The Technical and Economic Viability of Automated Highway Systems: A Preliminary Analysis

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Abstract

Technical and economic investigations of automated highway systems (AHS) are addressed. It has generally been accepted that such systems show potential to alleviate urban tra±c congestion, so most of the AHS research has been focused instead on technical design and implementation issues. It is demonstrated that, despite making a number of assumptions that are favorable to AHS, the actual viable implementation opportunities for AHS are scarce, and that most existing congested urban areas can be disquali¯ed on the basis of at least one criterion developed herein. Technical investigations are described, including realistic estimates of AHS capacity, interfacing with the local street system, and storage issues. Discussion then turns to identifying criteria to help establish the types of urban areas that might be likely candidates for AHS technology. These criteria relate to the nature of the surrounding infrastructure and the tra±c demands placed on it, as well as the economic realities of AHS implementation. Certain \boutique" locations where AHS might be bene¯cial are identi¯ed, but it is uncertain whether enough bene¯t could be realized to make AHS palatable to the general public. AHS technology is not dismissed, but the simple analyses contained herein should warn that much more research into these areas is required before fully informed decision making about the future of AHS technologies can be accomplished.

INTRODUCTION

There seems to be a noticeable gap in the literature on automated highway systems (AHS) with regard to issues of feasibility and preliminary systems analysis. This paper will present a macroscopic framework within which some of the characteristics of AHS can be studied, including their capacities, sensitivity to congestion, and their interface with the local street system.

Not enough attention has been paid to real-life capacity-reducing events on an AHS, such as entrances and exits, lane changes, and other necessary maneuvers. Because empirically determined capacities of conventional freeways already incorporate these phenomena, a goal of this paper is to quantify the resulting e®ects for an AHS, so that comparisons can be made on a \level playing ¯eld".

These e®ects on capacity, while signi¯cant, may not represent the true limitations of the system, however. What often controls tra±c °ow on many facilities is the ability of destinations to accommodate tra±c. No improvements to speed, °ow, safety, or other conditions on the freeway can increase the input °ow of tra±c to the city beyond the absorption capability of the terminus.

It is unreasonable to suppose that it will always be possible to build temporary storage bu®ers for whatever queues may accumulate as a result of oversaturated exits. Thus, a thorough study of the capacity of automated highway systems must incorporate parameters regarding the capacity of the destination nodes. If the system was designed improperly, queues could grow very quickly on the automated lanes, which lack the storage capacity of a conventional freeway with the same capacity.

There may be certain freeways which are better suited for automation than others. One example is a ring freeway which encloses an urban area, due to the fact that there is no freeway terminus. Although the o®ramp capacity still needs to be considered, these types of freeways typically serve areas of cities outside of the most congested inner area; hence the local street system and off-ramps typically have a greater capability of absorbing the incoming traffic. If the provision of capacity is the only real issue, then the choice of what to build (conventional vs. AHS) boils down to a question of economic feasibility.

This paper addresses all of these questions in more detail, in two main sections. The next section investigates capacity, entrance and exit design, storage capacity, and shockwave propagation. The following section identifies locations where the capacity of an AHS can be fully used and then compares the economic costs of an AHS vs. a conventional freeway in such locations. Throughout this paper, we will strive to make any required assumptions in a manner that is favorable to AHS, because it is our goal to narrow down the field of application contexts where an AHS may be successfully deployed by eliminating obviously inappropriate scenarios.

TECHNICAL INVESTIGATIONS OF AHS PERFORMANCE

GENERAL CONCEPTS

The precise system design for AHS has not yet been finalized. There are scenarios under investigation which employ platooning, and scenarios which do not. Likewise, there are alternatives which favor offboard control, and some which favor autonomous operation. The methodology proposed in this paper is intended to be applicable to all of the above. In the interest of demonstration, we will include an example which assumes that platooning will be used, and makes a number of other assumptions which will be clarified later. For now, we seek to introduce a general idea that holds in all cases.

We are interested in the sustainable flow past an entrance and exit ramp to an AHS. We do not at this point make any assumptions about the geometric design of the entrance and exit facilities, nor is it relevant whether or not the vehicles are platooned. Assuming the minimum sustainable average time headway on the AHS is h, we seek the maximum possible flow, Q, that can approach the pair of ramps and be stable. Certainly, Q must satisfy $Q < 1/h$, but there may be other requirements due to the entering and exiting flows.

