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Investigation of freeway traffic control strategies' performance

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This paper focus on evaluation of impacts of traffic control strategies on traffic congestion relief. The traffic simulation network is modeled in a traffic microsimulation environment and the calibration with the real world data. As for traffic control strategies, fixed-time ramp metering, speed limit control and integrated control are tested and performances are compared with the actual condition along the stretch. The results indicate that there is an optimal cycle length for fixed time ramp metering and the best performance achieved at 15 s. The speed limits are tested for 60 to 100 km/h range with 10 km/h increments. Maximum performance is achieved at the speed of 70 km/h. The performance of integrated control is also examined. The results show that, 15 s cycle time fixed-time ramp metering along with 70 km/h speed limit control has the best performance overall. Furthermore, the results also suggest that the viability of integrated traffic control in metropolitan freeways is highly auspicious.

Key words: Traffic simulation, congestion management, ramp metering, speed limit control.

INTRODUCTION

Freeways had been commonly recognized as to provide virtually unlimited mobility to road users, without any flow disturbance (Papageorgiou et al., 2003). The constant increase of traffic demand, however, yields either recurrent congestion which occurs daily during rush hours or non-recurrent congestion which is defined as unexpected or unusual congestion (Hallenbeck et al., 2003). The congested freeways are one of the main reasons of extensive delays and contribute significantly to greenhouse gas (GHG) emissions within and around metropolitan areas. Congestion also has a direct affect on travel speed and brings out safety concerns (Kwon et al., 2006; Barth and Boriboonsomsin, 2008; Golob et al., 2004). The continuously increasing traffic congestion problem has led to application of various control strategies. Basically, these are formed by controlling the number of vehicles entering the freeway and/or by changing the speed limit of designated section along the freeway. Advanced urban traffic networks include both urban roads and freeways utilize control strategies like signal control, ramp metering, variable message signs and route guidance (Pesti et al., 2007).

In this paper, the traffic control strategies namely; fixed-time ramp metering, speed limit control and integrated control are implemented along a freeway corridor and performances are compared with the actual condition through traffic micro simulation software. The remainder of the paper is organized as follows: First the literature review of traffic control strategies and brief information on study area and data collection procedure are established respectively. This is followed by calibration of the traffic model, traffic control experiment methodology and results. Thereafter, real case traffic control simulation results are discussed and finally the conclusions are

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given.

TRAFFIC CONTROL STRATEGIES

Traditional control strategies use advanced technologies and more efficient procedures by integrating into the context of freeway management strategies that seek to manage, operate, and maintain expressways in an efficient and cost-effective manner (Jacobson et al., 2006). The most effective control measures that are typically employed in freeway networks can be classified as ramp metering, speed management and integrated control.

Ramp metering

The use of traffic signals on-ramps to control the merging on freeways is called ramp metering. Ramp meters are installed to control the rate of vehicles moving into the mainline traffic thus it prevents the critical volume of a freeway in order to control the demand and moreover, breaks the platoon of vehicles entering the freeway upstream of the signal to decrease the weaving phenomenon at the merge area. Ramp metering is projected to relieve or even eliminate congestion, ameliorate traffic flow conditions, safety and air quality, reduce total travel time and improve the performance measures, and regulate the demand in order to establish a stable freeway system (Zhang et al., 2001).

Ramp metering is a well-known technique for freeways. In fact, various techniques of ramp control were used in the late 1950s and through 1970s in Japan and USA. By the early 1990s, the technological advancement both in computing and measurement techniques make more sophisticated ramp metering systems possible to analyze and implement. The specification of the metering which is the specific entrance allowance for vehicles from ramp to the freeway rate draws an important role in control success. An extensive literature reviews found on ramp metering algorithms and comparison of the performances of some of these algorithms are demonstrated in (Zhang et al., 2001; Hasan, 1999; Scariza, 2003; Horowitz et al., 2006; Bogenberger and May, 1999).

Three different metering operations can be defined according to the control logic as: fixed-time, local traffic responsive and coordinated traffic responsive. A fixed-time ramp-metering control uses historical traffic data and a time-of-day basis (Papageorgiou and Kotsialos, 2000). Local traffic responsive ramp-metering strategies use the measurements of traffic flow and the metering rate is based on prevailing traffic conditions in the vicinity of the ramp. Most prominent examples of the local ramp-metering strategies are the demand capacity (DC), the occupancy (OCC) strategies and (ALINEA) strategy (Papageorgiou et al., 1998). Local traffic responsive metering algorithms regardless of type of controller which can be either linear (Papageorgiou et al., 1991), artificial neural network (Zhang and Ritchie, 1997) or Fuzzy-logic (Taylor et al., 1998) are performed well without considering the system-wide optimization. The coordinated traffic responsive ramp metering aims at optimization of the performance of the entire freeway facility. Fixed time and/or local traffic responsive control approaches could be used in concert with the coordinated traffic responsive control approach by predicting the traffic conditions. Coordinated ramp-metering strategies benefits the measurements from the entire network to control all metered ramps. Some studies (Taylor and Meldrum, 2000; Ahn et al., 2007) stated that coordinated traffic responsive strategies are more efficient when the demand is extremely high. Contrary to some studies (Chu et al., 2004; Ozbay et al., 2004), coordinated control algorithms are obtained not superior to the local ramp metering strategies. The main drawback of coordinated traffic responsive ramp metering approach is the complex and costly nature to realize.

