Title

Steady State Conditions on Automated Highways

Authors
del Castillo, Jose M.
Lovell, David J.
Daganzo, Carlos F.

Publication Date
1995-11-01

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José M. del Castillo
Escuela Superior de Ingenierías Industriales
Universidad de Extremadura
06071 Badajoz, SPAIN

David J. Lovell and **Carlos F. Daganzo**
Institute of Transportation Studies
University of California, Berkeley
Berkeley, California 94720

November, 1995

Abstract:

This paper is concerned with technical investigations of traffic operations on automated highways. Estimates are made of the steady-state capacity of such freeways, paying particular attention to the effect of entry and exit maneuvers. The possibility of scheduling departing vehicles appropriately into platoons to minimize extraneous maneuvers is investigated. Characteristics of urban areas likely to be candidates for automated freeways are discussed, and some shortcomings of automated freeways, vis-à-vis conventional freeways, are pointed out. Finally, some areas of future research needs are identified.

*****Acknowledgements:** Research supported by the PATH program, Institute of Transportation Studies, University of California, Berkeley, under MOU-165. José M. del Castillo would like to thank the Spanish Ministry of Education and Science for the scholarship that has made this work possible.

Contents

1 INTRODUCTION

1.1 BACKGROUND

One of the most compelling research topics currently active in the field of transportation is automated highways. This type of highway will serve specially-equipped vehicles with onboard hardware that allows the vehicles to drive themselves, much like an autopilot on an aircraft. The automated features include acceleration and deceleration (braking), steering, and a number of required maneuvers, including merging and splitting of platoons. The vehicles will be controlled by means of two-way radio communications, both vehicle-to-vehicle and vehicle-to-controller. The controller is the hardware installed in and near the automated highway which determines how best to operate the stream of automated vehicles, and generates the radio commands which instruct the vehicles to behave accordingly.

This paper was written to present research results generated as part of a greater effort towards the understanding of hybrid freeways; i.e. freeways which serve (at least) two distinct types of traffic. There is a concern that the existing literature on automated highways systems (AHS) is overlooking some critical concerns, and that prediction of benefits are being made using definitions of concepts different from their traditional transportation engineering interpretation. This is probably due in large part to the fact that most of the research in AHS is conducted in the fields of electrical, computer, and mechanical engineering, and hence suffers from the absence of a true transportation engineering perspective.

1.2 PROBLEM IDENTIFICATION

Most existing literature on the capacity of automated highways systems deals first with the steady state capacity of a hypothetical straight pipe link in which there are no entries and exits. From this analysis, the "capacity" of an AHS is derived, presumably to be compared with understood capacities of similar, non-automated facilities. Of course, the caveat is given that once lateral flows (i.e. entering and exiting vehicles) are introduced, the achievable flows on the system are much less. The problem here is that this caveat is typically already taken into account when considering capacities of traditional transportation systems. The capacity of a system is a quantity used to determine long-term functional ability, and should represent a sustainable level of performance on a real-life system, not the maximum theoretical flow possible on a hypothetical system that will never exist. Comparisons of capacities between automated and conventional freeway systems should be made from the perspective of steady-state conditions, including all expected regularly-occuring events, such as entries, exits, lane changes, shockwaves, and even faults (if one wants to present a worst-case scenario).

Existing literature on the capacity of automated highway systems focuses on the analysis of single subsections. It is assumed that if the operation of a single subsection, including entry and exit gates, and manual and automatic lanes, can be controlled, then a working automated freeway system can be constructed simply by connecting a number of these subsections together. This logic is faulty, however, for the same reason that existing conventional freeways cannot be studied in this manner. What often controls the flow of traffic on many facilities is the ability of the destination(s) to accommodate traffic. For example, traffic on a freeway that reaches its terminus in a downtown area is restricted by the ability of the local city streets and parking areas to absorb the incoming traffic. No improvements to speed, flow, safety, or other conditions on the freeway can increase the input flow of traffic to the city center beyond the absorption capability of the terminus.

Offramps prove to be a similar problem. It is not reasonable to suppose that all offramp destinations will always be able to unload their demand. We should recognize that there may be a parameter such as the maximum absorption rate by local streets per unit mile of automated freeway that can be defined for various street systems or urban areas.

It is also not reasonable to suppose that there will always be sufficient real estate to build temporary storage for whatever queues may accumulate as a result of this. Thus, a thorough study of the capacity of automated highway systems must incorporate parameters regarding the capacity of the destination nodes. The fact that queues grow very quickly in reaction to very fast shockwaves on automated lanes suggests that it may be wise to restrict their use to closed loops bypassing congested termini that may generate such shocks.

There may be certain types of freeways, or certain possible locations of freeways, which are better suited for automation than others. One example of such is a ring freeway, or beltway, due to the fact that there is no freeway terminus. Although the offramp 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 offramps typically have greater capability of absorbing the incoming traffic.

1.3 RESEARCH NEEDS

While this paper delves into the subject of capacities of automated lanes, there is considerably more detail with which this can, and should, be studied. The main goal of this paper is to generate an awareness of critical issues and potential pitfalls. Further research should be conducted to define in more detail exactly how the impementation-specific parameters of an AHS will affect its capacity. Examples of such parameters include the nature of the freeway (linear, beltway, etc.), what type of urban area is being served, the design of the entries and exits, the availability of on- and off-ramp capacity, etc.

Other issues have come to mind while working on this current research, and although answers will not be provided here, a quick synopsis of the issues is given solely to activate interest in them. The question of safety has been addressed to some extent, but there are still gaps in the knowledge, particularly with respect to real-world empirical testing of automated operations under high-risk conditions. Also, most theories on the control of automated vehicles are generated under the assumption that the system is always fully functional. Some researchers have begun to study modes of failure of an AHS, but much more work is needed. Finally, once the consequences of studying automated highway systems with a slightly less optimistic (and probably more realistic) outlook are fully known, the question of economic viability must be addressed. These systems represent a huge investment in research, testing, and ultimately, implementation. While one may argue that the first two categories are wise undertakings in that they will yield side benefits (e.g. safer cars and better informed drivers), even if an AHS system is never built, the same cannot be said of implementation. In our view, an implementation decision requires an unbiased awareness of all of the potential benefits and disbenefits of these systems, not just by researchers and practitioners, but also by the general public.

