■ 論 文 ■

간격수락 이론을 이용한 다인승전용차로 진·출입을 위한 도로 디자인 지침정립

Design Guideline Development for Managed Lane Access Spacing Using Gap Acceptance Theory

양 철 수
(한국철도기술연구원 교통·물류연구실 박사후연수연구원)

Stephen P. Mattingly
(University of Texas at Arlington 토목공학과 교수)

김 현 웅
(한국철도기술연구원 교통·물류연구실 선임연구원)

권 용 장
(한국철도기술연구원 교통·물류연구실 책임연구원)

목 차

Key Words : 다인승전용차로, 위브(Weave), 간격수락, 고속도로, 교통용량
Managed lane, Weave, Gap acceptance, Freeway, Capacity

요 약

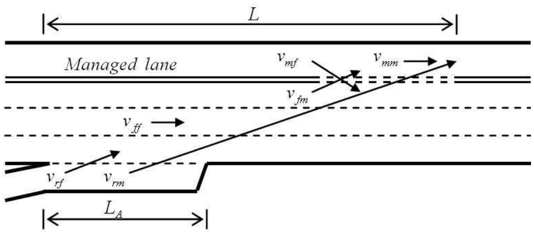
본 논문의 주요 목적은 도로디자인에 관련한 지침, 즉 고속도로 진·출입구간에서 도로중앙에 위치한 다인승전용차로 진·출입구간까지의 거리에 대한 지침을 마련하는 것이다. 다인승전용차로는 일반적으로 고속도로 중앙에 위치하고, 고속도로차로를 가로질러 위빙(weaving)하여 진입할 수 있다. 위빙구간에서의 많은 차로변경은 교통용량(capacity)에 심각한 영향을 끼친다. 위빙구간에서 이러한 차로변경의 영향을 분석하기 위해서는 간격수락이론이 유용하게 이용될 수 있다. 본 논문은 간격수락이론을 이용하여 산출된 차량의 상충의 정도를 기초로 하여 교통용량을 산출한다. 차량의 상충 정도는 주어진 위빙거리에서 차량의 위빙에 대한 성공 확률의 함수로 표현된다. 간격수락이론에 기초하여 위빙거리에 대한 지침을 마련함에 있어서, 이 논문은 위빙거리의 증가가 교통용량의 증가를 유발하지 않는 점을 위빙에 최소로 요구되는 거리로 정한다.

The principal objective of this paper is to develop road design guidelines, especially for managed lane access spacing between the expressway on-ramp (or off-ramp) and managed lane access point. Managed lanes are typically located in the expressway median and are accessed by weaving across the mainlines. The high level of lane-changing activity present in weaving areas affects capacity significantly. One promising tool for the analysis of lane-changing activity is “gap acceptance theory.” This paper estimates the capacity of weaving areas based on the estimated degree of traffic turbulence using gap acceptance theory. The degree of traffic turbulence is represented by a function of the probability that lane-changing vehicles can complete their maneuvers successfully in a given weaving distance. In developing road design guidelines based on the developed gap acceptance model, the minimum managed lane access spacing is determined where the capacity with respect to the managed lane access spacing becomes stable.

I. Introduction

Traffic growth in urban areas continues to rise, due in part to rapid population growth. This traffic growth results in worsening congestion on urban freeways that often cannot be addressed in a timely manner by widening existing facilities or the construction of new facilities due to funding limitations or the often-extensive time required for environmental clearances and actual construction. In some cases, the public may not support new freeway construction or expansion. The need for new roads to address this congestion exceeds not only the funding capacity but also the ability to gain environmental and public approval for large-scale construction projects.

One strategy for improving freeway performance is through the implementation of managed lane facilities. Managed lanes provide a good opportunity to increase capacity and improve operations of our urban freeways at a much lower cost than simply providing equivalent capacity with only general purpose lanes. Managed lanes are typically found adjacent to the freeway median and are accessed directly from frontage roads, local arterial streets, other managed lane facilities, or park-and-ride lots with grade-separated ramps or accessed by weaving across the general purpose lanes and entering them from the left lane. The second option is often preferred from a cost standpoint, but requires managed lane users to weave across the general-purpose lanes. In these cases, intense lane-changing maneuvers may cause traffic turbulence, which induces special operational problems that affect the freeway capacity and level of service.



