting the relative importance of roadway, traffic and control parameters for coordinating traffic signals is presented in Table 2. Traffic movement pattern at an intersection (viz. turning volumes and their share in total approach volume) is a pivotal measure to decide the need of signal coordination. It is the share of through traffic in total approach volume, which primarily raises the need for signal coordination. Signal control parameters (cycle length, phase settings) are also equally important to ensure the bunching of through traffic at the u/s stop- line. In addition to these, link attributes such as link length, driveway density significantly influence the vehicle progression along a coordinated link. All these have the causal effect on other parameters such as link speed, vehicle travel time, and queue length, thus affecting the signal coordination. From the intersection layout point of view, lane assignment for through traffic critically influences the platooning of vehicles at stop-line. However, variation in the desired speed of vehicles classes is yet to be considered by these measures to determine the need of signal coordination. Consideration of this aspect is critical for mixed traffic environment prevailing in developing countries. In the context of developing countries, mutual interaction of some of the system parameters such as platooning of vehicles at stop-line, quality of vehicle progression over length (emphasizing the non-lane based vehicle driving behavior) , variation in link travel time (emphasizing the heterogeneity in vehicle mix) etc. are also need to be investigated. In the context of signal control parameters, the effect of phase settings on bunching vehicles at stop-line requires investigation as the efficiency of signal timing plan, developed through traditional macroscopic signal design tools are often found suitable for non-lane based mixed traffic operation.

Table 2: Roadway, Traffic and Control Parameters
Considered for Assessing the Need of Signal Coordination

Roadway parameters	[1]	[2]	[3]	[4]	[5]	[6]	[7]	[8]
				Roadway Geometry				
Signal spacing	+			+	+	+		
Intersection layout			+					
				Traffic Condition				
Approach volume	+		+	+	+			+
Side street volume			+					
Mid-block volume			+					
Link speed		+		+	+	+		
Link travel time								+
Platoon dispersion			+	+				
Arrival profile							+	
Queue length					+			+
				Control Parameter				
Cycle length		+			+		+	
Phase settings							+	

[1] Yagoda et al. 1973; [2]: Chang, E.C-P 1985; [3]: Chang, E.C-P. et al. 1986;[4]:Hook & Albers 1999 (*SA*); [5]:Hook & Albers 1999 (*CF*); [6]:Koonce et al., 2008; [7]:Day, et al. 2011; [8]:Transyt-7F Manual

3.2 Design Approaches of Signal Coordination

3.2.1 Selection of design approach

Selection of design approach is ruled by the prevailing state of roadway, traffic and control parameters. The mutual interaction of roadway, traffic, and control parameters may result in 'signal cycle failure' which is usually represented by residual queue, queue spill-over or network grid-lock condition (Gattmen et al. 2012). Therefore, for a given roadway scenario, the measure termed as 'volume to capacity ratio of lane groups' is useful to determine a level of approach volumes as under-saturated or over-saturated approach volume (HCM 2010, Gazi 1964). This categorization of approach volumes aids to select a 'design objective' for developing a TOD plan. Based on the design objective a design approach is usually decided. Therefore, prior to discussing the 'design approach,' a brief discussion on 'design objective' is presented in the following paragraph.

An appropriate design objective should be identified for developing a TOD plan (Hajbabaie and Benekohal, 2013). Performance measure(s) are normally used as 'design objective.' In some cases, a number of measures may be used in combination by assigning weightages to them. Delay and its associated measures such as number of stops and link travel time are some of the widely used performance measures for optimizing the signal control parameters (e.g., cycle length, phasing sequence, split durations, and offset). Delay minimization was first used in RRL combination method, developed way back in the 1960"s by Road Research Laboratory, England to estimate cycle length and offset for a signalized arterial (Hiller, 1965/66). Later Robertson (1969) developed the TRANSYT model with an improvised version of delay based objective function by adding the number of stops with weightage. Both these measures were also used together as design objective by Dutta and McAvoy (2012) and Perrin et al. (2001). However, Mirchandani and Head (2001), Lucas et al. (2007) and Jhavery et al. (2003) minimized only delay to optimize a signal timing plan. Sun et al. (2006) minimized the link travel time to develop a dynamic signal optimization tool for an arterial network. Platoon bandwidth is another measure which was used by Morgan and Little (1964) to optimize signal timing plan. Bandwidth maximization is also used in several design tools such as PASSER II, MAXBAND and MULTIBAND (Messer et al. 1973; Little et al. 1981 and Gartner et al., 1990). Lan et al. (1994) combined two conflicting measures viz. delay minimization and bandwidth maximization simultaneously to optimize a signal coordination plan. Wallece et al. (1998) introduced a new measure namely number of progression opportunity (PROS) provided to motorists for reducing vehicle stops and delay. By definition, PROS is the number of successive green signals a motorist would able to pass at design speed without stopping. All of these design objectives discussed so far are more suitable for under-saturated approach volumes. For over-saturated approach volumes, care is taken to minimize the residual queue length along the critical approaches (Koonce et al. 2008, Bonneson et al. 2009). However, based on the spatial and temporal extent of residual queue, strategies may be formulated differently. A detailed discussion on this aspect is given in section 3.2.1.2.2. For an over-saturated traffic corridor, throughput maximization with queue management was also suggested as a better-suited design objective (Gettman et al. 2012). Hajbabaie and Benekohal (2013) introduced two more design objective viz. trip maximization and weighted trip maximization to optimize a signal timing plan for an over-saturated network.

Over the years, several design approaches were developed using all these design objectives. These design approaches may be categorized as (a) Design approaches for under-saturated approach volumes and (b) Design approaches for over-saturated approach volumes. Design approaches for under-saturated approach volumes may be further categorized as, a) Bandwidth based signal design approach and b) Delay based approach of signal design. A brief discussion of these design approaches is presented in following sub sections.

3.2.1.1 Design approaches for under-saturated approach volumes

3.2.1.1.1 Bandwidth based approach of signal design

Bandwidth is purely a function of signal timing plan as it is oriented in time and space (Koonce et al., 2008). It is primarily used to determine the capacity or maximum vehicle throughput across a signalized corridor. Bandwidth also reflects the measure of progression opportunities along a signalized corridor (US DoT 2015), and is defined as the maximum amount of green time for a designated movement to move across an arterial corridor. It can be defined for two consecutive intersections (referred to as link bandwidth) or along an entire arterial (referred to as arterial bandwidth). The bandwidth is calculated by taking the difference between the first and last vehicle trajectory that can travel at the progression speed without impedance and is typically measured in seconds. Bandwidth efficiency (E) and bandwidth attainability (A) are two relevant measures which are used to normalize the bandwidth with respect to cycle length. E may be estimated using Equation 1.

$$E = \frac{B_A + B_B}{2C} \quad (1)$$

Where, B_A and B_B are bandwidths in forward and backward directions, respectively; and C is cycle length. Four levels of vehicle progression are defined based on bandwidth efficiency (E). E in the ranges of 0.00 to 0.12, 0.13 to 0.24, 0.25 to 0.36 and 0.37 to 1.00 were categorized as 'Poor', 'Fair', 'Good' and 'Excellent' level of vehicle progression (Koonce et al. 2008).