The remainder of this report is dedicated to technical investigations of automated freeways. Section 2 deals with the steady-state capacity of automated highways. The platoon structure will be investigated thoroughly, including specific requirements for merging and diverging traffic at exits and entries. The exit maneuver tends to have the greatest impact on the capacity, so it will be examined in more detail, including the effect that limited exit ramp capacity may have on the capacity of the system. Section 3 investigates how to determine if a particular urban area is a promising candidate for an AHS. Section 4 looks more closely into the issue of storage, an important function that is widely recognized and observed on existing freeways (i.e. morning traffic jams), but seems to have been neglected in studies of AHS to date. Finally, Section 5 offers some parting comments and insights.

2 CAPACITY ESTIMATION

2.1 PLATOONING ON THE AUTOMATED LANES

Traffic flow on the automated lanes is organized in platoons of closely spaced vehicles with a large spacing between platoons. This traffic concept is called "platooning". Figure 1 depicts this concept. The variables that determine the capacity of the automated lane (AL) are:

- the intraplatoon distance, or bumper-to-bumper distance between vehicles of the same platoon, L_b;
- the interplatoon distance, or distance between consecutive platoons, L_p;
- the vehicle length, L_v;

- the vehicle speed, V;
- the platoon size, or number of vehicles per platoon, N.

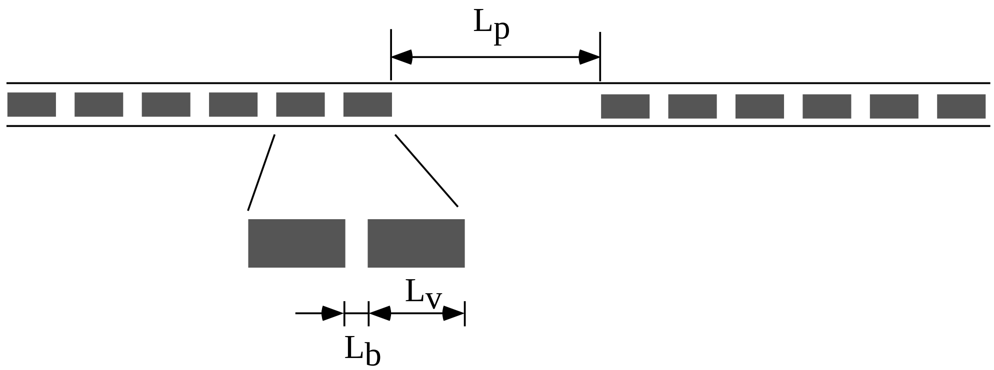

Figure 1: Scheme of platooning.

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

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

in units of meters, for example. Because there are N vehicles within this spacing, the density of traffic 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 (veh/hr), is given by:

$$C = \frac{3600VN}{NL_v + (N-1)L_b + L_p} \tag{2}$$

The above capacity formula has been used to support automated highways (see for example Varaiya, 1994). Very high capacity values are predicted, even with platoons of moderate length. The increase of capacity is not the only justification for the platooning concept. It is also claimed in the literature that platooning favors safety (e.g. Hitchcock, 1994). This claim is based on the fact that the relative speed of collision between vehicles reaches a maximum for intermediate values of the distance between them, whereas it is small for both short and long distances. Hence, some researchers conclude that in case of a system breakdown, the risk for the drivers is reduced by platooning.

Moreover, the interplatoon distance is a function of the speed. Godbole and Lygeros (1994) assume that the interplatoon distance is a linear function of the speed:

$$L_p = \lambda_v V + \lambda_p, \tag{3}$$

where λ_p is the fixed minimum interplatoon distance, and λ_v is the rate at which this distance increases with speed, in units of meters per meter per second, or simply, seconds. Such a distance between platoons is chosen so that if the preceding platoon applies maximum deceleration to come to rest, the following platoon should be able to react without risk of collision. They take $\lambda_p = 10$ m and $\lambda_v = 1$ s. Figure 2 shows the capacity-speed curves for different values of the platoon size, when the interplatoon distance is given by (3), the capacity by (2), and $L_b = 1$m, $L_v = 5$ m. These simple calculations show that, in principle, very high flows may be achieved on automated highways by means of platooning.

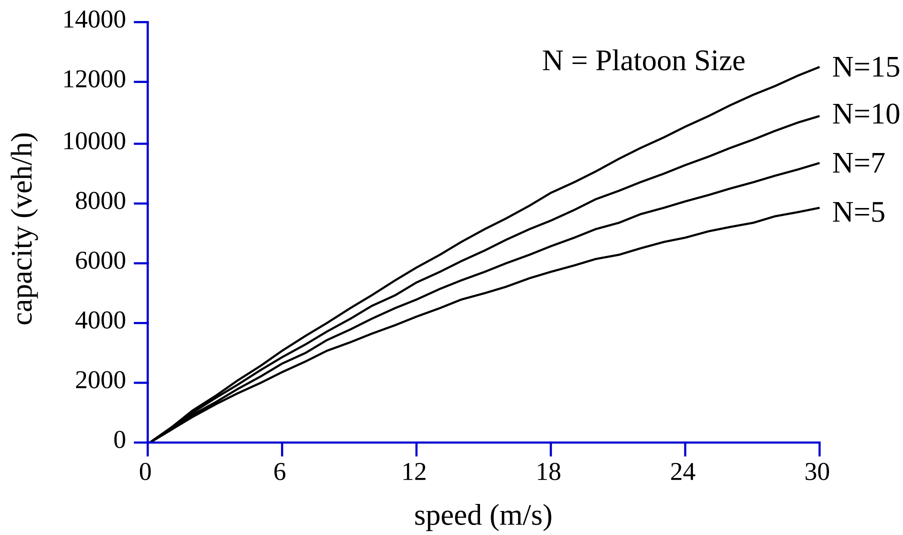

Figure 2: Capacity as a function of the speed.