Let μ denote the fraction of this stream that intends to exit at the off-ramp, and β the ratio of the entering flow to approaching flow. Thus, the flow downstream of the "diamond interchange" should be $Q(1 - \mu + \beta)$. If the design of the ramps requires that exiting and entering vehicles have additional average time headways of e and m, respectively, then stable flow can be achieved only if the approaching stream contains enough "holes" to accommodate the extra headways; i.e. if $Q = 1/\bar{h}$, where

$$\bar{h} = h + \max\{\mu e; \beta m\} \tag{1}$$

If the traffic pattern at the interchange is symmetric, then the smallest possible mean headway becomes:

$$\bar{h} = h + \mu \max\{e; m\} \tag{2}$$

The precise values of e and m depend on the system protocol, and perhaps on the geometry of the facilities used for exiting and entering vehicles, but the equations are general.

The following subsections deal with a particular scenario, one that is extensively studied in the AHS literature. This scenario assumes that the traffic stream will incorporate a platoon structure. Other scenarios can be evaluated by inserting in (1) or (2) proper values for e and m.

PLATOONING SCENARIO

Tra±c °ow on an AHS could be organized into platoons of closely spaced vehicles with large interplatoon spacing, shown in Figure 1. The variables that determine the capacity of the automated lane are the intraplatoon distance, L_b, the interplatoon distance, L_p, the vehicle length, L_v, the vehicle speed, v, and the platoon size, N.

The minimum spacing between lead vehicles in consecutive platoons is then given by:

$$S_{min} = NL_v + (N - 1)L_b + L_p; \tag{3}$$

in meters. Because there are N vehicles within this spacing, the density of tra±c is N/S_{min} veh/m. If the velocity of the platoons is v m/s, the hypothetical \capacity" of one uninterrupted automated lane, in vehicles per hour, is then:

$$C = \frac{3600vN}{NL_v + (N - 1)L_b + L_p} \tag{4}$$

This capacity formula has been used to support automated highways[1]. Very high capacity values are predicted, even with platoons of moderate length. It has also been claimed that platooning favors safety[2].

Reasonable values for some of the parameters might be $L_p = 50$m, $L_b = 1$m, and $L_v = 5$m. In this case, then °ows on the order of 7,000 veh/hr can be achieved with realistic speeds ($v \approx 30$ m/s) for platoons of size $N = 5$, and up to 12,000 veh/hr when a platoon size of $N = 15$ is used. This is only representative of an idealized scenario without lane-changes, entrances, or exits. The next three subsections look at the e®ect of some of these maneuvers.

EFFECT OF THE EXIT MANEUVER ON CAPACITY

This section assumes that the exits from the automated lanes consist of a given number of gates connecting the right-hand AHS lane to a parallel transition lane, or to dedicated exit ramps. Further, it is assumed that a safe execution of the exit manuever does not allow different vehicles from the same platoon to exit through the same gates[1]; hence, a platoon should be split upstream of the exit gates if it contains more exiting vehicles than there are gates. It is not clear at this point whether vehicles exiting from the AHS will transfer directly onto local streets, or if they should pass through some hybrid automatic/manual facility first.

The platoon division is executed by a split manuever, which decelerates the leader of the new platoon until it is a safe distance from the preceding platoon. This process may need to be repeated, until none of the new sub-platoons contains more exiting vehicles than there are exit gates available. Thus, the additional interplatoon distance required by the exit manuever will depend on the number of exiting vehicles and the number of exit gates.

Figure 2 shows the time-space trajectories of several platoons as they pass the exit gates. The first platoon did not have to be split because the number of exiting vehicles did not exceed the number of gates. The second platoon was split and the resulting trailing platoon decelerated until a new gap was created of length equal to L_p. While the figure depicts vehicles exiting from the rear of the sub-platoons, this need not be the case, and the following analysis is valid for any distribution of exiting vehicles within the sub-platoons.

A direct consequence of the splits is the capacity reduction caused by the necessity of accommodating the extra required distance, L_x. L_x is a random variable, because the number of exiting vehicles varies across platoons. Clearly, L_x will depend on the composition of the platoon; that is, on the destinations of its vehicles. If a vehicle's destination is not considered when granting or denying it access to any particular platoon, then the number of exiting vehicles in a platoon will follow a binomial distribution. For each platoon the actual value of L_x is some integer multiple of L_p; therefore the following holds:

$$E[L_x] = \beta_G L_p, \tag{5}$$

where β_G is a real number representing the average number of extra spacings required per platoon when the number of available gates is G. Thus, (4) needs to be redefined:

$$C = \frac{3600VN}{NL_v + (N - 1)L_b + L_p + \beta_G L_p} \tag{6}$$

Previous research by these authors[3] has shown that when one exit gate is available,

$$\beta_1 = N\gamma - 1 + (1 - \gamma)^N, \tag{7}$$

where γ is the fraction of the upstream freeway flow desiring to use the exit in question. Similarly, if there are two gates, it was shown that

$$\beta_2 = \frac{N\gamma}{2} - \frac{3}{4} + (1 - \gamma)^N - \frac{1}{4}(1 - 2\gamma)^N \tag{8}$$

Figure 3 depicts the revised capacity as a function of γ, for different values of the platoon size and assuming we have two exit gates. (Ignore for now the heavy line on the figure; this will be discussed in the next section.) Thus, the capacity has been calculated by substituting β_2 in (8) for β_G in (6). The figure shows that very high values of the capacity can be obtained even for moderately high values of γ. However, in the interval $0.1 \leq \gamma \leq 0.4$, the capacity decreases significantly with increasing γ. It will be shown later, however, that locations where automation could be beneficial would likely exhibit a low exit flow ratio.