Speed management

Variable Speed Limit (VSL) systems consist of variable message signs placed on gantries along the freeway and connected to traffic control center. The variable message signs, rather than traditional static signs, are used to display the regulatory or advisory speed limit, enabling freeway system controllers to dynamically intervene to the corresponding traffic conditions. In general, VSL control is implemented to homogenize traffic flow, improve safety, and reduce driver stress. Many VSL control strategies have been put into action in USA, UK, the Netherlands, Germany, Australia, Austria, Japan and Turkey (Mirshari et al., 2007). There are several recent studies investigating the impact of VSL on safety and traffic flow (Lee and Abdel-Aty, 2008; Abdel-Aty et al., 2008; Allaby et al., 2007). Much of the focus of VSL system evaluation studies has been on safety. German motorway data is utilized by Papageorgiou et al. (2008) to investigate the impact on aggregate traffic flow behavior considering the impact on the shape of the flow–occupancy diagram and efficiency increase. It is concluded that employment of speed limits below the critical occupancies decreases the slope of the flow–occupancy diagram and shifts the critical occupancy to higher values. However, with respect to the potential increase in efficiency, the results are indeterminate, as at some sections an increase have been attained, while at other sections no increase are noted. In conclusion, there appears to be limited empirical evidence and affirmative studies on the effects of VSL on traffic flow efficiency.

STUDY SITE AND DATA

The Bosporus Bridge in Istanbul, Turkey, is the first of the two highway crossings connecting Asia and Europe over the Bosporus

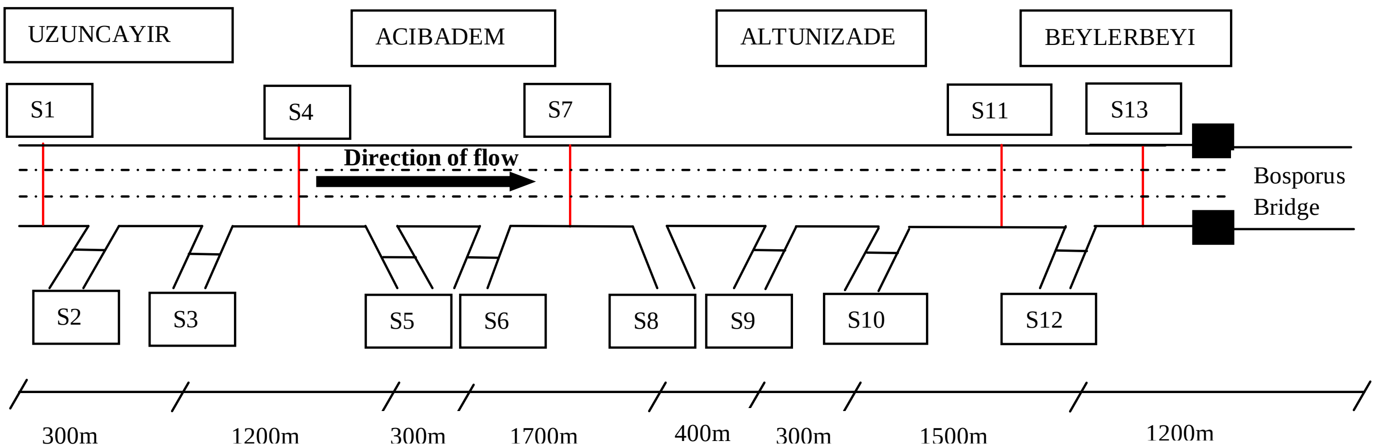

Figure 1. Study area – O1 Route Westbound Direction.

(a) (b)

Figure 2. O1 Route Westbound Direction: a) Simulated Network b) Aerial View (Google Earth®).

Strait. In this study, the traffic from Asia to Europe direction along O1 route, schematically shown in Figures 1 and 2, is selected as the study site. The corridor investigated has approximately 7 km of length, where there are 6 entrance ramps and 2 exit ramps up to the Bosporus Bridge. There are 4 main junctions entering / exiting to/from O1 Route and the bottlenecks are mostly occurring at around the downstream sections of the junctions (S4, S7, S11 and S13) due to the merging flow. In morning hours, especially weekdays the queue length may reach kilometers long and the average speed on the corridor can decrease down to 5 km/h which indicates a complete hyper-congestion.