2.2 EFFECT OF THE EXIT MANUEVER ON THE CAPACITY

The exit ramps of the automated lanes consist of a given number of exit gates connecting those lanes to a transition lane placed between the automated and the manual lanes. Since there is no need for a continuous transition lane, it can be designed to merge with the exit ramp from the manual lanes as shown in Figure 3, where AL and ML stand for automated and manual lanes, respectively. For safety reasons, there must be a physical barrier between the transition and the automated lanes which can be crossed only through the exit gates.

Since a safe execution of the exit manuever does not allow different vehicles from the same platoon to exit through the same gates (Varaiya, 1994), a platoon should be split upstream of the exit gates if the number of exiting vehicles it contains exceeds the number of gates. This is carried out by the split manuever, which decelerates the leader of the new platoon until it is at a safe distance from the preceding platoon. This process may need to be repeated more than

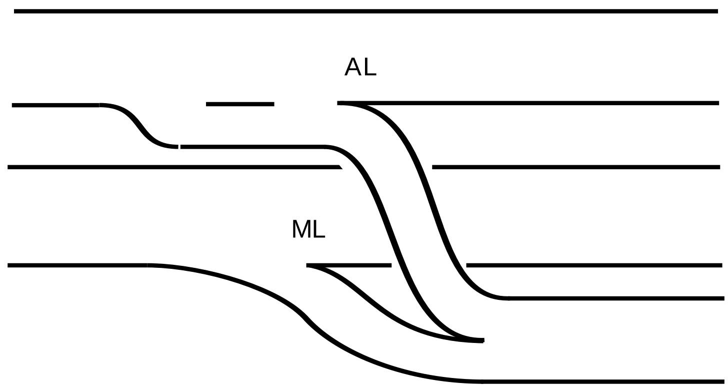

Figure 3: Exit ramp with two exit gates.

once, until there are no more vehicles in any one platoon that desire to use the upcoming exit 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 4 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 exit gates. The second platoon was split and the trailing platoon created in this manuever was decelerated until a new gap was created of length equal to the interplatoon or safety distance L_p. If this gap did not already exist in the traffic stream, a backward-moving shockwave would travel upstream, until such a point that this transient surge could be absorbed. For calculation of steady-state capacities, this space must be assumed to exist, and must be taken into account. In the figure, it has been assumed that the exiting vehicles are those in the rear positions in the platoons, but this need not be the case. Finally, the exiting vehicles form a platoon of their own once they pass through the exit gates. This platoon is called a 'postplatoon'.

A direct consequence of the split manuever is the capacity reduction caused by the necessity of accommodating the extra distance, L_x, required by the splits. Thus, the capacity formula given by (2) needs to be redefined in the following manner:

$$C = \frac{3600VN}{NL_v + (N-1)L_b + L_p + E[L_x]} \tag{4}$$

where $E[\cdot]$ denotes expectation. The extra distance, L_x, is a random variable, since the number of exiting vehicles varies across platoons. Clearly, L_x will depend on the composition of the platoon; that is, on the destination of its vehicles. If we assume that a vehicle's destination is not taken into account when granting or denying it access to any particular platoon, the number of exiting vehicles will follow a binomial distribution. Namely, the probability of finding n exiting vehicles in a platoon of N vehicles will be:

$$p_n = \binom{N}{n} \mu^n (1-\mu)^{N-n}$$

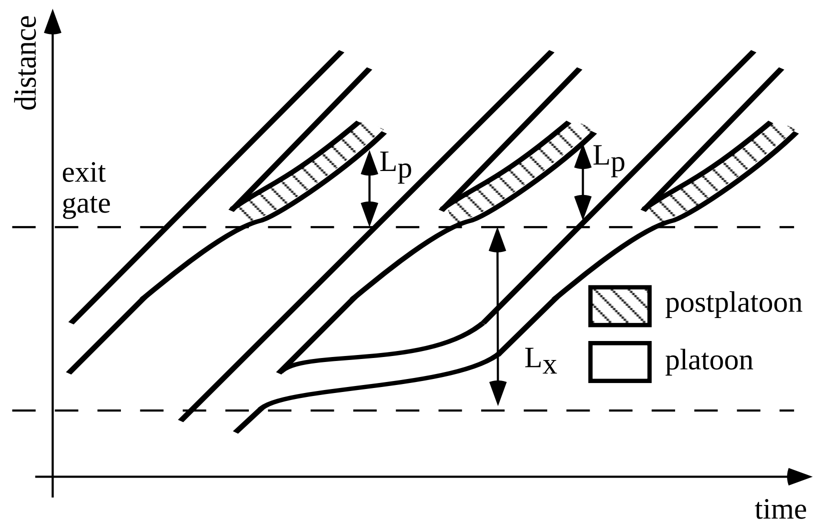

Figure 4: Extra interplatoon distance required for the exit manuever.

where μ is the fraction of the upstream freeway flow desiring to use the exit in question. With the previous assumption, one can easily calculate the mean additional distance required for the exit manuever. If there is only one exit gate,

$$E[L_x] = [p_2 + 2p_3 + 3p_4 + \ldots + (N-1)p_N]L_p = \Big[\sum_{n=2}^{N}(n-1)p_n\Big]L_p = \beta_1 L_p$$

From the mean of the binomial distribution:

$$N\mu = \sum_{n=1}^{N} np_n = \sum_{n=1}^{N} p_n + \sum_{n=2}^{N}(n-1)p_n = 1 - p_0 + \sum_{n=2}^{N}(n-1)p_n$$

then

$$\beta_1 = \sum_{n=2}^{N}(n-1)p_n = N\mu - 1 + p_0 = N\mu - 1 + (1-\mu)^N \tag{5}$$

If the exit ramp has two gates, the average exit distance is given by:

$$E[L_x] = [(p_3 + p_4) + 2(p_5 + p_6) + 3(p_7 + p_8) + \ldots]L_p = \beta_2 L_p$$

Then

$$\beta_1 - 2\beta_2 = p_2 + p_4 + p_6 + p_8 + \ldots + \begin{cases} p_N, & \text{if } N \text{ is even} \\ p_{N-1}, & \text{if } N \text{ is odd} \end{cases}$$

Suppose we define for any real numbers a and b and positive integers $N \geq n$:

$$s_n = \binom{N}{n} a^n b^{N-n}$$

Then from the binomial theorem,

$$(a+b)^N = \sum_{n=0}^{N} \binom{N}{n} a^n b^{N-n} = \sum_{n=0}^{N} s_n = s_0 + s_1 + s_2 + \ldots + s_N$$

