peak conditions (see *Network Flow Simulation for Urban Traffic Control System - Phase II*, Volume 1. Technical Report, by Peat, Marwick, Mitchell and Company, March, 1974; this still constitutes the base calibration for the model, per discussion with Henry Lieu). In practice few, if any, users have the ability to modify embedded distributions due to the amount and cost of required data collection. As a result, users effectively are modelling traffic using Washington, D.C. driver attributes. If the underlying distributions are not appropriate to the study area, the ability to simulate detailed vehicle interactions is questionable. In that case, model results are no more valid than those from macroscopic models that rely essentially on mean variable values instead of statistical distributions.

Related to the above, the processes and results of simulation models are stochastic in nature. This means that between different model runs, different answers can be achieved. Furthermore, as volume-to-capacity ratios at specific intersections or nodes approach 1.0—often the locations of greatest concern—the result variability from run to run increases to a maximum. For statistical validity then, it is necessary to conduct replications using different random number seeds. It is important that the user be aware of this fact, and be prepared to make multiple runs (see "Variability Assessment for TRAF-NETSIM", by Gang-Len Chang and Ammar Kanaan for recent discussion of this issue; ASCE *Journal of Transportation Engineering*, Volume 116, No. 5, Sept./Oct. 1990, pp. 636-657). Since runs of a microscopic model can require significant computer time, the user must budget time and money resources accordingly.

Another point is more practical. As noted in the paper, none of the microscopic models cited have signal timing (or other) optimization capabilities built in. As is apparent from use of any of the optimization models, interchange performance is very dependent on good signal timing. Thus to use effectively a microscopic model to evaluate alternate interchange forms, the user will typically need to run at least one of the optimization models to generate signal timings. For fair comparison between different interchange forms, the same optimization model should be used, yet some optimization models (e.g., PASSER III) are applicable to only one interchange form. The results from such an evaluation often are only as good as the timings produced by the optimized model(s), and even then the optimized timings may need to be "tweaked" to achieve reasonable results.

In broad perspective, microscopic models can be useful for investigating and understanding detailed traffic interactions—some users indicate that a simulation of existing conditions alerts them to conditions or phenomena they did not (or could not) observe in the field initially, but then do field verify. Testing of unusual traffic control or geometric features, as noted in the paper, can be first undertaken through simulation to avoid risky situations, or to eliminate the need to build an expensive facility. Because of the complexity of the models, however, there are theoretical and practical considerations of which the potential user must be aware. Not to take account of such considerations leaves the user in jeopardy of the basic rule of computing—"garbage in, garbage out."

A final point deals with potential use of the Highway Capacity Software (HCS) for weaving analysis on arterials. The entire topic of merge/diverge/weaving on freeways is under review currently as a part of NCHRP Project 3-37. Current procedures have been questioned in some respect and likely are to be updated in the near future; this suggests that the use of current freeway procedures to approximate arterial conditions should be undertaken with a good deal of caution.

REVIEW OF DIAMOND INTERCHANGE ANALYSIS TECHNIQUES: PAST AND PRESENT

Jim C. Lee, Lee Engineering, Inc.

Introduction

The diamond interchange interface with arterial streets has long presented formidable challenges for the traffic engineer. Especially in urban areas, it usually results in two closely spaced signalized intersections, often in close proximity to other signalized intersections. Urban freeways often act as traffic generators themselves, which cause some of the highest volumes on arterial streets near the ramp or frontage road terminal. Additionally, the fact that there are typically few streets on which to cross from one side of a freeway to another further concentrates traffic on the arterial street. For these reasons, diamond interchanges often dictate the capacity of the entire arterial street.

These closely spaced, signalized intersections associated with diamond interchanges also offer some operational problems. The method of timing the signalized intersections has been the subject of considerable research and discussion. With the importance of these signalized diamond interchanges on the arterial streets, it is surprising that we have not developed better analytical techniques to predict their capacity. Of particular concern is the prevalent practice of treating the two signalized intersections independently in the capacity calculations. Upon reflection, however, it is not surprising that the analysts would resort to this method when we examine the techniques available.