L : managed lane access spacing
L_A : length of the auxiliary lane
v : total traffic volume in the weaving area
v_{ff} : traffic volume from the freeway to the freeway
v_{mm} : traffic volume from the managed lane to the managed lane
v_{fm} : traffic volume from the freeway to the managed lane
v_{rf} : traffic volume from the ramp to the freeway
v_{rm} : traffic volume from the ramp to the managed lane
v_{mf} : traffic volume from the managed lane to the freeway

<Figure 1> Lane configuration and traffic movements

The typical lane configuration of freeway weaving areas with managed lanes where traffic maneuvers weave from the on-ramp to the managed lane entrance is shown in <figure 1>. The traffic movements can be decomposed as shown in <figure 1>. Similarly, the traffic leaving a managed lane to weave across the mainlanes to an off-ramp can be modeled in the same fashion. In this case, <figure 1> is reversed.

One of design issues introduced in <figure 1> is the managed lane access spacing (L) between the beginning of the on-ramp and the endpoint of managed lane access (or between the managed lane access point and the off-ramp in the reverse case). Relevant guidelines in managed lane design manuals are listed below:

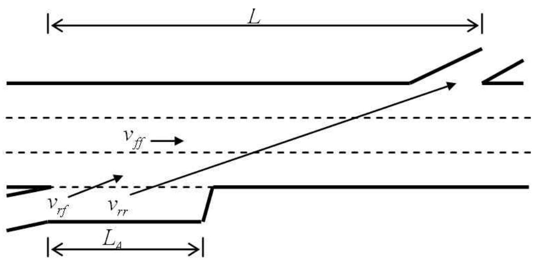
- the minimum managed lane access spacing (L) is 150m per lane (Caltrans, 1991),
- the minimum managed lane access spacing (L) is 150m per lane, and the desired one is 300m per lane (Fuhs, 1990), and
- the suggested managed lane access spacing (L) is 750m (Turnbull and Capelle, 1998).

The managed lane access spacing guidance in the managed lane manuals is most likely based on operational experience. Logically, it should be determined according to the free-flow speed of freeways, with more spacing required when it is higher.

This paper investigates the impact of the managed lane access spacing on the capacity of weaving areas. To do so, gap acceptance modeling is chosen because it can model the high level of traffic turbulence caused by lane-changing maneuvers. Once capacity with respect to weaving distance is estimated, the minimum managed lane access spacing is determined where the capacity with respect to the managed lane access spacing becomes stable.

II. Methodology and Model Development

The Highway Capacity Manual (HCM) (2000) concept can possibly be applied to the analysis of freeway weaving areas with managed lanes. Traffic weaving from an on-ramp across the mainlanes to the managed lane access can be modeled as a two-sided Type C weave. This process is shown in <figure 2>, where the off-ramp from the freeway mainlanes is the connection to the managed lane. Similarly, the traffic leaving a managed lane to weave across the mainlanes to an off-ramp can be modeled in the same fashion. In this case, <figure 2> can be reversed. <Figure 2> shows the critical movements in a weaving area between the on-ramp and the off-ramp (known as the managed lane access point).



L : spacing between the ramp and managed lane access point
L_A : length of the auxiliary lane
v : total traffic volume in the weaving area
v_{ff} : traffic volume from the freeway to the freeway
v_{rr} : traffic volume from the ramp to the ramp
v_{rf} : traffic volume from the ramp to the freeway

〈Figure 2〉 Lane configuration

Two general procedures are required to evaluate the impact of two-sided Type C weaving areas on the freeway mainlanes. The first is for weaving area analysis, and the second is for ramp junction analysis. The distinction between weaving areas and ramp junctions is strictly due to the lane geometry at the ramps. The HCM 2000 defines weaving as "the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway without the aid of traffic control devices (with the exception of guide signs)." In addition, weaving areas are formed "when a merge area is closely followed by a diverge area, or when an on-ramp is closely followed by an off-ramp and the two are joined by an auxiliary lane." If no lanes are added to or dropped from the freeway mainlanes in a series of consecutive ramps, they are analyzed as ramp junctions.