In addition,

$$(b-a)^N = \sum_{n=0}^{N} \binom{N}{n} (-1)^n a^n b^{N-n} = \sum_{n-0}^{N} (-1)^n s_n$$

$$= \begin{cases} s_0 - s_1 + s_2 - s_3 + \ldots + s_N, & \text{if } N \text{ is even} \\ s_0 - s_1 + s_2 - \ldots - s_N, & \text{if } N \text{ is odd} \end{cases}$$

Therefore,

$$\frac{1}{2}\Big[(a+b)^N + (b-a)^N\Big] = s_0 + s_2 + s_4 + s_6 + s_8 + \ldots + \begin{cases} s_N, & \text{if } N \text{ is even} \\ s_{N-1}, & \text{if } N \text{ is odd} \end{cases}$$

In particular, if we let $a = \mu$ and $b = 1$ - μ, then we arrive at the following:

$$\frac{1}{2}\Big[1 + (1-2\mu)^N\Big] = p_0 + p_2 + p_4 + \ldots + \begin{cases} p_N, & \text{if N is even} \\ p_{N-1}, & \text{if N is odd} \end{cases} = \beta_1 - 2\beta_2 + p_0$$

Then, taking advantage of (5) and bearing in mind the expression for p_0, one finally gets:

$$\beta_2 = \frac{N\mu}{2} - \frac{3}{4} + (1-\mu)^N - \frac{1}{4}(1-2\mu)^N \tag{6}$$

It is possible to express the capacity as follows:

$$\frac{1}{C} = \frac{1}{C_0} + \frac{\beta L_p}{3600VN} \tag{7}$$

where C_0 is the nominal capacity given by (2) if the exit manuever does not demand any extra interplatoon distance, and β is given by (5), (6), or their generalization. Figure 5 depicts the revised capacity as a function of the exit flow ratio μ, 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 shortly.) Thus, the capacity has been calculated by substituting β_2 for β in (7). The figure shows that very high values of the capacity can be obtained even for moderately high values of the exit flow ratio. However, in the interval $0.1 \leq \mu \leq 0.4$, the capacity decreases significantly with increasing μ. It should be said, on the other hand, that locations where automated highways could be beneficial would likely exhibit a low exit flow ratio (see Section 3).

If the freeway is operating at capacity, then the exit flow, q_c, can be calculated from

$$q_c = \mu C$$

This mean exit flow has been plotted in Figure 6, for two exit gates. The exit flow at capacity depends weakly on the platoon size.

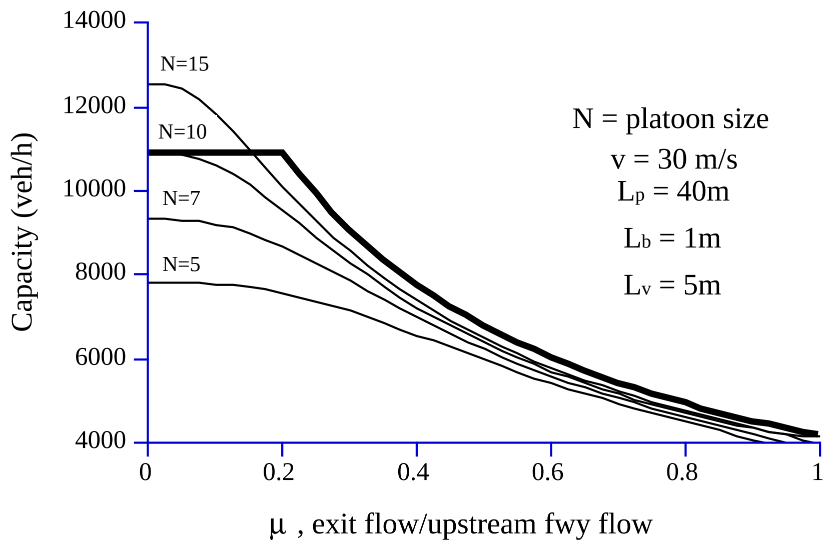

Figure 5: Capacity of an automated lanes with two exit gates.

2.3 SCHEDULING VEHICLES INTO PLATOONS

One may argue that capacity flows may be increased even more by "scheduling" entering vehicles into passing platoons so as to make sure that platoons passing the most congested approach to an 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 over a long period of time, 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 yet been explored, we examine below the improvements to capacity that can be achieved with scheduling and interpret the results as an upper bound to the capacity.

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, $\overline{E_b} = \mu N/G$. In the unlikely event that $\overline{E_b}$ is an integer, then conceivably every platoon can be constructed with exactly $\overline{E_v}$ exiting vehicles, and the number of splits will be minimized. Should this prove not to be the case, then the traffic stream should be split into two types of platoons:

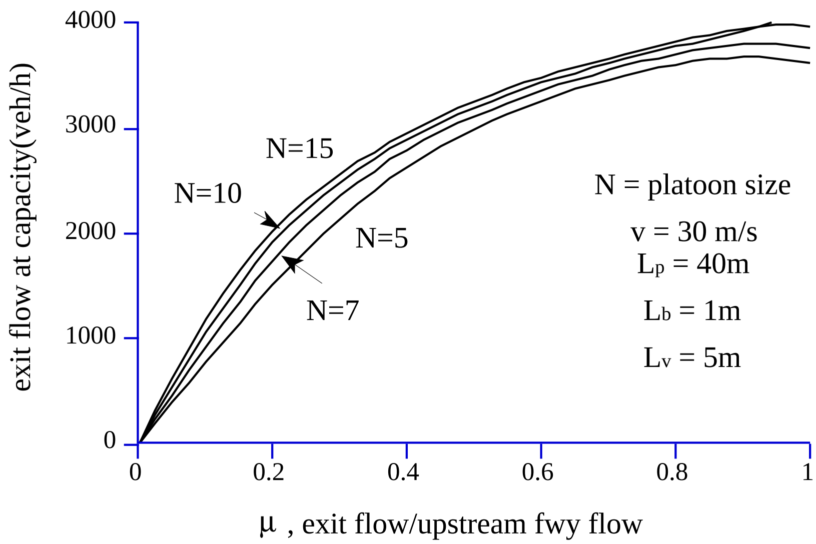

Figure 6: Maximum exit flow from an automated lane with two exit gates.

those carrying $\lceil \overline{E_b} \rceil^+$ exiting batches, and those carrying $\lceil \overline{E_b} \rceil^-$ exiting batches.[1] Again, this construction has the benefit of minimizing the total number of platoon splits that needs to take place[2].