The purpose of this paper is to present and compare various methods to estimate capacity and evaluate performance at diamond interchanges. The focus here is on practical, day-to-day approaches to the problem, for use mainly in planning applications. A few different methodologies are discussed.

A Historical View of Diamond Interchange Capacity

With the exception of computer simulation models, our ability to predict diamond interchange capacity at a planning level has progressed little in the last thirty years. Capelle and Pinnell wrote in the early 1960's (*1*, p.15):

> "After studying the problem of evaluating the capacity of diamond interchanges, it was determined that it would be necessary to consider the two signalized intersections as a single unit. This is due primarily to the requirements of signalization which should perform two basic functions. These functions are as follows: (a) all high-volume conflicting movements at both intersections must be separated, and (b) storing of vehicles between the two intersections must be kept to a minimum due to limited distance between the intersection."

Capelle and Pinnell selected a phasing plan that has since become known as a four phase with overlap operation to accomplish these objectives. They proposed a method of calculating what they termed the critical capacity N_H of a four phase with overlap diamond interchange as being:

$$N_H = \left(\frac{C+4-4D}{H} + 8 \right) \frac{3600}{C}$$

where C represents the cycle length, D the starting delay and H the headway. The starting delay used for their calculations was the time required for the first two vehicles in a lane to enter the intersection. The critical capacity, N_H, represents the maximum summation of the four critical lane approach volumes comprising the four external approaches to the interchange. Capelle and Pinnell computed critical capacity using values of $D=5.8$ and $H=2.1$ second.

Capelle/Pinnell Method Updated

Recent field studies by Hook (*2*) have provided measured saturation flow rates and lost times at conventional diamond interchanges. The Hook values for starting delay and average time-headway, weighted by the volumes of the movements in the example analysis are 7.1 and 1.89 seconds respectively. Capelle and Pinnell assumed that the starting delay was incurred by the first two vehicles, while the Hook research assumed it to be incurred by the first three vehicles. If the Capelle/Pinnell equation for critical capacity is modified for three vehicle starting delay and the Hook values substituted for the values of starting delay (5.2 seconds) and headway (1.9 seconds) used in the Capelle and Pinnell equation, updated values are obtained for critical capacity. Both the original Capelle/Pinnell critical capacity values and updated critical capacity values per current start-up/headway data are presented in Table 1.

It should be noted that the critical capacity is the summation of approach volumes to the interchange over a one-hour period assuming a uniform distribution of the traffic during that hour. Capelle and Pinnell accounted for this by increasing the actual approach volumes by 20 percent (*1*, p. 20):

> "In general, a 20 percent difference between expanded hourly demand and actual hourly demand was observed. Additional confidence in this figure was obtained from the 'Highway Capacity Manual' which stipulates a 20 percent difference between Practical (or Design Capacity) and Possible Capacity. Therefore it was determined that expected peak hourly volumes should be increased by 20 percent to obtain peak flow conditions for which Equation 5 would be applicable."

In effect, this adjustment procedure is equivalent to the procedure today of dividing the hourly volume by the peak hour factor to obtain the peak 15-minute flow rate.

Figure 1 is a depiction of the lane volumes used by Capelle and Pinnell in the example in their paper (*1*, p. 21). The sum of these four critical lane volumes (725, 225, 475, 450) is 1875 vehicles per hour. Capelle and Pinnell concluded this to be excessive and analyzed the problem with an additional lane on one approach in order to bring the sum of critical lane volumes under what they have proposed to be the maximum allowable.

TABLE 1 CRITICAL CAPACITY CALCULATIONS, CAPELLE/PINNELL METHOD

Cycle Length	Original Critical Capacity(1)	Updated Critical Capacity (2)
40	1,611	1,821
50	1,635	1,838
60	1,650	1,849
70	1,660	1,857
80	1,668	1,863
100	1,674	1,872
180	1,692	1,886

(1) Based on 1962 values for starting delay, D, of 5.8 seconds and average headway, H, of 2.1 seconds.