SCHEDULING VEHICLES INTO PLATOONS

One may argue that capacity flows may be increased by "scheduling" entering vehicles into passing platoons so as to ensure that platoons passing the most congested exit ramp always carry a number of exiting vehicles that is an integer multiple of the number of exit gates, G. This would have the effect of minimizing the total number of upstream splits that must occur, thereby minimizing the average headway between platoons, and thus maximizing the capacity. Although the feasibility and possible side-effects (e.g. extra on-ramp delays) of such an operation have not been explored, we examine below the improvements to capacity that can be achieved with scheduling and interpret the results as an upper bound.

For a given set of conditions (μ, N, G), there exists a steady-state average number of exiting vehicles per platoon, $\overline{E_v} = \mu N$, and a long-run average number of exiting "batches" per platoon, $\overline{E_b} = \mu N/G$. Ignoring the trivial case where $\overline{E_b}$ is an integer, the traffic stream should be split into two types of platoons: those carrying $\left\lceil \overline{E_b} \right\rceil^{+}$ exiting batches, and those carrying $\left\lceil \overline{E_b} \right\rceil^{-}$ exiting batches. Here the notation $\lceil\cdot\rceil^{+}$ and $\lceil\cdot\rceil^{-}$ denotes rounding up and down to the next integer, respectively. As explained in del Castillo et al[3], this has the benefit of minimizing the total number of platoon splits.

If the average number of exiting batches is less than one, no splits should ever be required, because the two types of platoons employed would be those containing no exiting vehicles, and those containing exactly the same number of exiting vehicles as available gates, for which no splits would be required. On the other hand, if the average number of exiting batches is greater than one, then the average number of splits per platoon is one less than the average number of exiting batches. This implies the following result:

$$\beta_G = \max\left\{0, \frac{N\mu}{G} - 1\right\}, \tag{9}$$

which is derived more formally in del Castillo et al[3]. Thus, an upper bound for the capacity is obtained by substituting this expression for the parameter β_G of (6). The heavy line in Figure 3 depicts this upper bound for the case $G = 2$ and $N = 10$. In this particular case, the upper bound offers a noticeable improvement in the range $0.1 \lesssim \mu \lesssim 0.4$.

THE EFFECT OF ENTERING VEHICLES

The effect of the entering flows is equally significant. This observation, corroborated by simulation experiments[4,5], can be understood easily if we recognize that the platoon separation must be greater than that which would ensure safe operation away from the ramps, L_p. Figure 4 displays the time-space trajectories for the gap between two platoons into which a new platoon of vehicles is to merge. We seek the gap width, L_p', which will ensure that the merging vehicles are never closer than a distance L_p from the platoon they are not joining. (A similar figure could be constructed for a system designed to merge vehicles from the front.)

We can ignore the physical dimensions of the merging pre-platoon if it includes the same number of vehicles as those which had left the target platoon at the previous exit ramp. This is reasonable because the traffic flow is then restored to its level upstream of the "diamond interchange".

A vehicle merging at speed $v_e \leq v$, with a margin of safety (head to tail) of Δ meters will fall behind the trailing end of the lead platoon by a maximum distance $L_m = (v - v_e)^2/2a + \Delta$ if the vehicle accelerates uniformly at a m/s^2. To maintain a gap of size L_p between the merging vehicles and the trailing platoon (see Figure 4), the interplatoon distance upstream of the "diamond interchange" should be at least $L_p + L_m$.

The design acceleration a and the design speed difference $(v - v_e)$ should be those which would apply to the most underpowered vehicle allowed to use the system, and not to the average. If we take $\Delta = 5$ m, $(v - v_e) = 5$ m/s, and $a = 1$ m/s^2, we find that $L_m \sim 17$ m; this distance drops to $L_m \sim 7$ m for $(v - v_e) \sim 2$ m/s. Of course, a definite choice for these parameters (Δ, a, and $v - v_e$) cannot be made until a better experimental understanding of the merging maneuver has been developed.