Bandwidth attainability (A) is the ratio of bandwidth to a sum of two-way through green time at the most critical intersection of a corridor. It reflects percentage use of available green time by a bandwidth. Bandwidth attainability is estimated by Equation 2. Attainability, as with efficiency, is typically reported as a unit-less decimal or percentage. 100 % attainability implies that at the most constrained intersection, the coordinated split is fully used by a green band.

$$A = \frac{B_{max}}{G_{o,j} + G_{i,j}} \qquad 2$$

where, B_{max} is optimal system bandwidth, $G_{0,j}$, $G_{i,j}$ is inbound and outbound green time, respectively of intersection j.

Bandwidth-based signal design approach is an improved version of time-space diagram technique. Morgan and Little (1964) proposed the first improvement on 'time-space diagram' technique using a computational algorithm. This computational algorithm estimates the offsets with an aim to maximize the platoon bandwidth. However, its application is limited to linear arterials (Seddon 1971). Later several improvements were carried out on bandwidth principal with a more systematic approach. It resulted into development of a series of computer-oriented design tools such as PASSER (Messer et al., 1973), MAXBAND (Little et al., 1981), and MUTIBAND (Gartner et al., 1990). In the following paragraph, key aspects of bandwidth based signal design approach are discussed.

Bandwidth is highly dependent on vehicle demands in non-coordinated phases and offsets along the arterial. It also varies based on the traveler"s origin and destinations along a coordinated corridor. The more vehicles traveling the length of a coordinated corridor, without making turning movements, bandwidth becomes more important as a performance measure and vice versa (Koonce et al. 2008). Therefore, the bandwidth-based signal design approach is suitable for an arterial corridor with a lesser number of vehicle turning movements. But with an increase in the number of signals, bandwidth-based solution becomes impractical. Such findings led to the concept of making partition of the whole arterial system into a number of subsystem. Tian and Urbanik (2007) developed a system partition technique based on a heuristic approach which seeks maximum bandwidth in the peak direction and the best partial bandwidth in off-peak direction. However, the study did not consider factors like volume to capacity ratio and link length. It only considered 100% attainability of the maximum bandwidth for selection of the number of signals in the subsystem. Peifeng et al. (2011) developed a method for maximization of variable bandwidth, without the inclusion of loop constraints which are very common in Mixed Integer Linear Programming (MILP) based bandwidth optimization models (e.g., MAXBAND, MULTIBAND, and BANDTOP). They proposed different solution methods for bandwidth optimization at an arterial level and network level. Considerations of cycle length, coordinated splits, and coefficients of weighted bandwidth are still under research to obtain a maximum bandwidth. Lin et al. (2010) developed a Mixed Integer Non Linear Programming (MINLP) based solution to optimize the bandwidths for continuous signals along a signalized arterial. It considered volume ratio of minor-street to major-street approach. However, the effects of platoon dispersion, non-uniform arrival, lane changing / overtaking, etc. were not considered in study methodology. Lu et al. (2012) proposed a new two-way band width maximization model that aimed to address some of the difficulties in MAXBAND solution, and it considered two scenarios. When there is no possibility of two-way progression, the model first guarantees maximum total bandwidth and then automatically prorates the bandwidth fully to the direction with higher vehicle demand. In favorable condition, the model automatically prorates the bandwidth in a ratio as close to the ratio of bandwidth demands. This study helped to explain the bandwidth solution under different scenarios, not addressed by MAXBAND. However, optimal settings of the proration impact factor are still under investigation.

Among the different descendants of PASSER, PASSER II and PASSER IV were developed for optimising the corridor and network level signal coordination plan respectively. In PASSER II, minimisation of delay and bandwidth optimisation both are duly considered in a two-stage process. Delay estimation for coordinated and non-coordinated links is carried out by two different models. PASSER II uses a graphical technique to develop the signal coordination plan. PASSER IV was developed with an aim to improve the limitation of MAXBAND 86 (Messer et al.1986; Chang et al., 1989). MAXBAND 86 uses a simplistic model to develop a signal coordination plan for a multi-arterial network, but it does not optimise the green splits. Link to link speed variation, dual consideration of bandwidth and delay optimisation, and scope of giving priority to a specific arterial as well as the direction of traffic are some of the practical features which are duly considered in PASSER IV (Chowdhury and Messar 1993). PASSER V (Chowdhury et al. 2002) eliminates some of the limitations of PASSER II. It results in smaller cycle length and better progression bands than that of PASSER II. PASSER V has the advantage of being useful in over-saturated approach volumes.

3.2.1.1.2 Delay based approach of signal design

In delay based approach of signal design, green splits are rationally allocated to several „lane groups" or signal phases to achieve a target volume to capacity ratio (Roess et al. 2010). In addition to this rationale, vehicle arrival profiles at a d/s signal significantly affect the utilization of green splits. Based on these facts, flow profile method emerged in the United Kingdom in the 1960s. In this method, the principal of 'vehicle progression' was introduced by Pacey (1956), Hiller and Rothery (1967) to develop the delay-offset relationship. The delay–offset relationship became the core of RRL combination method of signal coordination (Hillier 1965, 1966) which was developed by Road Research Laboratory in United Kingdom. Two key assumptions of RRL combination method are a) a common cycle length or sub-multiple of it across all the coordinated signals and b) delay experienced by motorists, using the coordinated links, is only due to the traffic signals which are at the end of the links. The first assumption leads to a prior estimation of cycle length for a traffic signal which has heaviest approach volume, and it becomes the common cycle length. The second assumption is used to obtain the delay offset relationship across the coordinated links, and it requires the data of vehicle arrival profiles at each d/s stop-line. Several sets of offset and delay along the links are estimated and finally, an optimal set of offsets are found by minimizing the network/corridor-level total delay (Farzaneh 2005). An improved version of RRL combination method was proposed by Robertson (1969). Robertson (1969) proposed a new index giving due consideration to vehicle stops and delay both. The new index considered a stop penalty of 4s per vehicle stop in addition to the average delay on links (vehicle-hour/hour) and in doing so, the index addressed the issue of a green wave along a corridor/network. Based on this philosophy, Robetrson (1969) proposed the earliest version of TRANSYT model for developing a signal coordination plan. In delay based approach of signal design, modelling of vehicle arrivals at coordinated stop-lines was found as a key aspect. Effective utilization of coordinated splits depends on vehicle arrival pattern at coordinated stop-lines.