The data is collected at the Istanbul Traffic Control Center at 14th of March, 2011. The traffic flow is observed from 6:00 a.m. to 11:30 a.m. through video recordings. Later, manual counts aggregated to 15 min and inserted to commercial spreadsheet program (Microsoft Excel). The volumes at ramps are high between 6:00 a.m. and 7:00 a.m.; especially most of the vehicles are medium type vehicles (minibus or midibus) which are used as service vehicles. Service vehicles could be classified as a special type of car sharing model which is mostly provided by companies free of charge to their employees. The hourly volumes of some ramps (S9, S10 and S12) even have higher volumes per lane than the mainline for a short time period.

The on-ramps at S2, S9 and S12 have 5 m width and designed as single lanes and S3, S6 and S10 have dual lanes with 3.5 m width. However, a virtual lane occurs at every single ramp during congested hours. The congestion starts at 6:45 a.m. and the flow decreases down to 400 veh./h/lane at the bottleneck downstream sections. It is observed that, once the breakdown occurs along the O1 Route, the congested flow remains invariant regardless of the time of the day which is verified by Sahin and Akyildiz (2005).

The speeds are also calculated through the recorded videos despite the measurement is not based on an approved method. The average speed is represented by randomly taken cars (medium, heavy) for a time period. The time is stored between two consecutive clicks passed between solid benchmarks which are generally the signboards or poles. The speed profiles are only used in visual conformity check and not considered for calibration purposes due to possible measurement errors.

However, the results indicate that the speed profiles at ramps are relatively lower than mainline speed. The average mainline speed decreases down to 20 km/h after 6:45 a.m. and oscillates between 30 to 40 km/h for automobiles afterwards. The average ramp speed is around 20 km/h for automobiles and after 9:00 a.m. the speed increases to 30 km/h.

SIMULATION MODELLING AND RESULTS

There are two ways to evaluate the performance of ramp metering systems: field operational tests and computer simulations. Although field tests provide more realistic results, due to the high costs and time consuming nature, traffic simulation studies are becoming more popular. In this study, widely accepted, discrete, stochastic, time step based microscopic traffic flow simulation software, VISSIM, is employed to test the performance of control strategies and compare their performances. VISSIM utilizes psychophysical car following models which combines a perceptual driver behavior model with a vehicle dynamic model by Wiedemann (1974, 1991) (PTV: VISSIM Version 5.40 User Manual, 2011).

The study corridor is simulated for the morning peak hours which start from 6:30 a.m. to 9:30 a.m. and performance measurement interval is selected as 15 min. The traffic composition and priorities at the ramp weaving areas are set through the analyses of video recordings. It is observed that the vehicles entering to the mainline are more aggressive than the vehicles cruising on the right most lanes. Therefore, priority is given to the ramp flows over mainline flows in simulation. It is also watched that, if there is enough gap the drivers tend to change the left most lane within the minimum possible distance. Typically, drivers are highly aggressive and breaking and acceleration values are taken higher than the default values. Lane changing is also highly strong in Istanbul traffic and drivers are frequently cutting in and overtaking. The car following model is selected as Wiedmann (1991), which has ten driver behavior parameters labeled CC0 – CC9. Several driver behavior parameters are reported to have significant impacts on roadway capacity and speed profiles thus, the parameters need to be optimized to attain the visual conformity and numerical correlation between the observation and simulation (Lownes and Machemehl, 2006).

Calibration

In the model calibration process, model parameters are altered until a qualitative and a quantitative balance between the simulation and the observation is reached. Traditionally, calibration requires several runs based on engineering judgment and experience. A three step calibration procedure is applied in this study, which are; calibration of driving behavior models, OD estimation and model fine-tuning.

The mean target headway and driver reaction time, which are the key user specified parameters in the car-following and lane changing models, can drastically influence overall driver behaviors of the simulation (Lownes and Machemehl, 2006). The calibrated values of the two parameters are 0.6 and 1.5 s, respectively in this study. The calibration of ten parameters in car following model could be performed through some optimization techniques in order to achieve the most representative model. However, this is not the focus of this paper. Likewise, the local arterial roads are not included in the studied network hence, route choice is not considered in this calibration process.

In this study, the observations of Chu and Yang (2003) confirmed that the precise geometry of merging angle and connector link length have an impact of simulation accuracy. Proportion of each vehicle type, vehicle characteristics and performance, such as the acceleration and deceleration rate, driving restrictions, speed limits and driving lane restriction, lookback distance at merging and bifurcation weaving area, priorities and traffic flow bases on conflicting areas also effects the simulation results.