(2) Based on current values of D = 5.2 seconds and H = 1.9 seconds, per (2). D derived from Reference 2 start-up lost time measurement.

Current Analytical Methods

The most widely used methods for capacity analysis in this country today are the planning analysis and the operations analysis of the *Highway Capacity Manual* (HCM; *3*). Neither of these methods specifically addresses diamond interchange capacity analysis. Although many people familiar with capacity analysis recognize that it is inappropriate to analyze signalized diamond interchanges as two separate intersections, it is still widely used and generally accepted by many practicing traffic engineers as the only methods. Indeed, the HCM neither provides another method nor does it issue a caution that the user should not analyze the two sides of a diamond interchange as two separate signalized intersections.

Highway Capacity Manual Planning Analysis

An example of the practice of treating the interchange as two independent signalized intersections and the results obtained was picked from an actual planning method analysis. This analysis recently came to this author and was not selected as an unusual or special case. In fact, in the author's opinion, it is representative of methods widely used by practitioners today for capacity analysis of diamond interchanges. The lane volumes for the example analysis are shown in Figure 2. This analysis shows the intersections projected to operate UNDER or NEAR CAPACITY in the year 2010 in the PM peak hour. Predictably, this analysis satisfies both the person doing the analysis, as well as the person reviewing the traffic impact study, and concludes that the proposed land uses could be satisfactorily accommodated by the geometrics proposed for this diamond interchange.

Highway Capacity Manual Operational Analysis

This same example was next analyzed using the HCM operations analysis. Because detailed design and operational information were not available for this future interchange, default values were used for the analysis.

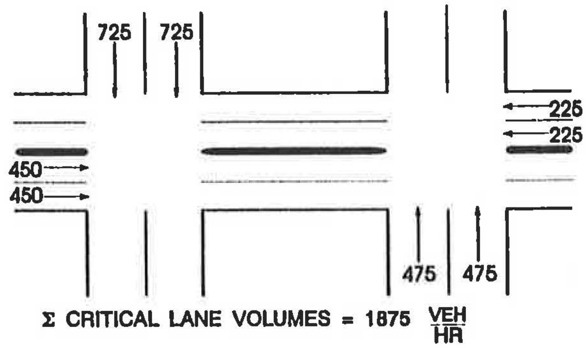

FIGURE 1 Capelle/Pinnell example critical lane volumes.

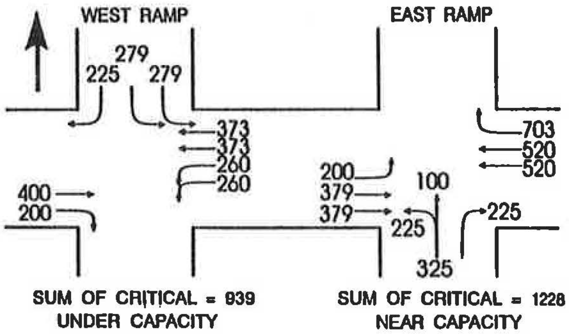

FIGURE 2 2010 projected pm peak hour volumes example analysis: planning method.

The same lane configuration proposed by the original analyst was analyzed using the operations analysis. The results of this analysis indicated that the west ramp will operate at level of service C (21 seconds of delay per vehicle) and the east ramp will operate at level of service B (13 seconds of delay per vehicle).

Church (Caltrans District 4) Method

Caltrans District 4 has developed a manual on intersection analysis (*4*) and is in the process of developing one on interchange analysis (*5*). This procedure first considers storage requirements between closely spaced intersections such as diamond interchange ramps, and then establishes signal timing to progress those movements with inadequate storage. The method used is intersection lane vehicles (ILV), which is the equivalent to the sum of critical volumes specified in the planning method in the 1985 HCM. The interchange area is treated as one operational unit with an assumed phasing, and the sum of ILV is computed for the entire interchange. In arriving at the ILV sum, the procedure considers the travel time between the signalized intersections by increasing the ILV sum by a penalty called an "equivalent ILV."