Non-uniform vehicle arrivals at coordinated stop-lines may result in an additional delay to vehicles using non-coordinated splits. Therefore, forecasting of vehicle arrivals at a coordinated stop-line is found to be important to estimate offset and split duration in a coordinated phase (Manar and Bass 1996; Farzaneh 2005). In this context, the concept of 'vehicle platoon' and its modelling is discussed in detail in following paragraphs.

Once released from a signalized stop-line, vehicles move towards downstream (d/s) direction in bunches which are usually termed as „vehicle platoon". A vehicle platoon initially moves in a tight form and tends to disperse as traveling further downstream (Rouphail, et al. 1992; Jiang et al. 2006; Bie et al. 2013; Farzaneh 2005). Dispersion of vehicle platoon occurs due to variation in motorists" desired speed. Vehicle maneuverings such as acceleration/ deceleration/ lane changing, approach volume and vehicle mix, roadway geometry (such as link layout, grade, curvature (Yu 2000) and roadside activity (e.g., on-street parking, pedestrians spillage over carriageway (Denny 1989; Tarnof and Parsonson (1981) are the major contributory factors which result in speed variation, and thus, affect the dispersion of a vehicle platoon. In the context of signal coordination, dispersion of vehicle platoon has been investigated by several researchers (Chang and Messer 1986; Denney 1989; Rouphail et al. 1992; Manar and Bass 1996; Yu 2000; Qiao et al. 2001; Jiang et al. 2006; Bie et al. 2013). These investigations may be categorized (Glomb A.J. 1989) as (a) experimental investigation and (b) theoretical investigation. In experimental investigations, vehicle platoon behavior was analyzed based on vehicle flow data collected from signalized corridors. One of the focus areas of early experimental investigations was to study the integrity of a platoon (i.e. how many vehicles still remains in a platoon) over several downstream (d/s) sections.

Lewis (1953) concluded that platoon dispersion increases linearly with the distance from an upstream (u/s) signal. For a low friction corridor, around 90% vehicles were found to be in a platoon when a d/s section is within 0.75 miles (i.e. 1.21 km) from u/s stop-line (Graham and Chenu 1962). These findings contradicted the rule of thumb in Manual on Uniform Traffic Control Devices for Streets and Highways (MUTCD) (US DoT 2015) which recommends to coordinating the signals when they are within ½ a mile (i.e. 800m) apart. Another experimental study, carried out by K. Bang (1967) reported that vehicle platoons with larger size (i.e. the number of vehicles in it) disperse less, as vehicles get less opportunity to overtake. Nemeth and Veceilio (1970) reported that the coefficient of the variation of velocity within a platoon is a good indicator of roadway and traffic condition. It was also found that platoon velocity and mean headway in a platoon decreased with an increase in platoon size.

On other hand, theoretical investigations resulted into mathematical models to analyze the vehicle platoon behavior. It also included the validation of a model by field experiments. These models are useful for forecasting vehicle arrivals at d/s signal and estimate the vehicle passing time across a d/s stop-line. Both vehicle arrivals and vehicle passing time across a d/s stop-line were found to be influenced by the platoon size, speed, and distribution of vehicular speed within a platoon (He et al., 2006). Three major focus areas of research namely kinematic wave theory (Lighthill and Witham, 1955), diffusion theory (Pacey, 1956), and recurrence theory (Robetrson, 1969) are found for modeling of vehicle platoon behavior. These models may be used to estimate offset and green split duration in a coordinated phase (Farzaneh 2005) while developing a time of day (TOD) signal coordination plan. Lighthill and Witham's 'kinematic wave theory' (LWR) (1955) uses an equation of continuity to model a change in traffic state due to the presence of a signal. A parameter, termed as 'shock wave' was introduced to explain the transition between two traffic states, and further used to describe vehicle platoon behavior along a signalized link. But, the LWR model fails to reflect platoon dispersion in the downstream direction of a signal (Seddon, 1971). Moreover, due to the requirement of precise mathematical description of a flow state and computational complexity, LWR model was not found suitable beyond evaluation (Denny 1989). In that sense, Pacey’s diffusion model is remarkable simpler and was built upon a pure kinematic theory (Denny 1989). With an assumption of normally distributed vehicular speed in an unrestricted overtaking condition, Pacey‟s model predicts the downstream flow profile as a continuous function (Pacey, 1956). Pacey (1956) derived the distribution of link travel time as transformed normal distribution. Based on field testing on two expressways in England, suitability of the Pacey‟s model was established model under low to moderate traffic flow condition along shorter links (0.4 miles i.e 0.64km or less) (Glomb 1989). It was later supported by Herman et al. (1964). Robertson used the discrete version of Pacey’s model to forecast vehicle arrivals (Day and Bullock 2012). It is primarily a semi-empirical model developed using an exponential smoothening technique (Glomb 1989). Robertson assumed a binomial distribution for vehicle travel time (Rouphail 1992) which overcomes the biased assumptions in Pacey‟s model, i.e. vehicles moves toward downstream signal with a constant speed with free overtaking (Manar and Bass 1996). Denney (1989) used the general concept of Pacey’s diffusion theory and introduced a new dispersion model. Instead of using a transformed normal distribution for travel time, Denney (1989) used an observed distribution of travel time derived from the field study. Qiao et al. (2001) developed a traffic dispersion model based on a three-layer Back-Propagation neural network. A field dataset was used to train the network, and the trained network was found to forecast the flow pattern accurately. Wu et al. (2014) proposed a macroscopic platoon dispersion model for a mixed vehicular stream (having car and busses within a same right of way) and validated the same using virtual traffic flow data.

One of the significant differences among all these dispersion models is the assumed distribution of vehicle travel time or speed, which are used to forecast downstream arrival of vehicles. In Pacey’s model, vehicular speed was modeled as normally distributed variable; whereas, Robertson‟s recursive model assumed a shifted geometric distribution of vehicle travel time. Various researchers under several roadway scenarios have also investigated the suitability of these two assumed distributions. Wang et al. (2003) found that Robertson‟s model is more suitable for modeling the distribution of travel time on short links. Lognormal distribution of speed was found more suitable for longer links. Wie M.et al. (2012) modified the Pacey‟s model by assuming the distribution of speed as truncated normal distribution. Field investigation carried out by Yi et al. (2006) has confirmed distribution of speed of vehicles within a platoon as a normal distribution. Table 3 summaries various key findings of the distribution of speed and travel time within a vehicular platoon by several researchers.

Table 3: Distribution of Speed and Travel Time as Used in Platoon Dispersion Models

Studies	[1], [2]	[3]	[4]	[5]	[6]	[7]
Parameter						
Vehicular speed	Normal distribution			Lognormal distribution (long links)		Truncated normal
Link travel time	Transformed or reversed normal distribution	Shifted geometric distribution	Observed distribution		Enhanced geometric distribution	

[1]: Pacey (1956); [2]: Yi et al. (2006); [3]: Robertson (1969); [4]: Denney (1989); [5]: Wang et al. (2003); [6] Farzaneh (2005); [7] Wie M. et al. (2012)

From an application point of view, Robertson’s model has been found to be rational for representing the dispersion of vehicle platoon in an under-saturated level of flow (Geroliminis and Skabardonis 2005). Moreover, Robertson’s model requires less computational effort and was also found to be suitable for a large network (Farzaneh 2005). Several signal design tools such as TRANSYT-7F (Robertson 1969), SCOOT (Hunt et al.