The required number of runs can be calculated according to the mean and standard deviation of a performance measure of these runs, which is estimated from;

$$N = \left(t_{\alpha/2}\frac{\delta}{\mu\varepsilon}\right)^2 \qquad (1)$$

where μ and δ are the mean and standard deviation of the performance measure based on the already conducted simulation runs; e is the allowable error specified as a fraction of the mean μ; $t_{\alpha/2}$ is the critical value of the t-distribution at the confidence interval of 1-*a*. It is found that 10 different simulation runs are required. Therefore, the random seeds are chosen by creating 10 random numbers between 0 and 100 are listed in Table 1.

In calibration process, GEH index (Chu et al., 2011) is often used to test the relative difference between observed (Q_o) and simulated (Q_s) link volumes. GEH formula can be calculated with Equation (2) and the GEH values are tabulated in Table 2.

$$GEH = \sqrt{(2(Q_o - Q_s)^2)/(Q_o + Q_s)} \qquad (2)$$

The simulation model is acceptable if the GEH scores are smaller than 5 in 85% of the links and smaller than 4 for the sum of all link counts. The GEH scores are below 5 for all the links.

For OD estimation and fine tuning, the methodology given by Chu et al. (2004) is adapted. The major

Table 1. Random seeds used in simulation.

Number	1	2	3	4	5	6	7	8	9	10
Random seeds	17	27	34	40	48	58	62	73	88	93

Table 2. GEH values.

Sections	1	2	3	4	5	6	7	8	9	10	11	12	13
GEH	0.59	0.20	0.49	0.71	0.69	0.10	0.51	0.10	0.61	0.30	0.10	1.26	2.44

Table 3. Measured O-D Volumes (veh./h).

O-D / Time a.m.	1-5	1-8	1-13	2-5	2-8	2-13	3-5	3-8	3-13	6-8	6-13	9-13	10-13	12-13
6:30 - 6:44	505	197	756	339	238	637	524	195	840	334	1078	767	937	1526
6:45 - 6:59	449	145	713	390	240	618	605	195	628	404	1147	1004	820	1402
7:00 - 7:14	389	80	638	489	243	770	425	182	999	332	1503	745	800	1844
7:15 - 7:29	388	85	684	526	272	831	317	163	1135	427	1610	977	860	1952
7:30 - 7:44	455	68	665	645	266	880	270	124	915	422	1592	1301	876	1938
7:45 - 7:59	508	78	666	681	276	859	248	115	809	389	1348	1214	924	1579
8:00 - 8:14	512	72	514	678	264	790	290	121	867	425	1326	1324	916	1565
8:15 - 8:29	678	119	806	689	302	788	302	118	852	521	1502	1042	904	1849
8:30 - 8:44	605	290	720	672	392	682	272	205	774	554	1243	1074	1016	1464
8:45 - 8:59	890	278	814	742	302	589	461	208	945	712	1201	601	1004	1803
9:00 - 9:14	603	211	941	647	323	831	341	157	1117	815	1437	525	900	1945
9:15 - 9:29	708	329	778	787	466	954	390	181	872	453	1205	448	984	1616

difference from the Chu's study is that the reference OD matrix is also created with the help of traffic counts in this study. The volumes shown in Table 3 are inputted as 15 min exact volumes in order to represent the exact static routing decisions.

Methodology

Historically, the solution of the conflicts between the multidirectional flows of traffic is sought by considering the allocation of saturation, time or delay among all movements. The use of traffic signal establishes an orderly movement of traffic and increases the capacity and safety of intersections thoroughly. The design process of timing plans for signalized intersections in Highway Capacity Manual (Transportation Research Board, 2000) treats the traffic merely as static volumes of conflicting movements that require right-of-way alternatively. With a given phase sequence and phase groups, the method can determine how much green time within a cycle will be allocated to each phase, or the green splits. One fundamental difference of these methods is the design logic to allocate green splits; and these logic will affect how efficient and equitable a timing plan can be. Three major logics have been developed, viz; equal-saturation strategy (Webster, 1958) where the green time is determined in such a way that the phase duration will be proportional to its critical volume/capacity (V/C) ratio, delay minimization strategy (Allsop, 1971) which is a policy that minimizes the total intersection delay and the capacity maximization policy (Papageorgiou and Papamichail, 2007) maximizing the intersection capacity through balancing the traffic pressures of conflicting approaches where the pressure is defined as the product of the approach capacity and its average delay for each approach link.

Like traffic lights, ramp metering control utilizes traffic signals at freeway on ramps or freeway interchanges to manage the rate of vehicles entering the freeway. However, there is no established analytical method for the specification of optimum cycle time for fixed time ramp metering. Therefore, the traditional analytical models developed for fixed time intersection control are examined and capacity maximization approach is modified for fixed time ramp metering simulation experiment.

Ramp metering algorithms aim to set the allowable

Table 4. Green times with varying cycle times.