Church Example Method

In order to compare the results of the Church method with the planning and operations methods of the HCM, the same example previously analyzed with was analyzed with the Church approach. This analysis is for one possible phasing option that progresses the westbound through traffic because it is the movement which would present the worst storage problem. This phasing is not necessarily the best option, but is intended to demonstrate how the Church procedure would analyze this phasing. The Church method results in a prediction of Level of Service F (Figure 3). The numbers in parenthesis in Figure 3 indicate stopped vehicles.

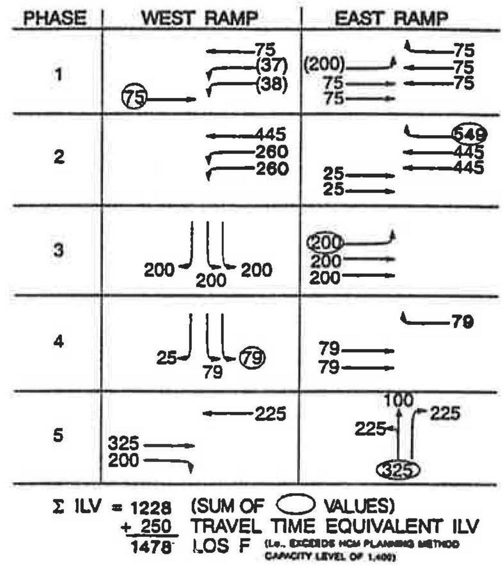

FIGURE 3 2010 projected pm peak hour volumes examples analysis: Church method.

Modified Planning Analysis

The author of this paper has used a modified version of the planning analysis of the HCM for the diamond interchange, assuming four phase with overlap timing. In this method, the sum of the critical movements includes the external approaches to the interchange and is reduced by the volume that can be accommodated in the overlap phases. Based on the volumes from the previous example, the sum of the critical movements of the four approaches to the interchange is 1707 (Figure 4). Assuming two 10 second overlaps, each of which can accommodate 3 vehicles per cycle, an additional 180 critical movements can be accommodated with a 120 second cycle. Reducing the 1707 sum of critical movements by 180 overlap vehicles results in a sum of critical movements of 1527, which falls in the OVER CAPACITY area per the HCM planning guidelines.

Modified Operational Analysis

A method has been developed by David Hook of Lee Engineering (*unpublished data*) which assumes a four phase with overlap diamond interchange operation and

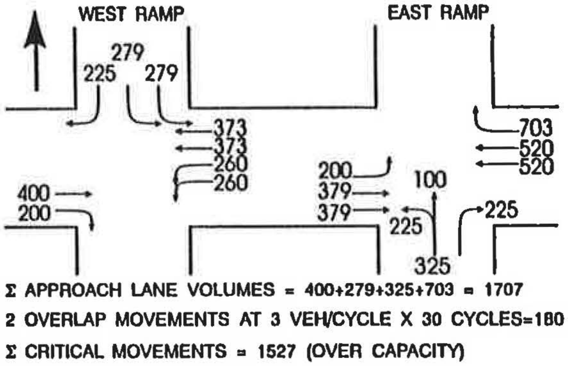

FIGURE 4 2010 projected pm peak hour volumes example analysis: Lee method.

assigns green time to each phase based on normal split calculations. The overlap time due to the early release of the two overlap phases provides for a cycle length less than the sum of the green time for the four phases. He has devised a spreadsheet which permits this calculation with the same equations used in the HCM operational analysis.

Using this procedure for the example analysis results in 118 seconds of delay per vehicle (LOS F) if the default values used in the operational analysis as two separate intersections are used (Table 2). Table 3, based on Figure 5 timings, indicates the results (46 seconds of delay per vehicle, or LOS E) if the estimate of saturation flow rates measured in the Phoenix area for diamond interchanges (*2*) is used in the analysis.