Let f denote the fraction of platoons carrying $\lceil \overline{E_b} \rceil^+$ exiting batches. In the long run, after the passage of $M \to \infty$ platoons, f must satisfy the following:

$$MN\mu = GM\left[f\lceil \overline{E_b} \rceil^+ + (1-f)\lceil \overline{E_b} \rceil^-\right] = GM\left[\lceil \overline{E_b} \rceil^+ + f - 1\right], \tag{8}$$

since the number of vehicles wishing to exit must match the actual number exiting. Each platoon requires one less split than the number of exiting batches it contains, unless the platoon contains no exiting batches, in which case no splits are required. Therefore, the average number of splits per platoon is:

$$\beta_G = \begin{cases} f\left[\lceil \overline{E_b} \rceil^+ - 1\right] + (1-f)\left[\lceil \overline{E_b} \rceil^- - 1\right], & \text{if } N\mu/G \geq 1 \\ 0, & \text{otherwise} \end{cases} \tag{9}$$

[1]Here the notation $\lceil \cdot \rceil^+$ denotes rounding up to the next integer, while $\lceil \cdot \rceil^-$ denotes rounding down to the next integer. Clearly, $\lceil \overline{E_b} \rceil^+ = \lceil \overline{E_b} \rceil^- + 1$.

[2]Since the number of splits is a convex function of the number of exiting cars, the minimum average number of splits is obtained by an average of the nearest integer neighbors to the (non-integer) average number of exiting cars.

Rearranging (8) yields

$$f = \frac{N\mu}{G} - \left\lceil \overline{E_b} \right\rceil^+ + 1, \tag{10}$$

which is simply the fractional part of $\overline{E_b}$. Substituting this for f in (9) and performing a little algebra, we find:

$$\beta_G = \max\left\{0, \frac{N\mu}{G} - 1\right\} \tag{11}$$

An upper bound for the capacity is obtained by substituting this expression for the parameter β of (7). The heavy line in Figure 5 depicts this upper bound for the case $G = 2$ and $N = 10$. In this particular case, the upper bound seems to offer a noticeable improvement in the range $0.5 \lesssim N\mu/G \lesssim 2$.

Whether the system can/should be operated by "scheduling" vehicles into platoons is not a critical issue, however, because in the range of μ where such an operation would make a difference, the capacity obtained without scheduling vehicles is already quite high. We therefore ignore the possibility of scheduling vehicles in what follows. This is reasonable, because the values of μ that are likely to arise in scenarios conducive to automated highway systems should be relatively low.

2.4 THE EFFECT OF ENTERING VEHICLES

Unfortunately, the effect of the entering flows is more significant and should not be ignored. This observation, corroborated by the simulation experiments in Rao et. al. (1993) and Tsao et. al. (1994), can be understood easily if we recognize that the platoon separation must be greater than that which would ensure a safe operation away from the ramps, L_p. Figure 7 displays the time-space trajectories for the gap in 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" (periodic boundary condition).

A vehicle merging at speed $v_e < v$, 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 rate of a m/s^2. It should be clear from Figure 7 that to maintain a gap of size L_p between the merging vehicles and the trailing platoon, the interplatoon distance upstream of the "diamond interchange" should be at least $L_p + L_m$.

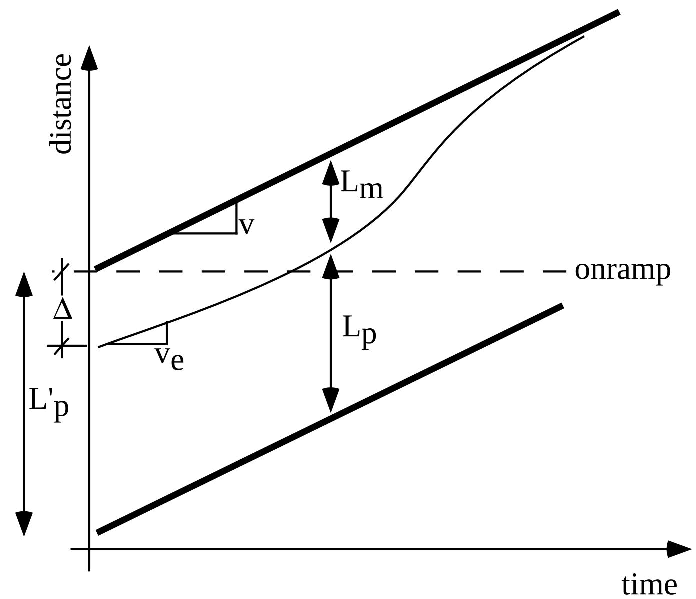

Figure 7: Gap necessary to merge new platoon between two existing platoons.

We note that for obvious reasons 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.

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 (7). For example, for $L_m = 20$ m, a 20% reduction in capacity results for the data of Figure 5 if $N = 10$ and $\mu \lesssim 0.2$. The fractional reductions are larger for smaller N and larger μ and can approach 30%.

Despite the downward corrections it would appear from Figure 5 that an AHS system could still pump traffic on a single lane past a diamond interchange at rates upwards of 8000 veh/hr if one can keep μ below 0.2 (for $G = 2$). This is still better than two lanes of conventional freeway, although not as high as initially thought.

In the remainder of this paper we explore two other questions: (i) where should an AHS be installed to achieve maximum benefit, and what would be the nature of said benefits? and (ii) what are some of the implications of installing an AHS in a less-than-optimal location? Section

3 shows that AHS diamond interchanges must be widely separated. This increases μ and as a result the exit flows of critical off-ramps must be many times larger than those of conventional ramps. Careful attention must thus be paid to the interface between the AHS system and the conventional system so as to make sure that the local street system can absorb the exit flows. It is concluded that AHS systems should not discharge traffic into congested local streets (e.g. near a Central Business District, CBD) because off-ramp queues would grow into the AHS with rather undesirable effects (see Section 4). A possible AHS application would be to metropolitan area beltways of wide diameter, as explained in Section 3. Section 4 explores the question of storage capacity of an AHS; it shows how by giving up the storage capacity of conventional freeways, an AHS can fill up with queues much more quickly, with the distinct possibility that upstream onramps can become blocked.