Section	S2	S3	S6	S9	S10	S12
Bottleneck downstream capacity (veh./h/lane)	1800	1800	1800	1800	1800	1800
Average excess demand (veh./h/lane)	182	234	527	680	368	580
Green ratio	0.90	0.87	0.71	0.62	0.80	0.68
Green times (5 s cycle time)	5	4	4	4	4	4
Green times (10 s cycle time)	9	8	7	7	8	7
Green times (15 s cycle time)	14	11	12	10	12	11
Green times (20 s cycle time)	18	15	14	13	16	14

ramp flow value r (in veh/h) which can be basically defined as (cr/3,600) vehicles where c denotes cycle time in seconds (Smith and van Vuren, 1993). Traffic lights are operated on the basis of a traffic cycle consisting of a green phase T_G, an amber phase T_A, a red phase T_R, and a red-amber phase T_{AR} which are adjusted in seconds such that:

$$c = T_G + T_A + T_R + T_{AR} \qquad (3)$$

In this study, the number of vehicles that the signals allow off the ramp is calculated as the difference between the actual demand at the bottleneck, more specifically the sum of the mainline and ramp flows) and the pre-specified capacity of the road. The most critical point is the specification of the bottleneck capacity since it varies over time. Nevertheless, in most cases bottlenecks are also considered as having the same capacity as basic freeway segments which takes values between 1800 and 2200 veh./h/lane. The excess demand (D_e) would be determined from:

$$D_e = D_a - C \qquad (4)$$

where D_a is the actual demand (in veh./h) including ramp and mainline flows and C (in veh./h) is the capacity of the downstream section of the bottleneck. Resulting from (4), the admissible ramp flow value (r) would be:

$$r = C - D_e \qquad (5)$$

The translation of the ramp flow value r into a corresponding green phase under a full traffic cycle plan, where the traffic cycle c is always equals to the metering period, would be based on the green ratio (f):

$$f = r/C \qquad (6)$$

Therefore, the green time leads to:

$$T_G = cf \qquad (7)$$

For example, at Beylerbeyi junction (S12), the hourly volumes of 3 lane mainline are 5020, 5076 and 4980, veh./h and the single lane ramp volumes are 953, 926 and 986 veh./h respectively. In order to maintain the capacity flow which is observed around 1800 veh./h/lane at the downstream of the section, the ramp volumes should be limited as the amount of the exceeding volumes. Hence, the excess volumes for each hour are calculated as 573, 602 and 566 veh./h respectively and the average excessive volume is determined as 580 veh./h consequently. Since the signal is placed to control only the ramp, the green time is calculated to restrict the flow only at ramp. The green ratio which is the ratio of effective green time to cycle length would be 0.67 and the corresponding green time for 10 s cycle time plan, which should be rounded-off to the closest integer value, would be 7 s. The signal timing plan for each section is shown in Table 4.

In order to determine the optimal cycle time and green time for fixed time ramp metering control and examine the cycle time duration effects on network performance, a set of simulation experiments is designed. At each ramp, the green times are calculated for the average flows of entire simulation period by varying the signal cycle time from 5 to 20 s.

With respect to the speed management, similar to the ramp metering control experiment a bunch of rationale speed limits from 60 to 100 km/h are employed with 10 km/h increments. Speed restriction areas are defined along the corridor in order to test the speed limit control in simulation environment. One of the important issues is compliance of drivers to the posited speed limits. However this is not the focus of this study and it is assumed that all the drivers are following the speeds with a 5% upper and lower margin. In order to achieve this in simulation environment the speed profiles are adjusted linearly for every speed limit examined.

In integrated control strategy in this study, the fixed time ramp metering control and speed management are implemented together without any coordination. As previously mentioned, the sophisticated control techniques are not considered feasible in short range therefore, the focus is given to the applicable control strategies. The results of ramp metering and speed management control are used to design the experimental setup for the integrated control. Ramp metering control is

Table 5. Performance measures of control strategies.

Control strategy \ Performance measures	Total travel time [h]	Total delay [h]	Number of stops	Average speed [km/h]
No Control	4942.8	2910.1	411772	29.2
RM_5 s	4875.0	2784.4	385791	29.6
RM_10 s	3598.1	1436.7	139553	41.5
RM_15 s	3368.5	1190.3	81634	44.7
RM_20 s	3595.2	1440.5	153010	41.4
SL_100 kph	5158.1	2732.0	445168	27.7
SL_90 kph	5077.0	2679.2	438311	28.2
SL_80 kph	4718.6	2656.6	381011	30.6
SL_70 kph	4125.7	2020.3	306180	35.4
SL_60 kph	4437.1	2115.1	336023	32.7
SL_50 kph	5505.1	2644.4	499931	25.7
Int_50 kph_15 s	4046.3	1026.1	152968	36.9
Int_60 kph_15 s	2957.9	406.5	51882	51.1
Int_70 kph_15 s	2586.6	386.6	45709	58.7
Int_80 kph_15 s	3331.5	1123.6	69012	45.3

found more effective than speed limit control. The 15 s cycle time has the best performance among other cycle times thus; the speed limits are tested from 50 to 80 km/h with 15 s ramp metering cycle time.