1981), SATURN (Hall et al. 1980) and TRAFLO (Lieberman and Andrews 1980) use Robertson's model thus indicating its wide acceptability for developing TOD plans. An overview of Robertson's model, and its application approaches are presented in the following paragraphs.

3.2.1.1.2.1 Robertson's platoon dispersion model

Robertson's platoon dispersion model was formulated using a discrete iterative approach (Denney 1989; Farzaneh and Rakha 2006). Formulation of Robertson's model may be expressed (Farzaneh and Rakha 2006) by Equation 3.

$$q_t^d = F_n * q_{t-T} + (1 - F_n) * q_{t-n}^d \qquad 3$$

Where, q_t^d is vehicle arrival flow at the d/s signal at time t; q_{t-T} is vehicle departure flow at u/s signal at time t-T; T is lag time (time gap between the initiation of green at u/s signal and arrival of the first vehicle at d/s signal); n is the size of modelling time interval over which vehicle counts for u/s discharge, and d/s arrivals are aggregated; and F_n is a smoothing factor. From Equation 2.1, it may be inferred that d/s arrivals in each time interval (q_t^d) are dependent on u/s discharge. q_t^d is a weighted combination of d/s arrival during the previous time interval, q_{t-n}^d and u/s discharge T seconds ago, q_{t-T}.
F_n, the smoothening factor, primarily reflects the quality of vehicle progression between u/s and d/s stop-lines. It is a function of lag time (T) and a shape parameter α which is termed as 'platoon dispersion factor'. The expression of F_n is given by Equation 4. Another parameter β, termed as travel time factor, was introduced to relate lag time (T) with average link travel time (T_a) and is given by Equation 5. While α is a theoretical parameter and β is an estimated parameter.

$$F_n = \frac{1}{1+\alpha T} \qquad 4$$

$$T = \beta T_a \qquad 5$$

Since Robertson's model estimates the d/s arrivals at given time intervals, the model is needed to be applied recursively to estimate the flow (Farzaneh and Rakha 2006). Seddon (1971) rewrote the Equation 6 as,

$$q_t^d = \sum_{i=T}^{\infty} F_n (1 - F_n)^{i-T} * q_{t-i+T} \qquad 6$$

Equation 6 indicates that estimated d/s arrivals follow a shifted geometric distribution, which considers the contribution of an u/s flow in $(t-i)^{th}$ interval to the d/s flow in t^{th} interval. F_n, smoothening factor quantifies the contribution of u/s flow at t^{th} interval that arrives at d/s stop-line at $(t+T)^{th}$ interval. Higher a value of F_n, more will be the d/s arrivals for a corresponding u/s discharge, which in turn represents the lesser dispersion of a vehicle platoon. In general, F_n is found to be within the range of 0 to 1 ,i.e. $0< F_n <1$. Therefore, F_n denotes an inverse relationship with α. However, α is found to vary with n (Rakha and Farzaneh 2006). Although α represents the level of dispersion for a vehicle platoon (Manar and Bass 1996), overestimation of „time span of downstream arrival‟ is a significant limitation for Robertson‟s model (Bie et al. 2013 and Rumsey and Hartley 1972). To overcome this limitation, Bie et al. (2013) reported a threshold level of flow below which corresponding modeling time intervals towards the tail of a forecasted vehicle arrival profile were eliminated. However, the implication of this assumption on an overestimation of dispersion of a vehicle platoon is yet to be investigated.

3.2.1.1.2.2 Approaches for application of Robertson's model

Approaches for application of Robertson‟s model may be categoriesd into three groups namely (a) Use of default values for model parameters, (b) Use of best-fit values for model parameters, and (c) Use of analytical models to estimate values for model parameters. A Brief discussion on studies carried out under these three groups is presented in the following paragraphs.

3.2.1.1.2.1 Use of default values of model parameters

In early 70s of the twentieth century, studies were conducted in United Kingdom and North America under several scenarios of roadway and traffic conditions (Yu 2000; Tarnoff and Parsonson 1981) to recommend the default values of α and β. Three levels of side friction such as low friction, moderate friction, and heavy friction were studied. However, Manar and Bass (1996) found that default values for α and β, instead of locally calibrated values may result in inefficient signal timing plan.

3.2.1.1.2.2 Use of best-fit values for model parameters

Studies, carried out in the later stage, calibrated the Robertson's model to obtain site-specific values of α and β. For a given profile of vehicle discharge at u/s stop-line, most of these studies focused on minimizing the errors between observed and estimated downstream arrivals. Following objective function (Equation 7) (Farzaneh and Rakha 2006) was used to obtain the best-fit value for α.

$$f(\alpha) = \sum_{t=1}^{N}\left[q'_d(t) - q_d^t(t)\right]^2 \qquad 7$$

where, $q'_d(t)$ is forecasted d/s arrival during time interval t and $q_d^t(t)$ is the observed d/s arrivals during time interval t. N is the number of time intervals in forecasted arrival profile.

In some of the studies (Collins and Gower 1974; El-Reddy and Ashworth 1978; Holroyd and Hiller 1969) a fixed value of β as 0.80 was assumed to obtain the best-fit value for α. Such an assumption is usually suitable while a value for α is used in Transyt-7F, a macroscopic signal design tool (Farzaneh and Rakha 2006). Subsequent studies found shortcomings in this approach of calibration (Bass and Lefebvre 1988; Guebert and Sparks 1989; El-Reedy and Ashworth 1989; Manar and Bass 1996). Apart from the best-fit approach of calibration, several other studies (Denney 1989; Bonneson et al. 2005.; Day and Bullock 2012) estimated the parameters of Robertson‟s model by achieving the statistically similarity between observed and estimated downstream arrival profiles. Three level scenarios of side friction, viz. low friction, moderate friction, and heavy friction were conceptualized based on the variation of α. For these scenarios, α was found to be in the ranges of 0 to 0.25, 0.35 to 0.50 and 0.60 to 0.75 respectively. However, these scenarios fail to demonstrate the effect of major traffic state parameters such as speed, flow, and density on vehicle platoon dispersion. To tackle this issue, Bass and Lefebvre (1988) proposed a formula to estimate the smoothening factor, F_n based on link travel time and traffic volume. Later, a parabolic relation between traffic volume and platoon dispersion factor (α) was established empirically by Manar and Bass (1996). α was found to increase with an increase in traffic volume and reaching its highest value when the volume reaches half of an arterial capacity. Thereafter, it decreases with an increase in volume and reaches its minimum value when volume attains arterial capacity. However, this parabolic relation was not found unique overall roadway configurations. Roadway width was also found to have influences on platoon dispersion (Denny 1989). Bie et al. (2013) proposed a set of double Gaussian functions to relate α and normalized flow (volume to capacity ratio at stop-line) for several corridors having variation in roadway width. Four corridors with number of lanes as two, three, four and five were studied. Across all corridors, α was found as high as 0.95 to 0.98 when the normalized flow varied from 0.70 to 0.75. The variation of α with the normalized flow was found to be more rapid for a wider corridor than the narrower one. Wang et al. (2017) proposed a number of non-linear equations to relate variations in α, β and F_n to percentage of buses ($\%_{bus}$) in a vehicle mix. β and F_n were found to have an decreasing trend with the percentage of buses. Whereas, α was found to be increasing with an increase in the percentage of buses.