Protocols have been proposed where the merging maneuver would take place as a simple lane-change with $v_e \simeq v$ and $\Delta \simeq 0$; i.e. with no significant extra headway requirement. This assumes that the AHS system will always be able to provide sufficient acceleration distance, even for the most underpowered vehicles, and that there is no need to provide a safety margin when vehicles are moving laterally with respect to one another. The former assumption is violated on most existing freeways, and because AHS promises to allow higher speeds (and attempts will be made to install it on conventional freeways, to avoid the cost of additional rights-of-way) the assumption $v_e \simeq v$ is likely to be violated in an AHS as well. The degree to $(\Delta \simeq 0)$ can be achieved depends on the quality of detection of relative locations and speeds between vehicles; only physical experiments can determine this properly.

It seems reasonable to assume that the interplatoon distance would have to be increased by somewhere between $L_m = 10$ m and $L_m = 20$ m in order to accommodate merges, which would have a considerable effect on capacity. The effect can be quantified by application of (6). For example, for $L_m = 20$ m, a 20% reduction in capacity results for the data of Figure 3 if $N = 10$ and $\mu \lesssim 0.2$. The fractional reductions are larger for smaller N and larger μ and can approach 30%.

Despite these downward corrections it would appear from Figure 3 (bearing in mind the effect of entering vehicles discussed here but not reflected in the figure) that an AHS system can still pump traffic on a single lane past an interchange at rates upwards of 6000 veh/hr if one can keep ^{1} below 0.2 (for $G = 2$). This is better than two lanes of conventional freeway, although not as high as initially thought.

INTERFACE WITH CONVENTIONAL STREET SYSTEM

For some scenarios, the exit flow that would have to be accommodated to avoid queues, given the mainline flows derived above, would also have to be very high; e.g. on the order of 1800 veh/hr if the exit flow ratio is 0.3. However, at a given point of the exit ramp, the control of the vehicle should be transfered to the driver, meaning that the capacity of the exit ramp from the automated lane will be the same as from a manual lane. Therefore, a capacity restriction for the AHS may also arise beyond the exit gates from the ultimate necessity of manually driving the vehicles. The exits should be designed so as to eliminate this potential bottleneck.

An exit flow of 1800 veh/hr is about two to three times the typical capacity of an exit ramp from a manual lane. The only way to achieve such a high exit capacity is by splitting the exit flow from the automated lane into several streams and feeding these into different streets or highways, as shown in Figure 5 (a similar mechanism must exist for transferring entering vehicles from the local street system onto the AHS). The additional construction cost of the exit ramps may become an important part of the total cost of the AHS, as will be recognized in the economic evaluation described later. In any case, the necessity of increasing the exit capacity of the automated lane must be borne in mind when designing the AHS, and one still has to ensure that the local street system can absorb the flow.

CONGESTION EFFECTS

Very little has been said so far about the ability of an AHS to withstand congestion effects. We include in this section a discussion of stationary queue storage and shock waves. We do not study the causes of congestion but rather its consequences for the traffic flow.

The storage capacity of a highway is conditioned by its jam density, given by:

$$K_j = \frac{N}{L_p(0) + N L_v + (N - 1) L_b}; \tag{10}$$

where $L_p(0)$ is the interplatoon distance at zero speed, whose value might be 10 m. For the usual values of the parameters (L_b = 1 m and L_v = 5 m) the jam density ranges from 128 veh/km for N = 5 to 152 veh/km for N = 15; we assume an average value of 140 veh/km. This is of the same order as the jam density for a conventional freeway lane. In principle, it seems that the conversion of some of the lanes in a conventional freeway to AHS useage would not make any difference in the ability of the facility to store queued vehicles. However, this is incorrect for two reasons.

The first reason is the possible need of converting one or more of the original lanes into a transition lane, or into space necessary to accommodate dedicated entry and exit ramps. Although some of the proposals for automated freeways do not require a continuous transition lane, we believe that for urban freeways with closely spaced off- and on-ramps, the transition sections will occupy a significant length of the freeway, leading to the almost complete loss of one of the original lanes for throughput purposes. The storage losses are more severe when one compares a stand-alone AHS facility with a conventional freeway of the same capacity. Obviously, the storage is cut by a factor of 3 in this case.

The second reason is the increase in capacity achieved by the AHS. If for some reason the much higher expected tra±c °ow collapses, the number of vehicles to store per unit time will be much greater than with a conventional lane. If two lanes of a four-lane conventional freeway are converted to AHS (one automated lane and one transition lane or set of ramps), and the capacity of the automated lane is four times that of a conventional lane, then the freeway as a whole has 50% greater capacity, but queues on the automated lane will grow four times as quickly as on conventional lanes.

If the capacity of the automated lane is approximately 9000 veh/hr, the density at capacity 80 veh/km, and the jam density 140 veh/km, then the velocity of queue growth, or shockwave speed, should be approximately -40 m/s, slightly higher than the prevailing tra±c speed[6]. Similar velocities will occur for transitions between any two queued states. Although the passage of such a fast shockwave should pose no problem for a properly functioning AHS (it requires a reaction time of approximately one second from one platoon to the next) the same cannot be said if some fault condition has arisen.