3 SCENARIOS FOR AHS

3.1 EXIT RAMP REQUIREMENTS

In the previous section, it has been demonstrated that, theoretically, very high flows on the order of 8000 veh/hr may be achieved on an automated lane. For some scenarios, the exit flow that would have to be accommodated to avoid mainline queues would also have to be considerably greater than that typical of a manual lane; e.g. on the order of 2400 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 that of an exit ramp 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 AHS exits should be designed so as to eliminate this potential bottleneck.

An exit flow of 2400 veh/hr is about three to four times the typical capacity of an exit ramp from a manual lane. The only possible 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 8. The transition from automated to manual driving should be carried out after the flows have been split. The additional construction cost of the exit ramps may become an important part of the total cost of the AHS, and this should be recognized in evaluation. In any case, the necessity of increasing the exit capacity of the automated lane must be borne in mind when designing the AHS. We also recognize that even if the ramps are properly designed, one still has to make sure that the local street system can absorb the flow.

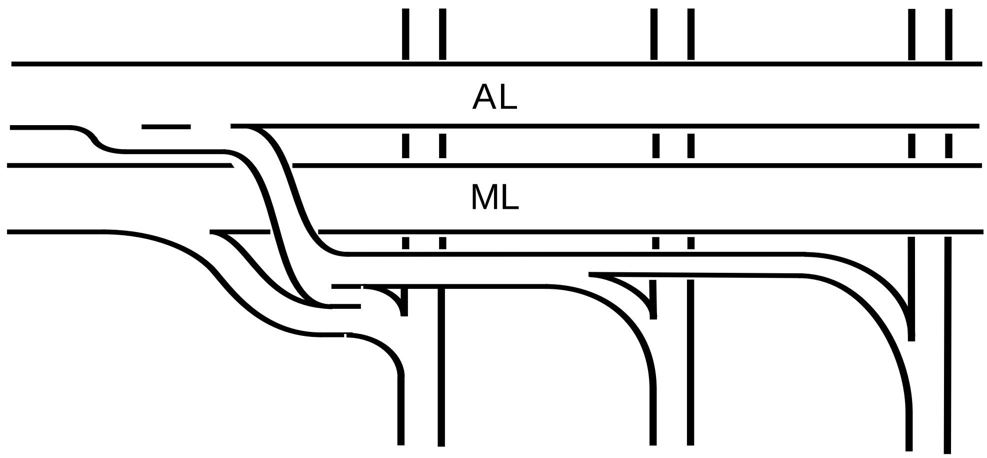

Figure 8: Exit ramp design for enhanced exit capacity.

3.2 ABSORPTION ONTO LOCAL STREETS

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

A freeway (or AHS lane), located in a region that is able to absorb f vehicles per hour per linear meter 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. This is a very important condition that bounds the capacity; i.e. there is no incentive to build a facility that will carry more than fl vehicles per hour. For example, a 10 km beltway for which l would be expected to be on the order of 2.5 km, and ringing a CBD with $f \approx 1$ veh/hr-m, 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. $l \approx 10$ km) but this would require off-ramps able to serve 1000 veh/hr to be placed approximately every kilometer.

Appendix A shows that AHS off-ramps must be spaced at least $d = 5$ km apart and will have to carry substantial flows (approximately Qd/l). These flows can be handled as in Figure 8 if Qd/l is greater than what a single street can absorb. They will have to be spread over a length $d' = (Qd/l)/f$. The quantity $Q/(fl)$ represents the proportion of local streets that must be reached by ramps, and given the geometry of Figure 8, the ratio $d'/d \approx Q/(fl)$ gives the proportion of the AHS lane that must be overlapped by a service lane to handle exits. A similar proportion would be required to handle entries.

Thus, in order to save the construction cost of an extra auxiliary lane, we must require:

$$2Q/(fl) << 1.$$

An AHS lane with $Q = 8000$ veh/hr would require $fl >> 16000$ veh/hr. For an average trip length of 10 km this would require $f >> 1.6$; e.g. $f \approx 10$, which seems large, even in suburban areas. (A two-lane off-ramp ending in a traffic signal may carry 1500 veh/hr onto an arterial street; thus $f \approx 10$ means that there would have to be six such streets per kilometer.)

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 strongly that AHS freeways cannot be a solution to the congestion problems in cities. The idea is further reinforced by the results of the next section, 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 than the traditional freeway.

4 STORAGE CAPACITY

The previous sections have been devoted to the analysis of the capacity of the automated freeways. However, nothing has been said about the vehicle storage capacity of such systems. This term refers to the potential of the automated freeway to accommodate a given number of vehicles when the system breaks down; that is, when severe congestion occurs. We are not interested in studying the possible causes of the onset of congestion but its consequences for the traffic flow on an automated freeway. For this reason, we believe it is sufficient to focus on the storage capacity of the system without needing the help of a dynamic model of traffic flow. We think this is a very important issue that has been surprisingly overlooked.

4.1 JAM DENSITY

The storage capacity of a highway is entirely conditioned by the jam density; that is, the maximum density at which vehicles can be stored on the freeway under highly congested conditions. It is very easy to estimate the jam density of an automated freeway if one assumes the interplatoon distance proposed by Godbole and Lygeros (1994) and given by (3). Then, the jam density of a single lane automated freeway becomes:

$$K_j = \frac{N}{L_p(0) + NL_v + (N-1)L_b}, \tag{12}$$

where $L_p(0)$ is the interplatoon distance at zero speed, whose value is 10 m according to (3), when $\lambda_p = 1$ 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 $N = 15$, so that we can assume an average value of 140 veh/km. This value 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[3]. 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. Although some of the arrangements proposed for automated freeways do not require a continuous transition lane, we believe that for urban freeways where off- and on-ramps have to be closely spaced, the transition sections will occupy a significant length of the freeway. This will lead to the almost complete utilization of one of the original lanes. The resulting loss of storage capacity has to be considered in the final design[4].