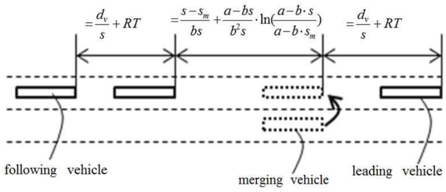
In the case of two-sided Type C weaving areas, both the weaving area and ramp junction analyses should be conducted separately. The capacity of ramp junctions is affected by the traffic turbulence caused by the conflict of the oncoming traffic volumes from the upstream freeway and the ramp, while the capacity of two-sided Type C weaving sections is determined by the degree of traffic turbulence between the ramp-to-ramp traffic volume and the through traffic volume.

Weaving areas and ramp junctions should be designed to preserve freeway capacity and level of service. A high level of lane changes occurs around them. One promising tool for the analysis of weaving areas and ramp junctions is "gap acceptance theory" which can models the decision-making procedures of driver's lane-change behaviors. This paper determines capacity based on the estimated degree of traffic turbulence in weaving areas or ramp junction using gap acceptance theory. The degree of traffic turbulence is represented by a function of the probability that lane-changing vehicles can complete their maneuvers successfully in a given distance.

Gap acceptance theory-based capacity models for Types A and B could be found in the literature (Lertworawanich and Elefteriadou, 2001, 2003). Lertworawanich and Elefteriadou (2001) developed a capacity model for Type B weaves by using linear optimization and gap acceptance theory. The optimiztion tool enabled them to estimate the capacity of weaving sections by systematically choosing the values of various demands with some constraints. Lertworawanich and Elefteriadou (2003) extended the Type B weaving methodology to estimate capacity for Type A weaves. In addition, gap acceptance theory-based models can be found in the literature of 김경환(1986) and 장정아(2008).

1. Ideal Safe Gap Estimation

“The ideal safe gap” is described as a time interval between successive arrivals of vehicles traveling in the same lane that would not cause a merging vehicle to collide with leading and following vehicles. In this research, the ideal safe gap estimation is based on the equations of vehicular motion proposed by Drew et al. (1967). The ideal safe gap for merging was developed on the basis of the time required for safe time headways between the merging vehicle and the leading and following vehicles and the time lost due to acceleration during the merging maneuver for merging vehicles traveling slower than or at the same speed as vehicles in a merged traffic stream. In order to avoid colliding with vehicles in the merged traffic stream, the merging vehicle requires the time interval given by the following (see <figure 3>):



<Figure 3> Graphical representation of equation 1

$$T = 2\frac{d_v}{s} + 2RT + \frac{s-s_m}{bs} + \frac{a-bs}{b^2 s}\cdot\ln(\frac{a-b\cdot s}{a-b\cdot s_m}) \quad (1)$$

where $s \geq s_m$,
- s : speed of merged vehicles,
- s_m : speed of merging vehicles,
- d_v : length of vehicles,
- RT : driver reaction time, and
- a and b: constants.

The ideal safe gap T can also be defined as a negligible risk gap. The ideal safe gap T is equivalent to the critical gap value where the accepted and rejected percentage is equal. Knox (1964) calibrated the parameters of a and a/b. The suggested values for a and a/b are a=7.9 kmph/sec and a/b=128 kmph, respectively. The “perception-reaction time” typically used in transportation design includes three elements: detection, identification, and reaction. The third element, reaction time (RT) is a response to an expected situation. The reaction time (RT) includes only the reaction element, which is different from the perception-reaction time. <Table 1> shows the ideal safe gap T for merging with a driver reaction time (RT) = 0.3 sec and vehicle length (d_v) = 4.5m.