3.2.1.1.2.3 Use of Analytical Models to Estimate Values for Model Parameters

Functional relations among α, β, and the inherent distribution of travel time were investigated for developing analytical models. Seddon (1972) expressed α as a function of β as given in Equation 8. A sensitivity analysis carried out by Yu and Van Aerde (1995) revealed that estimated flow profiles are independent of any particular distribution of travel time. A number of formulations (Equation 9 and 10) were proposed by Yu and Van Aerde (1995) and Yu (2000) based on statistical features of travel time viz. mean (T) and standard deviation (σ). Use of T and σ aid to validate the influence of site-specific factors such as grade, curvature, side friction, traffic volume, and other sources of impedance on vehicle platoon dispersion.Thus the primary shortcomings of the best-fit approach of calibration were eliminated by these analytical models (Farzaneh 2005). This approach is more suitable for advanced traffic management system (ATMS) where travel time data is acquired on real-time basis.

$$\beta = \frac{1}{1+\alpha} \tag{8}$$

$$\alpha = \frac{\sqrt{1+4\sigma^2-1}}{2T_a+1-\sqrt{1+4\sigma^2}} \tag{9}$$

$$F_n = \frac{\sqrt{1+4\sigma^2-1}}{2\sigma^2} \tag{10}$$

However, the accuracy of the proposed analytical framework depends on the size of modelling time step (n). Three enhanced geometric distributions of travel time were proposed by Rakha and Farzaneh (2005) to overcome this issue. Farzaneh and Rakha (2006) also investigated the sensitivity of performance index (PI) with α and β, and found that PI is more sensitive to β than α and n. A formulation was proposed to control β by varying average travel time, to overcome the shortcomings of TRANSYT -7F which uses a fixed value of β as 0.80. In TRANSYT-7F, a signal timing plan which results an optimal value of PI is reported as design solution (Wallace et al. 1984). For delay based signal design, PI estimates the weighted combination of disutility measures such as stops, delay, fuel consumption which occurs at a traffic signal. Therefore, PI is usually minimized to obtain an optimal signal timing plan.

Discussion on Robertson’s model and its application approaches indicates that over the years, application approaches of Robertson‟s model has been updated gradually. Early research works successfully distinguished the influence of several roadway and traffic scenarios on model parameters by defining three levels for side friction e.g., low friction, moderate friction, and heavy friction. Subsequent investigations attempted to establish the functional relations among model parameters and major traffic state parameters such as vehicle demand, link travel time (representing a link length), roadway width, a share of transit mode in a vehicle mix, etc. These rationales are key to develop an efficient TOD plan using delay based signal design approach. Another important contribution was the development of a methodology for controlling β by altering the link travel time thus removing a significant bias in Transyt-7F signal design tool. The variation in behavior of a vehicle platoon was also investigated over various test sections. These test sections were selected considering a wide range of variation in link length usually found over urban road networks. In this context, a summary of roadway characteristics considered while investigating vehicle platoon dispersion by various researchers are presented in Table 4. Table 4 also includes the period of data collection while carrying out those investigations.

Table 4: Roadway Characteristics Considered for Data Collection to Investigate Vehicle Platoon Dispersion

Studies	Distance of test sections away from a stop-line	Link length (in meter)	Roadway configuration	Period of data collection
[1]	100m, 200m, and 300m,	640 m	6 lane suburban arterial	4 hours
[2]	85m	90m	4 lane dual carriageway	1½ hour
[3]	200m and 1000m	--	3 lane simulated roadway	--
[4]	300m and 550m 280m and 640m 230m and 520m 230m and 530m	--	3 lane dual carriageway 4 lane dual carriageway 5 lane dual carriageway 6 lane dual carriageway	4 hours
[5]	119m, 500m,1000m and 1350m	1500	4 lane dual carriageway	Morning peak hour, flat hour, low hour
	50m, 350m, 650m and 930m	1500	2 lane dual carriageway	Morning peak hour, flat hour

[1]: Denny 1989, [2]: Qiao et al. 2001, [3]: Farzaneh, Rkha 2005, [4]: Bie et al. 2013 [5]: Wang et al. 2003

Discussion:

Based on the review of past works on vehicle progression and Robertson‟s model following research gaps are identified. Experimental investigations on vehicle progression/vehicle platoon behavior in a non-lane based mixed traffic environment is limited. It is yet to be investigated how the non-lane based vehicle driving behavior in such traffic environment influence the vehicle progression. Limited investigations are found on the calibration of Robertson‟s model in a non-lane based mixed traffic environment. It is yet to be investigated how the key aspects of such a traffic environment influence the best-fit values for model parameters. Further research is also required to resolve the issue of overestimated „time span of downstream arrival‟ by Robertson‟s model.

3.2.1.2 Design approaches for over-saturated approach volumes

While delay and bandwidth based signal design approaches are found to be effective for under-saturated approach volumes, the very same design approaches were found to be ineffective for over-saturated approach volumes (Abu-Lebdeh and Benekohal 1997, 2003; Liu and Chang 2011; Song et al, 2012; Li et al.2013). Therefore, for over-saturated approach volumes, a powerful design approach is very much necessary to develop a TOD plan (Chang and Sun, 2004; Abu-Lebdeh and Benekohal, 2003). But, measures dealing with over-saturated state of approach volume have numerous choices and constraints which make them very complex (Abu-Lebdeh & Benekohal, 2003). In that sense, identification of over-saturated state of approach volume for a whole network or part of it is a critical exercise, prior to the development of a congestion mitigation measure or a TOD plan (Gettman et al. 2012). Wu et al. (2010) also emphasized that congestion mitigating measures are largely influenced by the inception of oversaturation and a level its severity. Therefore, determination of a state of over-situation and selection of a mitigation measure both are equally significant from the design point of view. These two aspects are briefly discussed in the following paragraphs.