Capelle/Pinnell Method

Because the Capelle and Pinnell method was specifically derived for diamond interchanges (albeit four phase with overlap cases only), the same projected volumes of the example were analyzed with that methodology. This results in a sum of critical lane volumes as defined by Capelle and Pinnell of 2049 vehicles per hour as compared with the maximum capacity value of about 1650 for their original method (Figure 6) and parameters, assuming a mid-range value of critical capacity over all cycle lengths. If the critical sum is compared to the updated critical values of Table 1, the sum exceeds the mid-range critical capacity value of 1,850 by about 200, or about 11%.

Comparison of Analytical Techniques

The large discrepancies between a result of under capacity for the HCM methods widely used today, to

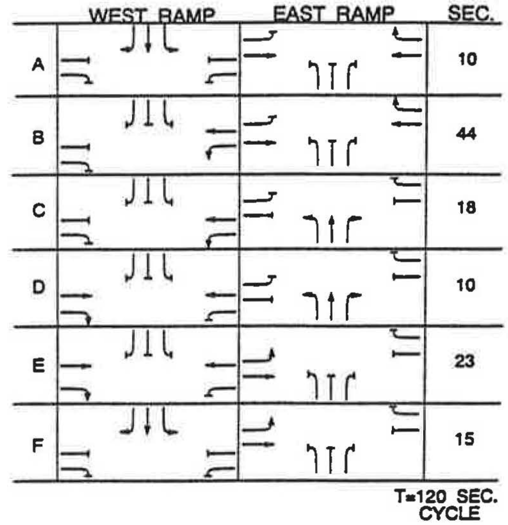

FIGURE 5 Four phase with overlap phasing used in hook modified operational analysis.

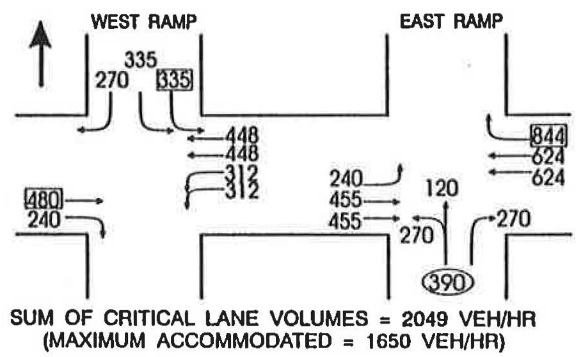

FIGURE 6 2010 projected pm peak hour volumes example analysis: Capelle and Pinnell method.

well over capacity for other methods should be of concern to those involved. One could dismiss the Capelle/Pinnell method as being out of date, however, to this practitioner it better fits the test of logic than the independent analysis frequently used today. The assumption of four phases with overlap phasing and a sum of critical movements of the four exterior approaches totals 2049 as shown in the Capelle and Pinnell approach. We can work backwards to an equivalent sum of critical volumes, for evaluation per the HCM planning guidelines. To do this, first we assume that during the overlap phase, the same 180 vehicles per hour used in the earlier example applies and reduce the 2049 by that amount, resulting in 1869 vehicles per hour as the critical sum. If we divide this figure by 1.2 to convert Capelle's design hourly flow to hourly flow, we

TABLE 2 HOOK MODIFIED OPERATIONAL ANALYSIS USING DEFAULT SATURATION FLOW RATES

EXAMPLE ANALYSIS
2010 PM PEAK

PHASE	TIME
A	10
B	44
C	18
D	10
E	20
F	18
CYCLE	120

MOVEMENT	VOL	LAN UTIL	PHF	ADJ VOLU	LAN	LANE VOL	GREEN	LOST TIME	GREEN, g	G/C	ADJ SFR	CA	V/C RA
NB LT	325	1	0.9	361	1	361	28	5.0	23.0	0.192	1515	290	1.244
NB THR	225	1	0.9	250	1	250	28	5.0	23.0	0.192	1515	290	0.861
SB LT	558	1	0.9	620	2	310	28	5.0	23.0	0.192	1337	256	1.210
SB TH	225	1	0.9	250	1	250	28	5.0	23.0	0.192	1515	290	0.861
EB TH	400	1	0.9	444	1	444	30	5.0	25.0	0.208	1782	371	1.197
EB RT	200	1	0.9	222	1	222	30	5.0	25.0	0.208	1515	316	0.704
WB TH	1040	1	0.9	1156	2	578	54	5.0	49.0	0.408	1782	728	0.794
WB RT	703	1	0.9	781	1	781	54	5.0	49.0	0.408	1515	619	1.263
							140	20.0	96.0				
							140						