2. Estimation of Time Required for Changing Lanes

<Table 1> Ideal Safe Gap

Ideal safe gap T (sec)		Speed of merged vehicles s (km/hr)						
		60	70	80	90	100	110	120
Speed of merging vehicles s_m (km/hr)	60	1.1	1.2	1.4	1.9	2.6	3.5	4.7
	70	-	1.0	1.1	1.4	2.0	2.8	3.8
	80	-	-	1.0	1.1	1.4	2.1	3.0
	90	-	-	-	1.0	1.1	1.5	2.3
	100	-	-	-	-	0.9	1.1	1.6
	110	-	-	-	-	-	0.9	1.1
	120	-	-	-	-	-	-	0.9

Kremser (1962a, b) estimated the service time that a vehicle spends in the first position at the stop line. Given the Poisson process for the arrival of vehicles on the main road at an intersection, all gaps are independently exponentially distributed with the expected headway of the main road 1/q where q is the flow. Let $w_s(q)$ be the waiting time at an intersection until the first gap t is greater than or equal to the ideal safe gap T. Kremser (1962a, b) suggested a model to estimate the expected waiting time of a vehicle in the first position near the stop line (the head of a queue) as

$$E[w_s(q)] = \frac{1}{q}\left[e^{T\cdot q} - (1 + T\cdot q)\right] \qquad (2)$$

The authors suggest that equation 2 can possibly be applied to parallel lane-change. In lane change theory, where the merging vehicle moves, the expected time that the merging vehicle waits for the first gap t that satisfies $t \geq T$ is transformed to

$$E[w(q)] = \frac{E[w_s(q)]\cdot s}{s - s_m} \qquad (s \geq s_m) \qquad (3)$$

where E[w(q)] is the expected waiting time of vehicles for merging, s and s_m are the speed of merged vehicles and the speed of merging vehicles, respectively, and q is the flow of the merged traffic stream. Combining equations 2 and 3 yields the expected waiting time for a lane change as

$$E[w(q)] = \frac{\frac{1}{q}\cdot\left[e^{T\cdot q} - (1 + T\cdot q)\right]\cdot s}{s - s_m} \qquad (s \geq s_m) \qquad (4)$$

3. Optimization of Merging Stream Speed

The key point of capacity estimation based on gap acceptance theory is how to estimate the speed of merging vehicles. As the speed difference $(s-s_m)$ between the merging and merged vehicles increases, the merging vehicle confronts more gaps in merged traffic stream during a given travel distance; however, the ideal safe gap T also increases. The natural assumption is that given the flow (q) and speed (s) of the merged stream, the speed (s_m) of the merging stream is determined to minimize the time required for a lane change as given below:

$$\text{Minimized} \qquad z(s_m) = E[w(q)] \qquad (5)$$

Let the minimized $z(s_m)$ be represented by min E[w(q)]. Once the speed of merging vehicles s_m is determined to minimize the expected waiting time as min E[w(q)], the distance traveled by a merging vehicle during a lane change is sm · min E[w(q)].

4. Capacity Estimation

For two-sided Type C weaves, more than one lane change is required to complete the weave. In probability theory, the Poisson distribution is a discrete probability distribution that expresses the probability of a number of events occurring during a specified period, if these events are independent of the time since the last event. The Poisson probability density function (pdf) that k number of events takes place is then given as

$$po[k] = \frac{\lambda^k \cdot e^{-\lambda}}{k!} \qquad k = 0, 1, 2, 3, \ldots \qquad (6)$$

where λ is the expected number of occurrences during a given interval. The formula for the cumulative Poisson pdf up to (N-1) occurrences is

$$cpo[N-1] = \sum_{k=0}^{N-1} \frac{\lambda^k \cdot e^{-\lambda}}{k!} \qquad k = 0, 1, 2, 3, \ldots \qquad (7)$$