3.2.1.2.1 Determining a state of over-saturation

Gazi (1964) characterized an intersection as over-saturated whenever 'the combined arrival rate in two or more competing approaches exceed the combined throughput so that queues would develop and maintained for a significant period of time.' Abu-Lebdeh & Benekohal (2003) identified the oversaturation as a traffic condition 'where a traffic queue persists from cycle to cycle either due to insufficient green splits or because of blockage.' For real-time signal design (such as SCATS: Sydney Coordinated Adaptive Traffic System, ACS-Lite: Adaptive Control Software Lite), the ratio of 'used green time to green phase duration' is treated as an indicator to detect saturation level for a particular approach (Sims and Dobinson 1980, Gettman et al., 2007). Wu et al. (2010) quantified the severity level of oversaturation (spatially as well as temporally) by introducing two parameters T-OSI (for residual queue) and S-OSI (for queue spillover). These two parameters assess the detrimental effects of oversaturation by estimating the ratio of unusable green time to total available green time in a cycle. For T-OSI, the 'unusable' green time is the equivalent green time to clear the residual queue in the following cycle. Whereas, for S-OSI, it is the time period during which the downstream link is blocked.

Gettman et al. (2012) gave an extensive insight into the conditions and scenarios which lead to oversaturation, and identified that both the factors viz. traffic demand exceeding the capacity and respective traffic control strategy are directly linked to oversaturation. To outline a spatial extent of oversaturation, they identified the traffic movements as a building block, and termed it as over-saturated, ;when the traffic demand for the movement exceeds the green-time capacity such that a queue that exists at the beginning of the green time is not fully dissipated at the end of the green time for that movement;. That specific traffic movement may also have the detrimental effects on other movements served by that approach to result in local starvation or blocking. Therefore, a specific approach may become over-saturated due to detrimental effect of oversaturation experienced by one or more traffic movements, served by that approach. Similarly, a signal phase would become over-saturated when all the movements, served by that phase are over-saturated. Whereas, at intersection level when two or more conflicting movements becomes over-saturated, the intersection will be termed as over-saturated. In addition to all these, length, growth rate, and duration of time over which the oversaturation sustain were also found critical to formulate the mitigation measures. Queue spill over or blocking towards an upstream movement is the next level indication of oversaturation which usually initiates along a route (containing two or more compatible movements along a travel path) and eventually may propagate to a network of arterial corridors. Simultaneous blocking of a number of compatible movements over a traffic network may lead to gridlock situation when ;traffic remain unable to proceed in any direction.; Apart from this spatial categorization, the occurrence of oversaturation may also be classified as an incidental event (residual queue lasts over a one or more consecutive cycles, and thereafter dissipate gradually), a recurrent event (due to frequent transition between under-saturated and over-saturated flow states), a persistent event (overflow condition survives over a significant portion of time duration for which a specific TOD plan are scheduled) or a prolonged event (overflow condition continues over a large time duration for which traffic patterns are supposed to get change). Over the years, several mitigation measures were developed and tested to resolve the various states of oversaturation along a corridor or a network.

3.2.1.2.2 Selection of a mitigation measure

Gazi (1964) developed an optimal control policy to reduce delay for a pair of closely spaced over-saturated intersections. The proposed control policy was further extended by Michalopoul and Stephanopolus (1977) to optimize a dual level objective function including queue constrain and delay minimization. Instead of delay per cycle, total delay during the entire study period was recommended to minimize. During the period of oversaturation, the developed algorithm also attempted to identify a proper switchover point to interchange the timing of the approaches. Chang and Lin (2000) developed a discrete type model, to upgrade Michalopoulos and Stephanopolos‟ (1977) „bang-bang‟ like control model. Chang and Sun (2004) further developed the model by conceptualizing three types of scenarios such as (a) Just-matching progression, (b) Advanced-matching progression, and (c) Delayed-matching progression, to classify the signals over a congested network. To maximize the throughput, the cycle length of every over-saturated intersection is made to be equal to the cycle length of the pivot controller, the most congested intersection at a certain cycle period. The control pivot may also be shifted to another intersection at new cycle steps. This indicates that cycle length is not necessarily being constant. Abu-Lebdeh & Benekohal (2003) proposed a dynamic control algorithm which was capable of providing a suitable queue management strategy for a given flow level. Four types of traffic management strategies were evaluated by assigning different levels of priority to manage arterial and side street queue. However, arterial and side streets were assumed to have only one-way traffic movement. To assess the merits of each queue management strategy and system performance at different flow level, five measure of effectiveness (MoE) parameters such as arterial traffic content (number of vehicles present in the system during a given cycle), average arterial speed (speed of all arterial traffic that was able to get in and out of the system), percentage of traffic held back (proportion of traffic that demanded entry into the system but was not allowed in), arterial throughput (product of arterial output rate in vehicle-miles/hour by the average speed) and finally a time-space diagram (a measure to assess degree of flow continuity associated with each management scheme) were introduced. Girianna and Benekohal (2002) developed a discrete-time signal coordination model for a two-way arterial network with over-saturated intersections. The objective function formulated the throughput as the number of released vehicle from intersection weighted by the ratio of distance travelled to the maximum link length within a network. The occurrence of queue at the end of green phase was taken care of by a disutility function, which was also the part of the objective function. The study concluded that under over-saturated states, the control scheme should clear the queue in the primary direction first (with large negative offset), and later queue in the secondary direction should be released. During the period of oversaturation, 'interaction between adjacent signals' is another critical factor which influences the system output (Abu-Lebdeh et al. 2007). To model the 'interaction between adjacent signals' Abu-Lebedh et al. (2007) proposed eight flow regimes based on three state variables viz. (i) wasted green time due to 'traffic starvation', (ii) blockage due to downstream traffic, and (iii) continuity of flow between neighbouring links. Liu and Chang (2011) proposed a set of enhanced macroscopic formulation to explicitly consider the dynamic interaction of queues among adjacent lanes and intersection during the period of spillback and blockage. Apart from maximizing the total throughput, the total time spent by the all vehicles within the control area was also minimized by the proposed function (Liu and Chang 2011).

Discussion:
Design of coordinated signal system during the period of oversaturation is a multidimensional issue. The spatial and temporal aspects of congestion, itself determine the spectrum of the problem. Due to the dynamic nature of the problem, for a fixed time traffic control system it is vary impractical to search for a specific timing plan which can serve the purpose. Rather, based on the trend of traffic flow pattern, it is more suitable to adopt a control strategy to identify when and where oversaturation is more likely to be a persistent or prolonged event (as discussed earlier). Otherwise, real-time traffic controllers are better equipped than fixed time controller. Adaptive control system like SCOOTS, SCATS, and RHODES are capable of handling the oversaturation condition by predicting the arrival profile within the system (Gettman et al. 2012).

3.3 Selection of Design Tool

Selection of design tool depends on the design objective, adopted for developing a TOD plan (Chaudhary et al. 2002). Corridor throughput is required to be prioritized when a traffic corridor caters heavy through movement. When a significant amount of turning movements are catered by a corridor, minimization of delay, stops or queue length along the coordinated links are required to be emphasized. Apart from the design objective(s), efficiency to model the vehicular flow at different states of approach volume (under-saturated/over-saturated) also influence the rational design of signal timing plan. Several signal design tools such as TRANSYT-7F, SYNCHRO, PASSAR II, and PASSAR V are found in practice for design of coordinated traffic signal system (Chaudhary et al. 2002, Sabra et al. 2000). Several features of the design tools are presented in Table 5.