Another important point is that it would take as little as three minutes for a queue to travel the 5 km distance between adjoining diamond interchanges, and that (in order to avoid gridlock e®ects; see Daganzo[7] for a discussion) it may be necessary to close many of the on-ramps upstream of the congested bottleneck, transferring much of the vehicle storage to the local street system. This illustrates the severe systemwide consequences of local disturbances in an AHS.

It is possible that some advanced tra±c management strategy could be employed to alleviate some of these e®ects. Certainly, with the information resources available with an AHS system in place, this could be accomplished much more e®ectively than existing means allow. In most cases, however, this would require the storage of these vehicles on the local streets, as they attempted to travel to an alternative entry point to bypass the congestion. While analysis of this type of phenomenon is beyond the scope of this paper, it should be clear that the impacts of such a strategy will be worst felt on the local streets, and cannot be assumed to be negligible.

POSSIBLE SCENARIOS FOR AHS

In this section, we examine an AHS from a more macroscopic point of view, and address two main issues. The first task is to determine the ability of a particular urban area to accommodate the substantial entry and exit flows required for an AHS to operate efficiently. The second issue is the question of whether or not an AHS is a viable economic alternative to conventional freeways. These two issues can be resolved by examining a set of criteria which help determine whether a particular location is a feasible candidate for an AHS or not.

ABSORPTION ONTO LOCAL STREETS

We now consider a (sub)urban AHS lane, theoretically able to carry the equivalent of three freeway lanes, which distributes traffic to the local streets (e.g. during the morning commute near a CBD). Clearly, such a facility cannot be expected to achieve what a three-lane freeway is unable to do. In particular, we show that neither a freeway nor an AHS can alleviate congestion inside urban areas.

A freeway (or an AHS lane), located in a region that is able to absorb f vehicles per hour per linear kilometer of freeway, will be able to carry a flow of Q veh/hr if $Q < fl$, where l is the average length of a trip in kilometers. This quantity is an important parameter that bounds the capacity, since there is no incentive to build a facility that will carry flows greater than fl. For example, a 10 km beltway for which l might be 2.5 km, ringing a CBD with $f \approx 1000$ veh/hr-km, cannot serve more than 2500 veh/hr. Flows on the order of 10,000 veh/hr may be possible on a longer (e.g. 40 km) ring road with longer average trip lengths (e.g. l ¼ 10 km) but this would require o®-ramps able to serve 1000 veh/hr to be placed approximately every kilometer.

Previous research[3] has shown that AHS o®-ramps might be spaced at least d = 5 km apart. As a result, they would have to carry substantial °ows. Since the formula for the exit ramp °ow is $Qd{=}l$ (in an AHS-friendly symmetric scenario where all ramps are considered identical), these exit °ows will have to be handled as in Figure 5 if $Qd{=}l$ is greater than what a single street can absorb. In this case, they would have to be spread over a length $d^0 = (Qd{=}l){=}f$. We say that symmetry is AHS-friendly because the capacity of a conventional freeway can be adjusted gradually by adding and dropping lanes to conform closely with asymmetric °ows. The quantity $Q{=}(fl)$ represents the proportion of local streets that must be reached by ramps, and given the geometry of Figure 5, the ratio $d^0{=}d$ ¼ $Q{=}(fl)$ gives the proportion of the AHS lane that must be overlapped by a service lane (or other facilities) to handle exits. A similar proportion would be required to handle entries.

If transition lanes are employed, then in order to save construction costs comparable to the cost of an extra auxiliary lane, we must require:

$$2Q{=}(fl) << 1{:} \tag{11}$$

An AHS lane with Q = 6000 veh/hr would require $fl >> 12000$ veh/hr. For an average trip length of 10 km this would require $f >> 1200$ veh/hr-km; e.g. f ¼ $10{;}000$ veh/hr-km, which seems large, even in suburban areas. (A two-lane o®-ramp ending in a tra±c signal may carry 1500 veh/hr onto an arterial street; thus f ¼ $10{;}000$ veh/hr-km means that there would have to be six or seven such streets per kilometer.) Any other exiting facility, such as dedicated ramps, will likely have a cost on the same order of magnitude as a transition lane, and the conclusion in this case would likely not be much different than above.

In view of these facts it seems that the best places for AHS are regions where l is large compared with 10 km; i.e. for interurban trips. This suggests that AHS freeways cannot be a solution to the congestion problems in cities. The idea is further reinforced by the previous results on storage capacity, which show that if an AHS lane is placed in a congested urban location as a substitute for a traditional multi-lane freeway, the AHS is affected by congestion in a much less satisfactory way. In the next section, we present a simple but generic economic comparison of AHS and conventional technologies when congestion is not a problem.