MOVEMENT	FLOW RATIO	CRIT?	SUMV/	D1 DELA	LANE GRP CAP	D2 DELAY		TOTAL DELAY	PROG FACT	DELAY	APPROACH DELAY
NB LT	0.24	1	0.238	45.6	290	160.8	0.403	206.49	1.00	206.49	145
NB TH	0.17	0	0.000	41.6	290	15.3	0.397	56.94	1.00	56.94	
SB LT	0.23	0	0.000	45.3	513	125.5	0.397	170.73	1.00	170.73	138
SB TH	0.17	1	0.165	41.6	290	15.3	0.403	56.94	1.00	56.94	
EB LT	0.25	1	0.249	44.4	371	123.5	0.765	167.88	1.00	167.88	127
EB TH	0.15	0	0.000	39.1	316	4.7	0.471	43.80	1.00	43.80	
WB LT	0.32	0	0.000	27.6	1455	2.2	0.471	29.76	1.00	29.76	98
WB TH	0.52	1	0.516	38.4	619	160.4	0.765	198.83	1.00	198.83	

INTERCHANGE DEL 118 SECONDS LOS F

obtain a critical sum of 1558. This sum then falls in the OVER CAPACITY area of the HCM planning analysis.

The other methods presented (Church, Lee and Hook) attempt to consider the relationship of the two closely spaced signalized intersections while using the principles of ILV, HCM planning analysis and HCM operations analysis respectively.

Summary of Comparisons

The results of the various analyses of the same example are shown in Table 4. One must be concerned with the wide disparity of results, covering the gamut from good operation and little delay predicted by the widely used (if incorrectly) independent intersection method to capacity deficient operation predicted by other methods.

TABLE 3 HOOK MODIFIED OPERATIONAL ANALYSIS USING MEASURED SATURATION FLOW RATES

EXAMPLE ANALYSIS
2010 PM PEAK

PHASE	TIME
A	10
B	44
C	18
D	10
E	23
F	15
CYCLE	120

MOVEMENT	VOL	LAN UTIL	PHF	ADJ VOLU	LAN	LANE VOL	GREEN	LOST TIME	GREEN, g	G/C	ADJ SFR	CA	V/C RA
NB LT	325	1	1	325	1	325	28	5.0	23.0	0.192	1800	345	0.942
NB THR	225	1	1	225	1	225	28	5.0	23.0	0.192	1800	345	0.652
SB LT	558	1	1	558	2	279	25	5.0	20.0	0.167	1800	300	0.930
SB TH	225	1	1	225	1	225	25	5.0	20.0	0.167	1800	300	0.750
EB TH	400	1	1	400	1	400	33	5.0	28.0	0.233	1800	420	0.952
EB RT	200	1	1	200	1	200	33	5.0	28.0	0.233	1800	420	0.476
WB TH	1040	1	1	1040	2	520	54	5.0	49.0	0.408	1800	735	0.707
WB RT	703	1	1	703	1	703	54	5.0	49.0	0.408	1800	735	0.956
							140	20.0	102.0				
							140						

MOVEMENT	FLOW RATIO	CRIT?	SUMV/	D1 DELA	LANE GRP CAP	D2 DELAY		TOTAL DELAY	PROG FACT	DELAY	APPROACH DELAY
NB LT	0.18	1	0.181	42.4	345	24.4	0.306	66.82	1.00	66.82	57
NB TH	0.13	0	0.000	39.7	345	3.0	0.280	42.75	1.00	42.75	
SB LT	0.16	0	0.000	43.7	600	15.3	0.280	59.03	1.00	59.03	56
SB TH	0.13	1	0.125	42.2	300	6.8	0.306	49.05	1.00	49.05	
EB LT	0.22	1	0.222	40.2	420	23.3	0.613	63.54	1.00	63.54	54
EB TH	0.11	0	0.000	35.2	420	0.7	0.400	35.85	1.00	35.85	
WB LT	0.29	0	0.000	26.2	1470	1.1	0.400	27.30	1.00	27.30	35
WB TH	0.39	1	0.391	30.6	735	17.0	0.613	47.52	1.00	47.52	