The second reason is the increase in capacity achieved by the automated freeway itself. Obviously, this improvement should lead to much higher traffic flows. Accordingly, if for some reason the traffic flow collapses, the number of vehicles to store per unit time will be much greater than that of a conventional lane. In other words, the resulting queue will grow much more rapidly and will become much longer. Figure 9 illustrates the effect of an incident of a given duration for a four-lane freeway, assumed first to be a conventional freeway, and second to have acquired automated vehicle capability. Both systems are assumed to be at capacity. Before the construction of the automated lane, each lane carried a traffic flow of C veh/hr, so an incident of a given duration would create a queue of given length (shown for illustration purposes as three vehicles; the actual length is dependent on the actual duration of the incident and other parameters). The automated lane can carry $4C$ veh/hr, meaning that the capacity of the freeway as a whole increases by 50%. In this condition, an incident creates a queue four times longer on the automated lanes than on the manual lanes.

4.2 SHOCK WAVES

The reader familiar with traffic engineering analysis will recognize the situation if s/he plots the flow-versus-density (e.g. $q(k)$) curve for the AHS lane and some typical data; e.g. $v = 30$ m/s, $N = 10$, $L_p = 40$ m, etc. See Figure 10 for an example of such a curve. While the exact shape of the curve is not known, we know it must pass through the points 'O', 'C', and 'J'. The slope of the dashed line connecting states 'C' and 'J' in the figure is the velocity of growth of the queue (also called the shockwave speed), which is on the order of -34 m/s, approximately the same magnitude as the prevailing traffic speed. Although not shown in this figure, similar velocities will occur for transitions between any two queued states (i.e. with densities in the range $k \in (0.07, 0.14)$ veh/m). Although the passage of such a fast shockwave should pose no problem for a properly functioning AHS (it essentially requires a reaction time of at most one

[3]Another meaningful comparison would be that of a newly built AHS lane vs. an equivalent 4-lane conventional freeway.

[4]The storage losses are more severe when one compares the AHS with a conventional freeway of the same capacity. Obviously, the storage is cut by a factor of 4 in this case.

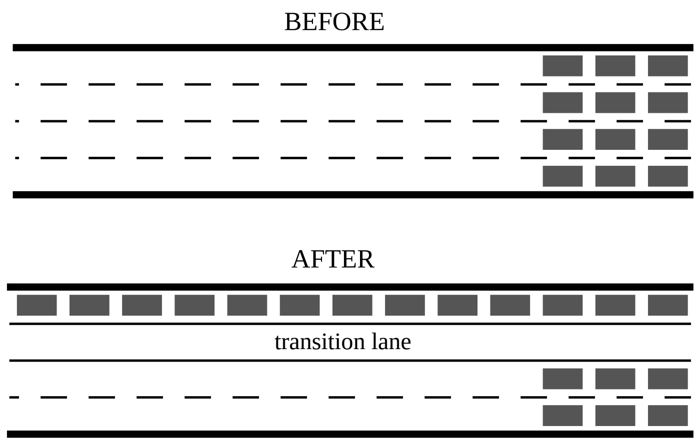

Figure 9: Effect of an incident before and after AHS implementation.

second from one platoon to the next) the same cannot be said if one or more of the affected platoons are experiencing internal communications problems.

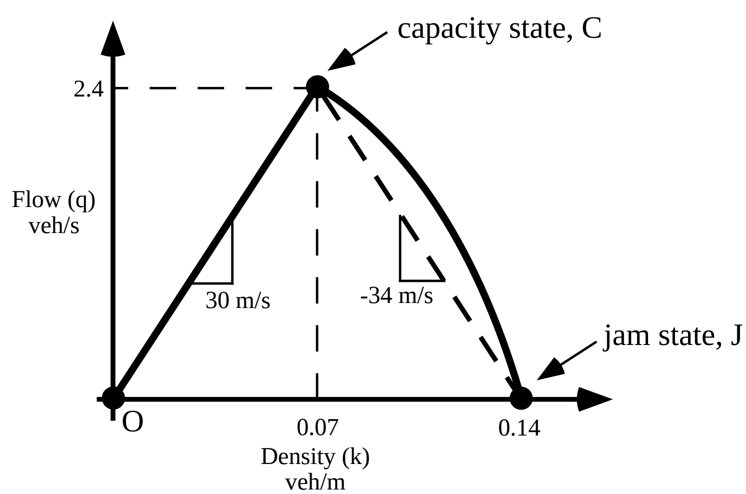

Figure 10: Flow-versus-density curve for a typical automated highway lane.

Another point worth noting 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 effects; see Daganzo, 1995, for a discussion) it may be necessary to close many of the on-ramps upstream of the bottleneck causing the queue, which would transfer much of the vehicle storage to the local street system. This illustrates the severe systemwide consequences of local disturbances in an AHS. Further research into dynamic ramp control strategies to avoid, manage, and recover from disturbances should be given some priority.

5 CONCLUSIONS AND FUTURE RESEARCH

This research has shown that estimates of AHS capacity that prevail in the existing literature are optimistic, and are likely being used in comparison with understood capacities of conventional freeways in an invalid manner. 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 a system for judiciously scheduling vehicles into platoons according to their destinations is considered.

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; improving the freeway system itself does nothing to solve the congestion problem. Existing conventional freeways have the ability to store many vehicles in queues during the rush periods. While these queues are certainly frustrating to drivers, this is nonetheless an important function of freeways, as these queues would be stored all over the local street system otherwise. Because of the reduction in effective lane-mileage available for storage under AHS, this critical function of the freeways will be impaired.

Because of the above (and perhaps other) reasons, there may only be certain "boutique" locations for which AHS implementation is a realistic solution to a congestion problem. One should certainly avoid constructing AHS systems where downstream termini would be likely to "back up" onto the AHS. Therefore, closed loop systems such as circumferential ring roads seem likely candidates for this type of technology. There are very long freeways (such as Interstate 5 through the western United States) where no such termini exist (at least in rural areas); however these freeways are rarely congested, so the benefits of AHS improvements would be slight.