ECONOMIC COMPARISON OF A SINGLE-LANE AHS AND A THREE-LANE FREEWAY

If the primary purpose of an AHS was to increase the capacity of a corridor whose local streets were capable of absorbing the traffic, the same objective could be met by building a larger conventional freeway, in which case, a comparison of the two technologies must boil down to an economic evaluation. In this section, this comparison is carried out for the case where a single-lane AHS is considered a substitute for a three-lane conventional freeway, although the formulae can easily be extended to more general cases. This example will show that AHS is viable in an economic sense only where reasonably long trip lengths are expected (as compared to the spacing between ramps), and that some control over this phenomenon can be gained by judiciously choosing which vehicles are allowed entry to the AHS.

Suppose that within some corridor where an AHS is feasible, the average trip length of highway users is l. Suppose also that the design spacing, d, is chosen to be equivalent for both the AHS and the conventional freeway, so as to control for the level of service being o®ered by the two competing technologies. This assumption is favorable to AHS; with conventional freeways, the option always exists to build more miles of smaller facilities (e.g. 2-lane freeways) and distribute these more evenly in space, increasing the system's accessibility. We will also assume that $d << l$, to ensure that the amount of backtracking needed by most people to get on the freeway is small. This allows us to assume that both l and the freeway entry °ows per unit length are independent of the design variable, d. This is also reasonable because the design then maximizes the bene¯t that the freeway o®ers, compared to local street travel.

The cost of an AHS ramp will exceed that of a conventional ramp by some value A, which is de¯ned in arbitrary monetary units. This cost di®erence A should be of the form $a + bQ_0$ (a ¯xed component plus a variable component which is proportional to the exit °ow, Q_0), for some constants a and b. The same can be said for entry ramps. Suppose as well that each kilometer of AHS costs c monetary units <u>less</u> than a freeway kilometer. If we choose as our monetary unit the cost of one lane-km of conventional freeway, and denote it ±, then we conjecture that c ¼ 1:5±, implying that a single AHS lane is approximately 50% more expensive per unit length than a single freeway lane, when three freeway lanes are being constructed.

We assume that enough trips are generated to require full use of the freeway; i.e. $ql = 6000$, where q is the entry °ow generated per kilometer of freeway. This assumption is favorable to AHS because with less demand, conventional freeways could be built with only two lanes, but the AHS system could only realize savings through smaller (or fewer) on- and o®-ramps. We treat q as a constant that is independent of the system design, because we have assumed that d is small compared to l; e.g. $3d < l$. Note that $Q_0 = qd$, allowing us to write the extra AHS-ramp cost per kilometer as $(1=d)(A) = (a + bQ_0)=d = a=d + bq = a=d + 6000b=l$. Thus, the AHS savings per kilometer are:

$$c - \frac{a}{d} - \frac{6000b}{l}; \qquad \text{with} \quad 3d < l \tag{12}$$

Using $c = 1.5\delta$ and $d \approx l/3$, this yields savings of $\delta[1.5 - (3a + 6000b)/l]$. The choice of $d \approx l/3$ is the most favorable for the AHS, because the savings is an increasing function of d. If we estimate $a \approx 3\delta$ and $b \approx 2000^{-1}\delta\cdot$hr/veh, then the savings are approximately $\delta(1.5 - 12/l)$. Thus, we would need $l \geq 8$ km to break even, or $l \geq 24$ km to save the cost of one freeway lane-kilometer. In view of the favorable assumptions we have made, these results seem to rule out efficient use of AHS to relieve congestion near city centers. The results also suggest that one role of the control mechanism allowing entry to the automated highway would be to discourage short trips from using the AHS, thereby effectively increasing the average trip length.

These results are sensitive to the conjectured parameters used, but (12) holds nonetheless. When estimating a, it is important to include the cost of the auxiliary transfer lane, which might be about two kilometers long, plus the extra barrier, and additional structure if the ramp is elevated, etc.

CONCLUSIONS

We have shown that estimates of AHS capacity that prevail in the literature are somewhat optimistic, and are likely being used improperly in comparison with understood capacities of conventional freeways. The entry and exit maneuvers have a distinct effect on steady-state capacity; however the AHS system still shows promise when compared to existing facilities, particularly when incorporating a system for judiciously scheduling vehicles into platoons according to their destinations.

A more troublesome capacity constraint exists at the interface with the local street system. This constraint is what causes a large proportion of existing congestion on conventional freeway systems; in these cases improving the capacity does nothing to solve the congestion problem, although increased storage can be bene¯cial[7]. Existing freeways have the ability to store many vehicles in queues during rush periods. While these queues are certainly frustrating to drivers, this is nonetheless an important function of freeways, as these queues would be stored on the local streets otherwise. Because of the reduction in lane-mileage available for storage under AHS, this critical function of the freeways will be impaired.