Table 5 indicates that only TRANSYT -7F, PASSAR II and V are able to model the over-saturated approach volumes. While TRANSYT -7F adopts the delay based signal design approach, PASSAR II and V follow the criteria of maximizing the vehicle throughput. Therefore, TRANSYT -7F is found more suitable assuming that turning movements along the study corridor are considerable. Modeling of vehicle progression along a signalized links is also a key for the development of a TOD plan. TRANSYT-7F models the vehicle progression using Robertson‟s model which is a widely accepted, and a flexible model.

Table 5: Key Aspects of Signal Design Tools

Design Tools	TRANSYT-7F	SYCHRO	HCS	PASSAR II,V
Scale of analysis	Mesoscopic	Macroscopic		
Modelling of over-saturated flow	Yes	No		Yes
Design objective	Weighted combination of delay, stop and queue spillback	No. of stops and queue length	Volume to capacity ratio	Vehicle throughput
Consideration of vehicle dispersion	Yes	No	Yes	
Design output	Split, Phase, Cycle, Offset			
Consideration of double/ half cycle	Yes		No	

Apart from a weighted combination of delay, stops and queue spillback (Disutility index-DI), TRANSYT -7F has the flexibility of adopting other measures for development of timing plan (Chaudhary et al. 2002). Measures such as progression opportunity (PROS), throughput (TP), queuing ratio (QR) and an alternate combination of these parameters (PROS & DI, TP& DI, QR& DI) may be adopted as optimisation criteria in TRANSYT-7F. Therefore, TRANSYT-7F has the edge over other tools such as SYNCHRO, HCS for the design of coordinated traffic signal system. In present work, TRANSYT-7F is used for developing an initial signal timing plan.

Discussion:

Majority of the widely used signal design tools consider lane based vehicle driving behavior to analyze the traffic operation at signalized intersection. Such tools cannot be applied directly to obtain an efficient signal coordination plan for a given non-lane based mixed traffic environment. Therefore, there is a need for developing a methodology to carry out the design of traffic signal coordination in non-lane based mixed traffic environment. Utility of the microscopic traffic simulation models may be explored in future research investigations.

3.4 Development of Signal Timing Plan

Design of coordinated signal system results in Time of Day (TOD) signal timing plans. A TOD plan provides a number of signal timing parameters such as Coordinated Phase, Cycle Length, Splits, Force-offs, Yield point, offset etc. These signal control parameters are briefly described in the following paragraphs.

Coordinated Phase: For each traffic signal, a coordinated phase is designated to ensure the right of way for vehicles, departed from an upstream signal and destined to pass the downstream signals in the group. For corridor-level signal coordination, the signal phase which caters the through moving vehicles is termed as coordinated phase (Bonneson et al. 2009).

Cycle Length: Cycle length is the duration of time which caters to a complete sequence of signal indications. Usually, a common cycle length for all the traffic signals in the group is desirable for maintaining a consistent time-based relation among them. But, if it is found not suitable, some of the signals may have longer cycle length which is multiple (viz. 2 or 3 times) of the smallest one. Within a signal group, the signal which requires longest cycle length usually determines the system cycle length (Bonneson et al. 2009).

Splits: Splits are the portion of cycle length which is allocated to each phase. Coordinated splits are estimated based on phasing sequence and approach vehicle demand which requires the immediate green indication on arrival at stop-line. A split time includes green interval, yellow interval and red interval for clearance within a phase.

Force offs: Force offs are the time in a cycle when the uncoordinated phase ends and coordinated phase starts. Force offs cannot override the minimum split time or clearance interval.

Offsets: Offsets define the time relationship between the master clock and the coordinated phases of local traffic signals. The offset may be estimated as time lapse between initiation of coordinated phase at a local signal and initiation of the green interval at master signal (Bonneson et el. 2009, Gordon and Tighe 2005).

Yield point: It is a point in cycle time when a new TOD plan is adopted to cater a coordinate phase.

TOD plans are usually implemented through fixed time signal controllers. Variation in approach volume over different hours the day usually results into 2 to 6 TOD plans. On other hand, semi actuated or fully actuated signal controllers consider the short time variation in approach volumes. Cycle to cycle variation in approach volumes or variation over short period of time e.g. 5 to 15 minutes are duly considered by semi actuated or fully actuated signal controllers. Such signal controllers are functionally assisted by the vehicle detectors which are able to capture the short time fluctuation in approach volumes. Operation of an actuated signal controller is generally initiated with a base signal timing plan. If, the fluctuation in approach volumes as obtained from the detector is found to be significant cycle length and split intervals are adjusted. These intervals are usually extended or terminated according to high or low vehicle demand. A signal phase may be skipped if the detected approach volume is found insignificant (ITE, 1997). In the context of traffic signal coordination, actuated signal controllers are studied extensively.

When actuated signals are coordinated, they usually operate with a common cycle length to ensure vehicle progression along the major corridor (Zhang and Lou, 2013). Other signal timing parameters, e.g. split intervals and offset are obtained through direct optimization from the field operations (Skabardonis, 1996; Henry 2005). Offsets are first optimized using the fixed time method, and afterwards, offset values are updated according to the prevailing roadway and traffic condition (Zhang and Lou, 2013). Various investigations were carried out for coordinated actuated arterial signals. These investigations may be categorised in three groups (Zhang and Lou, 2013) viz. (a) evaluation of coordinated actuated signals (Luh and Lee, 1991; Skabardonis et al. 1998; Shafahi et al.1998, Lee and Messer, 2003; Lee et al. 2006; Stevanovic et al. 2009), (b) optimising the offsets for actuated signals with the given timing parameters (Jovanis and Gregor, 1986; Skabardonis, 1996; Chang, 1996; Shoup and Bullock 1999; Abbas et al. 2001; Zhang and Yin 2009), and (c) optimizing the offsets and signal timing parameters for individual signals (Stevanovic et al. 2007; Park and Lee 2009; He et al. 2012). Majority of these investigations were carried out for lane based traffic system, prevailing in developed countries.