The results presented in this paper are mainly intended to provide insights into some of the important issues surrounding automated highway systems and their implementation. Some of the analysis could, and should, benefit from a better understanding of the precise mechanisms and algorithms planned for an AHS. In addition, the results should be extended to include more circumstances under which an AHS may operate, including a consideration of economic factors and other possible designs.

Because this work is part of a greater research effort towards analyzing hybrid freeway systems, there are other issues which have been identified, but for which significant results have not yet been published. A brief description of some of the most important issues follows.

Little research work has been published pertaining to the likely operation of an AHS under faulty conditions, in which at least one vehicle ceases to function as expected. History is ripe with examples of serious accidents as a result of supposedly foolproof technology gone awry, and there is no reason to think this phenomenon will not repeat itself with an AHS. A careful understanding needs to be developed of the possible ways in which an AHS could fail, the possible ways in which this failure could be detected, the possible ways in which the failure could be mitigated, and the likely damages should this process fail.

We are aware that the current technology makes it possible to handle huge amounts of information with extraordinary reliability. However, we are specially concerned about a problem that has not been addressed in any previous work: the necessity of checking the condition of the automated vehicles prior to their admission to the AHS. What the system cannot guarantee is the absence of any mechanical or electrical failures of vehicles in platoons. Such failures might lead to disasterous results under extreme (but not unexpected) operating conditions, such as deceleration under the influence of a shockwave.

One could imagine an implementation of an AHS where prior to admission to the automated lanes, vehicles are required to pass a safety inspection station, to ensure that their steering, tires, brakes, etc. are all in working order, that the vehicle has sufficient fuel to reach its destination, and so on. It may be possible to combine the functions of this safety inspection station with the "scheduling" system described in Section 2.3.

While simulation is a valid and (relatively) inexpensive method for testing potential AHS situations, it is simply not capable of representing all possible conditions, particularly those where very unlikely and dangerous incidents may occur. For this type of research, there is no substitute for real-life empirical data. Of course, this data is expensive to gather, for determining the results of a collision (for example) requires that a collision be generated, and costly equipment damaged in the process. This cost should pale in comparison, however, to the cost (both monetary and otherwise) of implementing a system whose behavior under adverse conditions is not fully understood.

Finally, it is suspected that due to some of the reasons described above, and possibly others, that the full costs of AHS have not yet been accurately predicted. There are also constraints on their implementation that have not yet been fully considered. These facts combined present the need for a thorough economic study of the viability of these systems. The gains in capacity suggested by an AHS may or may not justify its pursuit as a solution to problems of congestion.

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A OFF-RAMP SPACING

This section is concerned with estimating the distance between off-ramps. Figure 11 shows a possible arrangement of the exit and entry ramps on an AHS. The distances are expressed in meters. The abbreviations AL, TL, and ML stand for automated lane, transition lane, and manual lane, respectively. The exit ramp should have a dedicated lane so that the exit flow does not interfere with the flow on the manual lanes. The entry to the automated lanes could be done from the manual lanes to avoid the cost of an elevated structure. However, this may cause a capacity reduction on the manual lanes due to the weaving flow. Varaiya (1994) suggests for safety reasons a gate length and an intergate distance of 150 m. The distance between the last entry gate to the AL and the entry gate from the TL to the ML should be about 250 m. This is sufficient to stop a vehicle with an initial velocity of 30 m/s and a constant deceleration of 2 $\mathrm{m/s^2}$. Such a braking distance is actually 225 m. On the transition from the ML to the AL, vehicles may be forced to come to rest before entering the AL. Thus, a distance of $2 \times 225 = 450$ m will be required from the entry to the TL to that of the AL. This distance, denoted on Figure 11 with an X, is required to accommodate the grade required by the elevated structure of the exit ramp. We can set $X \geq 150$ m. Then it is easy to see that the distance of $450 + X$ meters allows for the regrouping of the platoon on the AL to fill the gaps created by the exiting vehicles. This join manuever may be carried out by accelerating from v to v' and decelerating back to v the vehicles upstream of the gaps with a constant acceleration and deceleration, a. Then, the space required by the manuever would be:

$$s = \frac{(v' + v)(v' - v)}{a},$$

whereas the distance travelled by those vehicles relative to the vehicles dowstream of the gaps would be:

$$\Delta s = \frac{(v' - v)^2}{a}$$

Since $\Delta s = 2 \times 6$ m is the gap left by two exiting vehicles, and taking $v' = 32$ m/s, $v = 30$ m/s, we get $a = 0.33$ $\mathrm{m/s^2}$ and $s = 372$ m, considerably less than $450 + X$ meters.

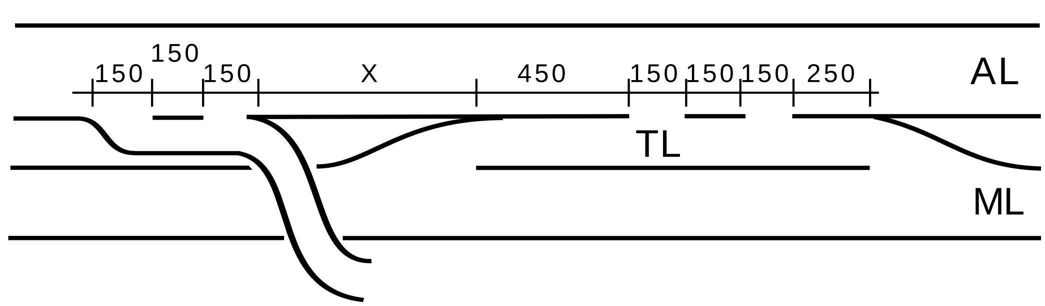

Figure 11: Exit and entry ramps of the automated lanes.

In conclusion, it seems that a realistic design of the exit and entry ramps with two gates would require about 1800 m. Hence, it does not seem reasonable to locate ramps at distances less than 3 km; then the mininum total length of each automated freeway section should be about 5 km. A 50 km long beltway could accommodate up to ten sections, and the average trip length measured in number of ramps would be less than 5, which implies $\mu > 0.2$. Thus, even in such a long beltway, the exit flow percentage would be rather high.