In lane change theory, the expected number of lane changes that occurs within weaving distance L is

$$\lambda = \frac{L}{s_m \cdot \min E[w(q)]} \tag{8}$$

Then, the probability that at least an N number of lane changes occurs within weaving distance L is

$$P[N, L] = 1 - \sum_{k=0}^{N-1} \frac{\left\{\frac{L}{s_m \cdot \min E[w(q)]}\right\}^k \cdot e^{-\frac{L}{s_m \cdot \min E[w(q)]}}}{k\,!} \tag{9}$$

where N is the required number of lane changes for weave. $P[N, L]$ represents the probability that a weaving vehicle with the size of ideal safe gap T completes its weave successfully. Then, the mean number of vehicles assumed to fail their weaves from the on-ramp to the off-ramp is $v_{rr} \cdot (1-P[N, L])$. Note that, in reality, weaving vehicles would not fail in the weave, because vehicles worried about failing their weaves would drive more aggressively. This means that these vehicles would reduce the size of their ideal safe gap T. This aggressive driving behavior causes traffic turbulence in weaving areas. As such, $v_{rr} \cdot (1-P[N, L])$ represents the degree of traffic turbulence in the weaving area A as shown in <figure 4>. The capacity of weaving areas is limited by the degree of traffic turbulence $v_{rr} \cdot (1-P[N, L])$.

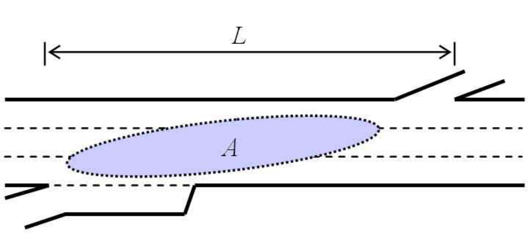
As the traffic volumes increase under a given volume ratio of the ramp-to-ramp volume to total volume in the weaving segment v_{rr}/v, the degree of traffic turbulence $v_{rr} \cdot (1-P[N, L])$ increases, resulting in more aggressive lane-changing behavior in the weaving area A. This driving behavior causing the traffic turbulence affects the capacity of weaving areas. Capacity estimation procedure is as follows. Given traffic demand volumes, compute the difference between the degree of traffic turbulence $v_{rr} \cdot (1-P[N, L])$ and the tolerance index of traffic turbulence δ as

$$\mathrm{Diff}(L) = v_{rr} \cdot (1 - P[N, L]) - \delta \tag{10}$$

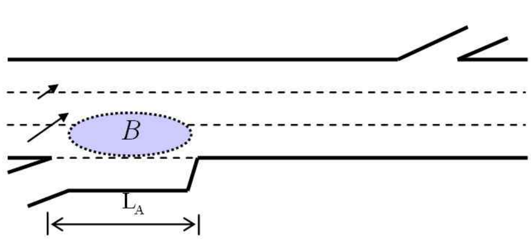
where the tolerance index of traffic turbulence represents the allowable degree of traffic turbulence in the weaving area that does not limit the capacity of weaving areas. If Diff(L) is less than zero, increase the traffic volumes maintaining the given volume ratio of the ramp-to-ramp volume to total volume v_{rr}/v. The capacity estimation process ends when Diff(L) is close enough to zero. The capacity of the weaving area with the given volume ratio v_{rr}/v is the total traffic volume (v) when $\mathrm{Diff}(L) \approx 0$.

Similarly, the capacity of a merge ramp terminal can also be obtained as follows. Let L_A be the length of acceleration lane as shown in <figure 5>. From equation 9, the probability that a merging vehicle can make one lane change from the on-ramp to the freeway's rightmost lane is

$$P[N = 1, L_A] = 1 - e^{-\frac{L_A}{s_m \cdot \min E[w(q)]}} \tag{11}$$



〈Figure 4〉 Areas of traffic turbulence



〈Figure 5〉 Areas of traffic turbulence

Then, the degree of traffic turbulence is described as $(v_{rr}+v_{rf})\cdot(1-P[1,L_A])$ The traffic turbulence around the on-ramp (area B) limits the traffic volumes from the ramp and upstream freeway to the downstream freeway as shown in <figure 5>. The arrows in <figure 5> represent the turning movement that avoids getting into the traffic turbulence area B.