Partly because of the above reasons, there may only be certain \boutique" locations for which AHS implementation is a realistic solution to congestion problems. One should certainly avoid constructing AHS where downstream termini are congested and likely to \back up" onto the AHS. In order for AHS to be economically viable, large ramp spacings and long trip lengths might be required. The average trip length can be manipulated slightly by refusing admission to those vehicles with nearby destinations. These constraints, however, could rule out the use of AHS near congested city centers.

Likely candidates for AHS might include closed loop systems such as large diameter ring roads which enclose large urban centers. These systems typically serve long trips, do not have a predominant terminus, and tend to exhibit fairly balanced °ows around the loop. Other candidates might be tunnels, bridges, or other forms of infrastructure which are very expensive to construct, but which would not directly feed congested areas. Unfortunately, this \boutique" deployment of AHS systems is unlikely to encourage users to incur the extra expense of an appropriately equipped automobile. We must conclude that if AHS technology is to become widespread, the impetus will have to come from its comfort and (possibly) safety advantages.

The results presented herein are mainly intended to provide insights into some of the important issues surrounding automated highway systems and their implementation. The level of analysis with which these issues are studied does not warrant very strong conclusions; it su±ces to identify and clarify where some potential problem areas may lie. As these systems become closer and closer to being realized, it is important that these issues be brought to the forefront, together with safety issues not addressed in this paper, and that critical resources not be spent in vain on e®orts to install AHS technology where it cannot possibly succeed.

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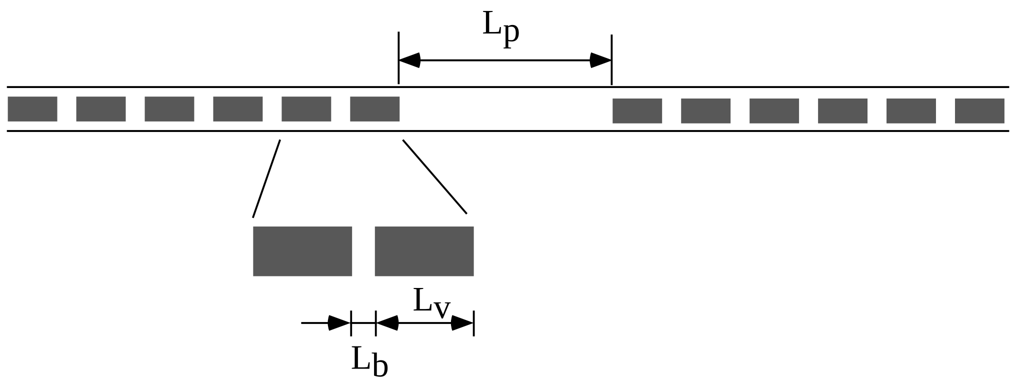

Figure 1: Scheme of platooning.

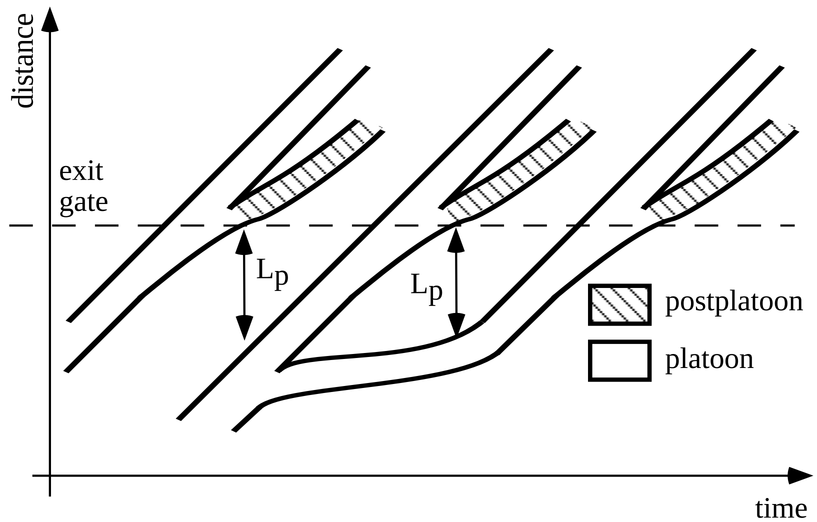

Figure 2: Extra interplatoon distance required for the exit manuever.