Performance measures

The objective of the freeway traffic control process is to optimize a performance index that mostly consists of efficiency measures. Performance index can be stated to minimize the travel times, delays, number of stops, or some other parameters such as fuel consumption and environmental pollution or in a more social context the optimization temporal and spatial of equity along the network or a more comprehensive objective that considers all the aspects with suitable weighting. However, only the efficiency properties are investigated for each control strategies in this study.

The first performance measure is selected as total travel time. The total travel time is calculated in hours for all active and arrived vehicles. In addition to the total travel time, the total delay in hours, the total number of stops and the average speed in km/h are evaluated by averaging values of 15 min intervals for each simulation run.

Table 5 compares the performance measure of control strategies investigated. It shows that all the traffic control strategies significantly increase the network performance. According to the results obtained, only 50 km/h speed limit control (SL_50 kph) performs worse than no control case in total travel time, number of stops and average speeds considering all the control scenarios. Nevertheless, even for the 50 km/h speed limit control, the total delay gets smaller values than no control case. The best network performance was achieved for ramp metering control at 15 s cycle time (RM_15 s). When the 15 s cycle time control is compared with no control case, it can be seen that the total travel time, the total delay and the number of stops decreased by 32, 60 and 80% respectively and the actual average speed increase from 29.2 to 44.7 km/h.

In case with the speed limit control, the best performance is attained at 70 km/h limit control where average speed increase from 29.2 to 35.4 km/h. and 19.8% decrease in total travel time. With speed limit control, the total travel time, the total delay and the number of stops are reduced by 17, 31 and 26% respectively a dramatic increase in all measures for integrated control. The average speed of the network increased to 58.7 km/h and 91.1%.

The results indicated that the total travel time can be significantly diminished by the examined control methods. Figure 3 shows the resulting total travel times for each control strategy and the comparison of best scenarios with no control case. From the results, for any scenario of ramp metering control the overall performances are increased. Nevertheless, for speed limits of 100, 90 and 50 km/h the performance of total travel times, the number of stops and the average speed measures are decreased. Interestingly, the total delays of these speed limits still have smaller values when compared with no control. This is possibly explained by the fact that speed harmonization may affect the total delays on the traffic network. When the best scenarios of all types of control are compared, it can be concluded that integrated control