The capacity of a ramp terminal is estimated as follows. Given traffic volumes, compute the difference between the degree of traffic turbulence $(v_{rr}+v_{rf})\cdot(1-P[1,L_A])$ and the tolerance index of traffic turbulence ρ(N) functioned by the number of freeway mainlanes N as

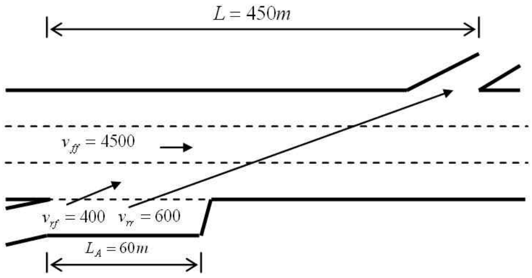
$$\mathrm{Diff}(L_A)=(v_{rr}+v_{rf})\cdot(1-P[1,L_A])-\rho(N) \tag{12}$$

If $\mathrm{Diff}(L_A)$ is less than zero, increase traffic volumes maintaining a given volume ratio of the ramp-to-freeway volume to total volume $(v_{rr}+v_{rf})/v$. The capacity estimation process is repeated until $\mathrm{Diff}(L_A)\approx 0$. The capacity of the ramp terminal with the given volume ratio is the total traffic volume (v) when $\mathrm{Diff}(L_A)\approx 0$. The tolerance index of the traffic turbulence ρ(N) plays two important roles. One is to describe the allowable traffic turbulence that does not limit the capacity of ramp junction. The other is to adjust the capacity increase due to the turning movement, which avoids getting into the traffic turbulence area, allows more traffic from the ramp and the upstream freeway to the downstream freeway.

III. Application of the Developed Methodology

This section provides an application of the developed methodology to estimate the capacities of a weaving area and ramp terminal, respectively, as shown in <figure 6>. Other related information is shown below.

The following are a step-by-step application of the developed methodology. The time that a



- speed of merged vehicles (s) = 80 km/hr
- driver reaction time (RT)= 0.3 sec
- length of vehicles (d_v) = 4.5 m
- δ= 65 pc/hr and ρ(N=3) = 600 pc/hr

<Figure 6> Type C weaving area

merging vehicle waits for the first gap t that satisfies t≥T is

$$E[w(1833)]=\frac{(3600/1833)\cdot\left[e^{T\cdot(3600/1833)}-(1+T\cdot(3600/1833))\right]\cdot s_m}{|80-s_m|}$$

Given the speed (s=80 mi/hr) and traffic flow (q=1833pc/hr/ln) of merged stream, the speed of the merging stream is optimized to minimize the time E[w(1833)] required for making a lane change. <Figure 7> shows that the optimized speed of merging stream is 67 km/hr. Then, Diff(450) is obtained as

$$\begin{aligned}\mathrm{Dff}(450)&=600\cdot(1-P[3,\ 450]-65\\&=600\cdot(1-0.977)-65=-51.2\ \ \mathrm{pc/hr}\end{aligned}$$

Diff(450) is less than zero. Thus, the traffic volumes must be reduced until Diff(450)≈0, maintaining the given volume ratio (v_{rr}/v=0.109) Finally, the capacity comes up with 6878 pc/hr.

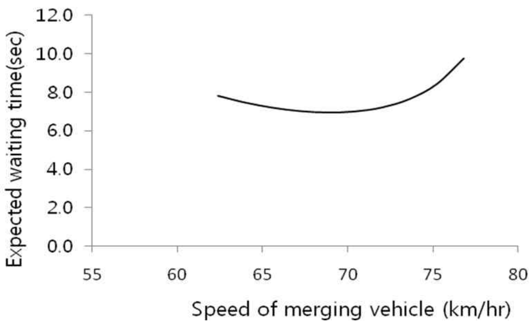
<Figure 8> shows the distribution of cumulative arrivals at the leftmost freeway lane under the capacity of 6878 pc/hr. Note that the distribution of cumulative arrivals at L=450m does not reach 100%. The residual percent represents the portion of vehicles driving aggressively to complete their weaves, which limits the capacity of the weaving area. In addition, the capacity of the ramp

terminal comes up with 6883 pc/hr (the process is not shown in this paper). The overall capacity of the subject area is 6878 pc/hr.