3.5 Performance Evaluation

The basic principle of signal coordination is to ensure smooth progression of vehicles along a signalized corridor to reduce the level of performance measures such as travel time, vehicle stops and delay (Koonce et al. 2008). Therefore, TOD plans are developed to ensure drivers‟‟ right of way at d/s stop-lines who are released from u/s stop-line during coordinated splits. Driver‟‟s experience and their perception about a signal timing plan, and performance measures both are significant for determining the effectiveness of a coordinated traffic signal system (Koonce et al. 2008; Beak et al., 2017). Average delay for vehicles in platoon (Gartner et al., 1975), disutility due to delay and stop, queue length, throughput, weighted combinations of delay, queue length and throughput (Wallace et al. 1984), weighted combination of total vehicle delay and number of stops (Trafficware 2013, PTV 2014) are primarily the measures which indicate the performance of a coordinated traffic signal system. From drivers‟‟ point of view quality of progression are studied to assess the effectiveness of a coordinated traffic signal system. Several measures such as percentage of vehicle arriving on green (POG) (Day et al. 2010), vehicle arrival type represented by platoon ratio (ratio of the number of vehicles arriving during green to the g/C ratio) (Highway Capacity Manual 2000, 2010), bandwidth efficiency (ratio of the bandwidth to the cycle length) (Robertson 1986; Roess et al. 2011), progression opportunities (PROS), (number of successive green signals through which traffic flow can progress without any stops based on the desired speed) (Wallace et al. 1984), and smoothness of the flow of traffic (SOFT) (a measure indicating variation in vehicle speed while passing through multiple intersections) (Beak et al., 2017) are studied to measure the quality of vehicle progression. Evaluation of a coordinated traffic signal system may be carried out by field implementation of a TOD plan or by using micro-simulation model. In the context of non-lane mixed traffic environment, motorist's perception about moving across number of coordinated splits is yet to be investigated.

4. KEY AREAS OF FURTHER RESEARCH

As very limited investigation is found to appraise the suitability of traffic signal coordination in developing countries such as India, this section highlights the key research areas in this aspect. Research areas are identified under following aspects such as (i) vehicle progression, (ii) state of traffic flow, (iii) strategy to implement the signal coordination, (iv) modeling of vehicle progression, and (v) design of signal coordination. These are briefly discussed in following paragraphs.

(i) **Vehicle Progression:** Based on earlier discussion, quality of vehicle progression (QVP) is identified as the key aspect for coordinated operation of traffic signals. It affects the vehicle arrival profile at downstream signal, thus influencing the design of green split and off-set of a coordinated phase. The state of traffic changes over time and space, so as the factors influencing QVP. The major factors influencing QVP is the interaction of vehicles with roadway environment, which is commonly referred as side friction, and the interaction among the vehicles within a platoon. To investigate the effects of side friction on QVP, investigation should be carried out along a link over various traffic periods and compare the various levels of QVP. Further to compare QVP across the links for a particular corridor, similar traffic state should be considered. On the other hand, effect of mutual interaction of vehicles on QVP should be investigated under various levels of approach volume and composition. These are due to the fact that traffic periods represent the variation in traffic state descriptors viz. platoon size, vehicle mix etc., whereas link characteristics reflect the variation in carriageway configuration, layout of intersection, roadside features such as driveway density, on-street or off-street bus stops, presence and frequency of parking, pedestrian spillage over carriageway etc. As an example, presence of kerb-side bus stop may influence QVP, but the level of influence may not be similar for four-lane divided carriageway and six lane/ eight lane divided carriageway. Further, bus headway and dwelling time at a bus-stop also changes over different hours of the day. The effect of road abutting land use in conjunction with the factors contributing side friction should also be investigated comprehensively.

(ii) **State of traffic flow:** Effective implementation of traffic signal coordination also depends on the ratio of arterial volume to cross street volume. High traffic volume from cross street at signal impedes the QVP, however, in few cases with appropriate phasing side street turning movements may be used to help platooning of vehicles along the arterial approaches. Vehicle maneuvering from un-signalized side roads may also be controlled favoring QVP by allowing them to join the tail of a platoon. Similar strategies should be investigated under different corridor and traffic scenarios. The onset of „over saturated flow state‟ may also be influenced with the help of appropriate traffic control strategy. This in turn may help to understand the influence of flow state on arterial performance, and therefor to adopt the suitable strategies for coordinating the traffic signals.

(iii) **Strategy for implementing signal coordination:** Variation in directional split and link length also play critical role in strategizing the coordination of traffic signals. Therefore strategy should be developed with due consideration of one way/ two way coordination, coordination breakpoint, peak/off peak period coordination, double cycling, zero offset, speed limit etc. These strategies should also be investigated for "Time of day" operation for a given traffic corridor. While implementing a strategy for traffic signal coordination, regulatory measures such as speed limit, no stop/no parking, and control of un-signalized intersections should be investigated to improve the arterial performance.

(iv) **Modeling of vehicle progression:** Vehicle progression may be analyzed and modeled using microscopic measures such as „proportion of vehicle arrivals" for a given green split as an alternative to macroscopic measures such as vehicle dispersion as modeled by platoon dispersion models. Effect of vehicle composition on vehicle progression should also be investigated using microscopic analysis.

(v) **Design of signal coordination:** Use of state of the art micro-simulation tools such as VISSIM, CORSIM, AIMSUN etc. should also be investigated for design of signal coordination. This is due to the fact that the effectiveness of macroscopic signal design tools such as TRANSYT-7F, SYNCHRO etc. for non-lane based mixed traffic environment is yet to be established. In non-lane based traffic environment, vehicle to vehicle interaction is not only influenced by the longitudinal spacing but also by the lateral spacing. For example, under non lane based traffic operation, vehicles are positioned in rather dense and non-uniform manner at the stop line, resulting in a distinct vehicle discharge pattern which is different from the lane based traffic movement (Sharma et al. 2009). Such variation in vehicle discharge pattern may only be reflected by the microscopic analysis. Use of micro-simulation model may be extended to investigate whether non-lane based driving behavior is advantageous to vehicle progression or not for different flow state and vehicle composition.

5. CONCLUSION

The present work presented detailed discussions on two key aspects e.g. (a) the role of traffic signal coordination in a sustainable urban transportation system, (b) the need of policy measures for developing countries to adopt traffic signal coordination as a measure to alleviate the prevailing state of urban mobility. As the policy measures are to be well supported by the wisdom of research investigations, it is essential to carry out a detailed review of past research works on traffic signal coordination. Therefore, a detailed literature review is carried out followed by the discussions on research gaps in various facets of traffic signal coordination. The literature review highlights that coordinated operation of traffic signals is indeed a multi-dimensional complex entity. Mutual interaction of roadway, traffic and control conditions plays a key role in coordinated operation of traffic signals. It requires various engineering investigations e.g. (a) assessing the need of signal coordination, (b) defining the flow state, (c) selection of design approach (d) selection of design tool, (e) performance evaluation of the system to carry out the design of a coordinated traffic signal system. The past research works were primarily carried out in the context of lane based traffic system prevailing in the developed countries. The wisdom from past research works are case specific and are not spatially transferrable to the roadway system prevailing in developing countries. Therefore, in light of the roadway system in developing countries such as India, various key aspects are identified in the present work to carry out further investigations. Development of understanding on these key aspects is essential to draw the policy measures for adopting traffic signal coordination as a key measure of urban mobility.

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