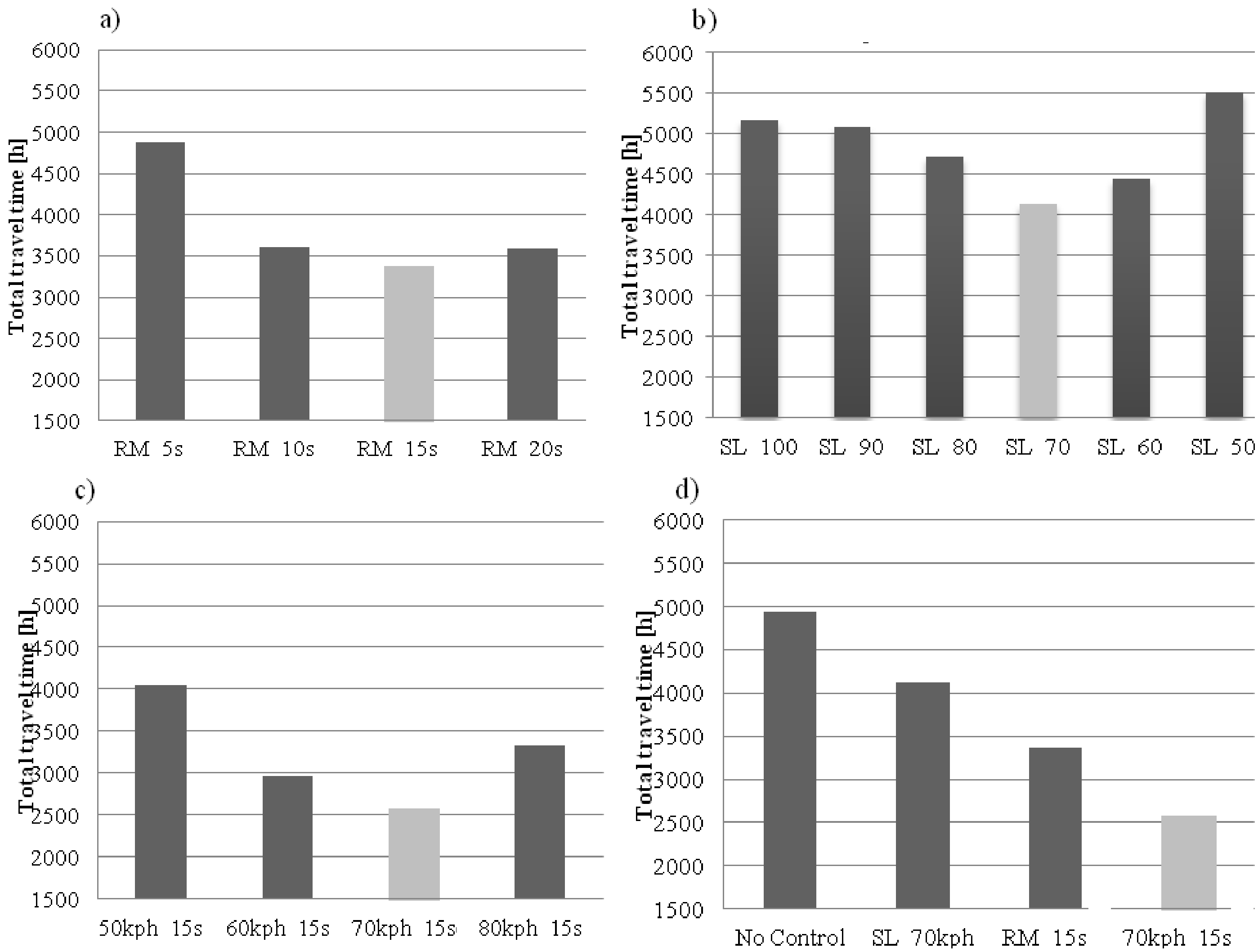

Figure 3. Simulation results for the total travel times: a) Ramp Metering Control b) Speed Limit Control c) Integrated Control d) Comparison of strategies with no control.

shows better performance than ramp metering and speed management in all measures analyzed. With the best scenario of integrated control, 70 km/h speed limit combined 15 s cycle time ramp metering control (Int_70 kph_15 s), the travel time reduces 48%, the total delay and number of stops decreases 87 and 89% and the average speed increases from 29.2 to 58.7 km/h when the integrated control is compared with no control case.

The traffic control strategies evaluated in this paper show that the freeway delays can be reduced through the increased capacity at bottlenecks which is remarked by previous studies (Newman et al., 1969; Cassidy and Rudjananakanoknad, 2002). It is clear that integrated control plays a positive role in increasing the average speeds. Under best scenario of integrated control, the average speed remains stable around 60 km/h for the entire simulation period. In addition, the capacity breakdown at bottleneck locations of Uzuncayir and Acibadem (sections 3 and 6) are prevented with integrated control. The integrated control increases the capacity of active freeway bottlenecks by either postponing or sometimes eliminating the bottleneck activation. This result is complied with Zhang's findings on ramp metering control (Zhang and Levinson, 2010). However due to excessive demand, the congestion cannot be fully avoided at Altunizade and Beylerbeyi (sections 9-10 and 12) on ramps even with the presence of integrated control but it is delayed compared to previous scenarios. Figure 4 plots the relationship between speed profiles of control strategies and the time of day (a) and sections (b).