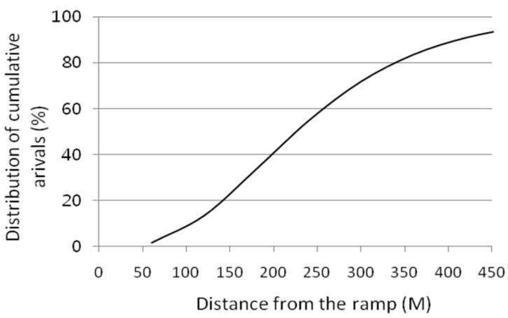
IV. Capacity Estimation and Key Findings

<Table 2> shows tabulated capacity values of the weaving segment with three freeway mainlanes (N=3) for various volume ratios and weaving distances (L). The following capacity is any combination of flows that causes the speed of merged vehicles (or the speed of all vehicles in the weaving segment) to reach 80 km/hr. The constraint employed in the capacity estimation is that the capacity of weaving segments cannot exceed the capacity of the equivalent basic free-way segment (2300 pc/hr/ln).

The length of weaving sections has a significant impact on capacity. Capacity with respect to weaving length stabilizes quickly when the ratio



〈Figure 7〉 Optimized speed of merging stream



〈Figure 8〉 Distribution of cumulative arrivals at the leftmost lane

<Table 2> Capacity of Two-sided Type C Weaving Areas with Three Freeway Mainlanes

Volume ratio v_{rr}/v	L (m)					
	300	360	420	480	540	600
0.1	5291	5963	6588	6900	6900	6900
0.2	4687	5317	5906	6460	6900	6900
0.3	4398	5006	5574	6111	6618	#
0.4	4215	4807	#	#	#	#

represents that the acceptable flow rate from the ramp to the freeway exceeds the ramp capacity of 2000 pc/hr.

<Table 3> Capacity of Ramp Junction with Three Freeway Mainlanes (pc/hr/ln)

Volume ratio $(v_{rr}+v_{rf})/v$	LA (ft)			
	60	75	90	105
0.1	6900	6900	6900	6900
0.2	6564	6900	6900	6900
0.3	5421	5998	6528	#
0.4	4787	#	#	#

represents that the acceptable flow rate from the ramp to the freeway exceeds the ramp capacity of 2000 pc/hr.

of the ramp-to-ramp volume to total volume v_{rr}/v is low. As the volume ratio v_{rr}/v increases, the capacity stabilizes at longer weaving distances.

<Table 3> shows the tabulated capacity values of ramp junction with three freeway mainlanes (N=3) for various volume ratios and acceleration lane lengths. The capacity of ramp junction is less than that of basic freeway segments, only when the ratio of the ramp-to-freeway volume to total volume $(v_{rr}+v_{rf})/v$ is high and the length of the acceleration lane is relatively short.

V. Recommendations and Future Research

The minimum managed lane access spacing (L) is determined where the capacity stabilizes. Regarding the modeling scenario of the volume ratio v_{rr}/v, note that Fuhs (1990) recommends indirect access ramps to serve relatively low weaving volume from the ramp to the managed

<Table 4> Minimum of the Managed Lane Access Spacing

Number of freeway mainlanes (N)	3	4	5
Minimum managed lane access spacing, L(m)	500	650	800

lane less than 400 veh/hr (maximum of 500 veh/hr), and Fitzpatrick et al. (2003) recommends indirect access ramps when it is less than 400 veh/hr and 275 veh/hr in a more conservative situation. The National Research Council's HOV Systems Manual (Turnbull and Capelle, 1998) states that HOV lanes are viable when the HOV-eligible traffic ranges from 400 to 600 veh/hr, and the maximum vehicle carrying capacity of managed lanes may range from 1,200 to 1,800 veh/hr/ln. This suggests that, in the modeling scenario, the flow rate weaving from the ramp to the managed lane would not be high. In this paper, the modeling scenario of the volume ratio $v_{rr}/v = 0.1$ is chosen. If so, the capacity with respect to the managed lane access spacing is stabilized as shown in <table 4>.