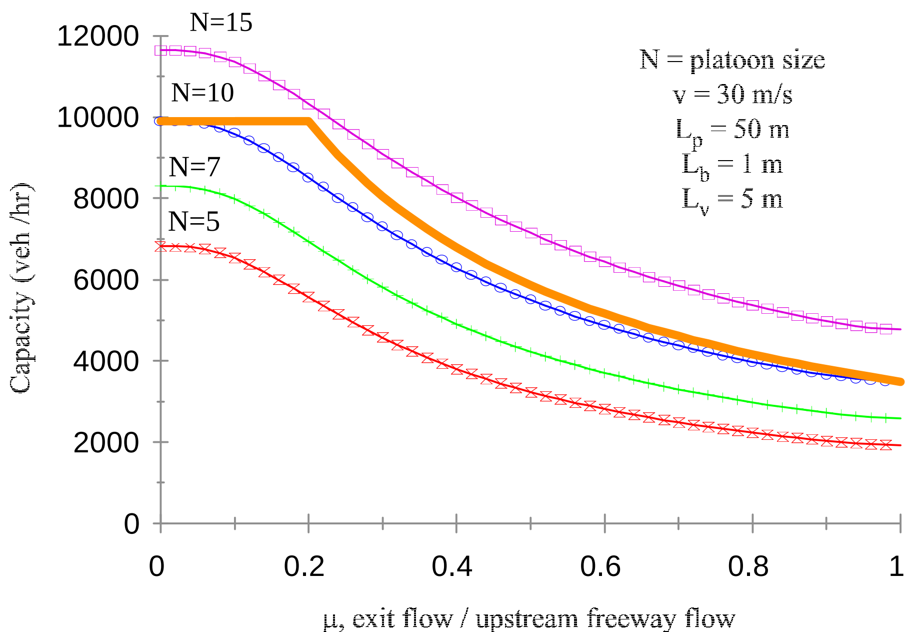

Figure 3: Capacity of an automated lane with two exit gates.

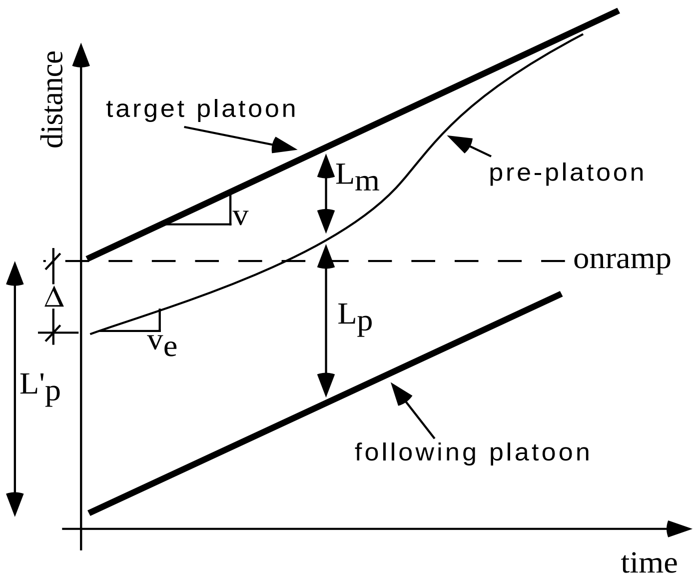

Figure 4: Gap necessary to merge new platoon between two existing platoons.

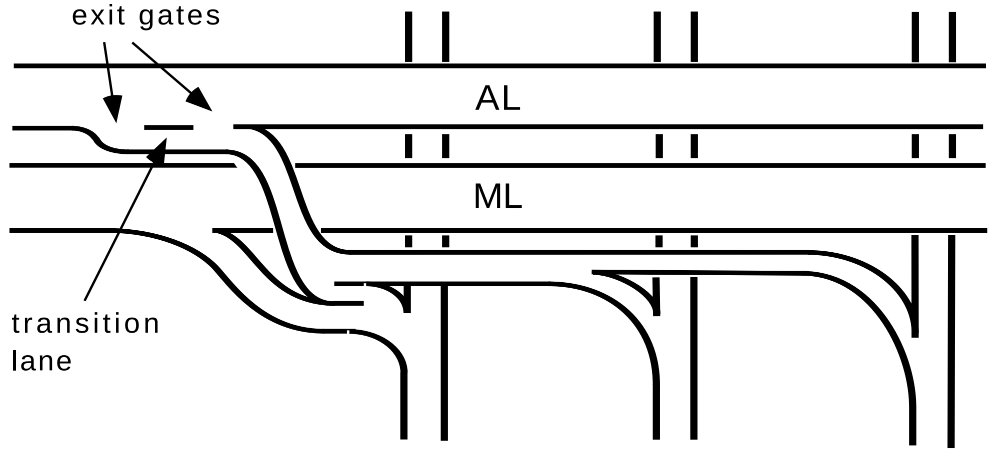

Figure 5: Exit ramp design for enhanced exit capacity.