DISCUSSION

This paper has presented the results of applying the fixed

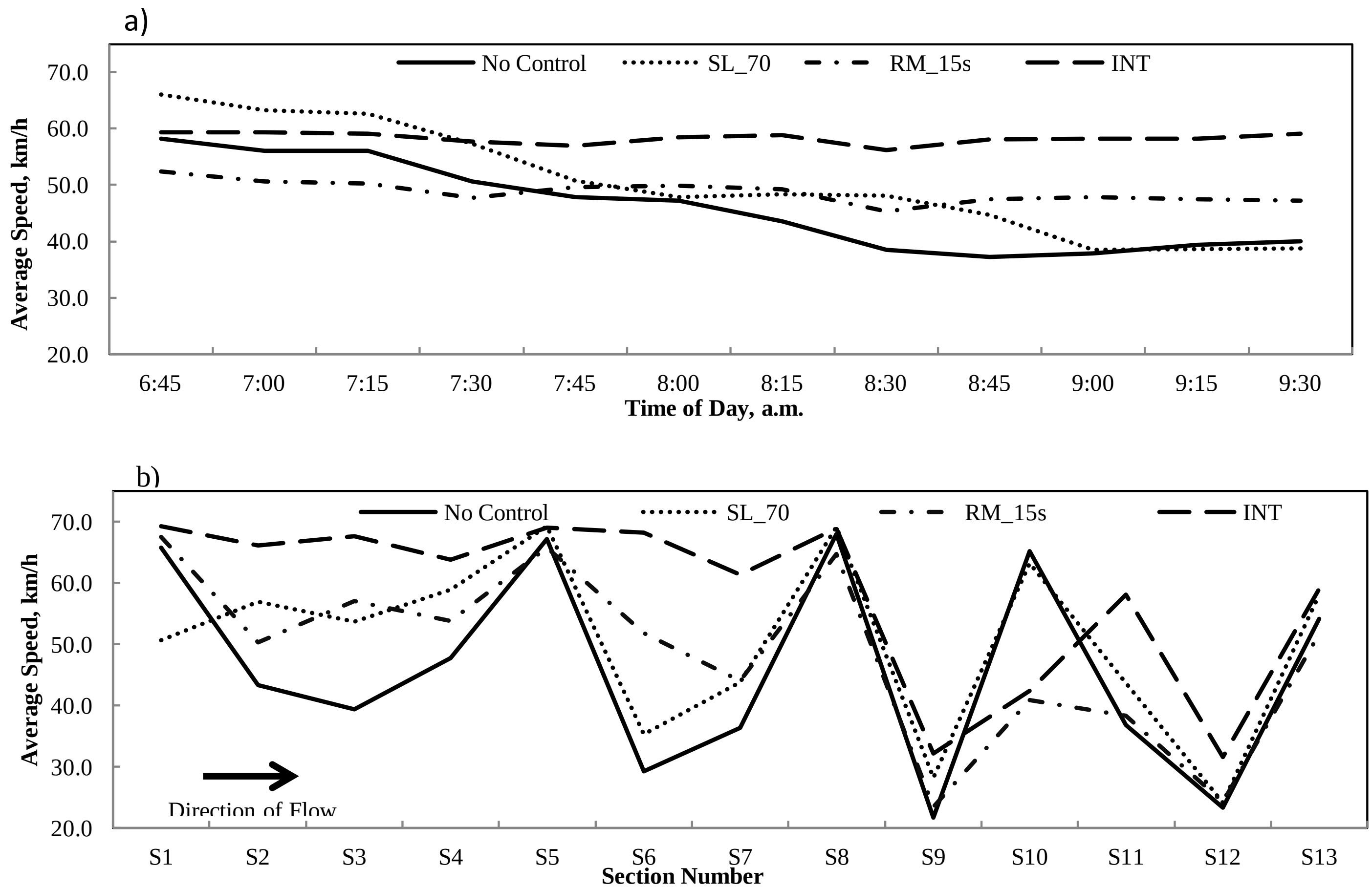

Figure 4. Average speed profiles of control strategies: a) Time of day b) Along the sections.

time ramp metering and speed limit control for the regulation of inflow from on ramps to the mainstream of a three lane freeway corridor. The prevailed results lead thus revealed the following:

1) There is an optimum cycle time that can be determined for each on ramp considering the bottleneck downstream capacity and flows. The short cycle lengths have a tendency to increase the start-up lost times and limits the merging rate, therefore the delay increases rapidly. Long cycle lengths allows platoon of vehicles entering the mainline which also contributes an increase in delay. The model shows that there is an optimum cycle length obtaining the best performance values for the merging section. The result of this study is highly congruent with previous findings on sensitivity of delay to cycle length for intersections exhibited on 16-16 at Highway Capacity Manual (Transportation Research Board, 2000).
2) Speed limit control strategy has also an optimal value which optimizes the performance measures. One of the possible reasons behind this is, at low speed limits the average speed drops under the effective level and at higher speeds drivers are applying more aggressive breaking which causes sudden shock waves.

3) Ramp metering and speed limit control integration significantly increase the performance even without any coordination and ramp metering depicts a relatively better performance increase in total travel time savings than speed limit control.

Probably the most beneficial output of this analysis is that all the control strategies analyzed increase the effectiveness of the traffic flow referring the total travel time, the total delay, the number of stop and average speed, in a satisfied manner. One of the reasons behind this extreme performance increase is not regarding the queue spillover effect. The ramp queues would be extended due to the control implementation in reality which yields a decrease in on ramp flow performance. Besides this, strict metering (high rate of ramp metering) brings equity concerns for ramp users and makes ramp metering a difficult policy to be accepted by society.

Conclusion

This study demonstrates the potential of implementing traffic control strategies in alleviating the traffic congestion on an urban freeway. Three traffic control strategies, namely fixed time ramp metering control, speed limit control and integrated control are analyzed in this study. The traffic simulation network is modeled in a traffic micro simulation environment and the traffic model is calibrated for the analyses. As indicated by the simulation results, there is an optimal cycle length for fixed time ramp metering and optimal speed limit for the speed limit control. The integrated control which is comprised of ramp metering control and speed management shows the best performance among all the control strategies investigated. It is not purported that the control methods described in this paper are the only strategies for traffic control. Future research is needed to compare the presented methods with the other approaches that have been recently developed, in order to understand the best features of each approach. Additional benefits of this course of research are the insights gained in traffic control and bottleneck formation typologies. Identifying the limits of traffic control strategies would assist in in efficiently directing these treatments.

On the other hand, the distribution of the performance increases is one of the issues that should be analyzed in detail. The efficiency measures are studied prior to the performance; however there are other measures such as equity and emissions that can be taken into consideration. Therefore, new studies are planned in this context including the equity properties of traffic control strategies and controlling the system to increase the efficiency, equity and to decrease the emissions simultaneously.

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