Even in the case of having an intermediate ramp between the ramp and the managed lane access point where a Type A ramp weave is formed, no additional weaving distance is added to the minimum managed lane access spacing as shown in <table 4>. The logic behind this recommendation is based on a recent capacity model for Type A ramp weaves of Denney and Williams (2005). Key findings from field data of four sites indicate that weaving vehicles make their lane changes very early in the weaving area, especially under capacity conditions. The vast majority of the lane changes occurred within the first 150m of the weaving area. This implies that drivers want to get into an objective lane as soon as possible under heavy traffic conditions. In the minimum managed lane access spacing in <table 4>, more than 150m per lane of weaving distance is added.

The capacity results and suggested guidelines are developed based on the developed analytical model using gap acceptance theory. As a future Research, the validation of the developed model with field data is recommended, especially the reaction time (RT) and related result of <table 3>.

References

1. Denney, R.W. Jr., Williams, J.C.(2005), Capacity and Quality of Service of Weaving Zones, Final Report, NCHRP Project 3-55(5).
2. Department of Transportation(Caltrans) (1991), High Occupancy Vehicle (HOV) Guidelines for Planning, Design, and Operations. Division of Traffic Operations, Business, Transportation and Housing Agency, State of California.
3. Drew, D.R., Lamotte, L.R., Buhr, J.H.(1967), Wattleworth, J.A. Gap acceptance in the freeway merging process. HRR 208, pp.1~36.
4. Fuhs, C.A.(1990), High Occupancy Vehicle Facilities: A Planning, Design, and Operation Manual. Parsons Brinckerhoff, Quade and Douglas, Incorporated, New York, NY.
5. Highway Capacity Manual(2000), Transportation Research Board, National Research Council, Washington, D.C.
6. Knox, D.W.(1964), Merging and weaving operations in traffic. Australian Road Research, Vol. 2, pp.10~20.
7. Kremser, H.(1962), Ein einfaches Wartezeitproblem bei einem Poissonschen verkehrsstroemen (A simple problem of delay with Poisson traffic flow). Oesterreichisches Ingenieur Archiv (Austrian Engineering Archive), Vol. 16.
8. Kremser, H.(1962), Ein zusammengesetztes Wartezeitproblem bei Poissonschen verkehrsstroemen (A complex problem of delay with Poisson traffic flow). Oesterreichisches Ingenieur Archiv (Austrian Engineering Archive), Vol. 16.
9. Fitzpatrick, K., Brewer, M.A., Venglar, S.P.

(2003), Managed Lane Ramp and Roadway Design Issues. Texas Transportation Institute, The Texas A&M University System.
10. Lertworawanich P., Elefteriadou L.(2001), Capacity Estimations for Type B Weaving Areas Using Gap Acceptance. Transportation Research Record 1776, TRB, National Research Council, Washington, D.C., pp.24~34.
11. Lertworawanich P., Elefteriadou L.(2003), A methodology for estimating capacity at ramp weaves based on gap acceptance and linear optimization. Transportation Research B, Vol. 37, pp.459~483.
12. Turnbull, K. and Capelle, D.(1998), HOV Systems Manual. National Cooperative Highway Research Program, Transportation Research Board, National Research Council.
13. 김경환(1986), The gap-acceptance of left-turn drivers, Journal of Korean Society of Transportation, Vol. 4, No. 1, Korean Society of Transportation, pp.72~87.
14. 장정아·이정우·최기주(2008), A Gap-acceptance Model Considering Driver's Propensity at Uncontrolled Intersection, Journal of Korean Society of Transportation, Vol. 26, No. 6, Korean Society of Transportation, pp.71~80.

♧ 주 작 성 자 : 양철수
♧ 교 신 저 자 : 양철수
♧ 논문투고일 : 2010. 5. 31
♧ 논문심사일 : 2010. 7. 1 (1차)
2010. 7. 14 (2차)
♧ 심사판정일 : 2010. 7. 14
♧ 반론접수기한 : 2010. 12. 31
♧ 3인 익명 심사필
♧ 1인 abstract 교정필