INTERCHANGE DEL 46 SECONDS LOS E

TABLE 4 SUMMARY 2010 PM PEAK HOUR EXAMPLE ANALYSES

METHOD	RESULTS	
HCM PLANNING	UNDER (WEST)	NEAR (EAS
HCM OPERATIONS	LOS C (WEST)	LOS B (EAS
CAPELLE/PINNELL	OVER CAPACITY	
CHURCH	LOS F	
LEE	OVER CAPACITY	
HOOK	LOS F	
CAPELLE/PINNELL UPDATED	OVER CAPACITY	

Conclusions and Recommendations

There are those who are well versed in capacity analysis and recognize that the analysis of a diamond interchange as two separate intersections should not be done, however they might be unaware of the widespread nature of the practice. Many practitioners seem to believe that it must be appropriate, since there is no other method provided in the HCM or is there any indication that separate intersection analysis for closely spaced signalized intersections will produce invalid results.

One must wonder as to the consequences of this practice. We all recognize that the closely spaced signalized diamond intersections are among the most congested locations on our roadway networks. Is it possible this could be the result of improper capacity analysis of these locations over the last 20–30 years?

This situation must be corrected as soon as possible. If there are acceptable analytical techniques for capacity analysis of diamond intersections, we should present them to the traffic engineering community. If there are not, we should develop them and, in the meantime, provide guidance to the practitioner as to how to determine the capacity of these facilities. If the appropriate method is PASSER III (see the first conference paper in this Circular), it should be made clear that is the recommended procedure. As a minimum, the *Highway Capacity Manual* should caution the user as to the possible overestimation of capacity when analyzing the diamond interchange as two separate intersections.

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DISCUSSION

Jim Powell, Session Moderator

Jim Lee raises some useful points and identifies some of the complexities in evaluating performance at closely-spaced diamond interchanges. It is important to remember that the goal of the paper is to address evaluation methods practitioners have used in actual applications, based on their knowledge of HCM methods or similar procedures. In fact, as will be seen in the next paper, many practicing state transportation engineers rely heavily upon HCM procedures for interchange analyses, using them more than any other analysis techniques.

It is worthwhile to consider some of the detailed assumptions made in the paper for the various analysis techniques. A good example occurs in the Capelle/Pinnell methodology using the updated critical capacity sum (see Figure 6). An important adjustment to derive the sum of critical lane volumes is the multiplication of approach volumes by 1.20, which is equivalent to dividing by a peak hour factor of 0.83 (1/0.83 = 1.20). Without the adjustment, the critical lane volume sum would be 1,707 (i.e., the same as in Figure 4). This sum would be well below the updated critical capacity value of about 1,850 (per Table 1). The question then becomes, is it appropriate or not to apply the peak hour factor adjustment?

Similar questions arise in the more detailed Hook analysis procedure (Tables 2 and 3). The careful reader will note that in Table 2, a peak hour factor of 0.90 has been used, which is a fairly typical default value. In Table 3, however, the peak hour factor has been changed to 1.0, with the effect that the delay estimates are reduced. More important, however, Table 3 is based on saturation flow rates that are typically about 20% higher than in Table 2 (e.g., 1800 vphpl vs 1515 vphpl), resulting in even greater delay reductions. Given the planning context of the problem, is it legitimate to apply this kind of adjustment factor, even if based on "hard" field data?

These questions suggest a broader conclusion regarding planning methodologies: it is difficult to simplify the complicated characteristics and interactions of closely-spaced, coordinated signals into a straightforward set of equations. As the paper shows, there are many assumptions and operating parameters that need to be considered, and it is not immediately obvious how best to incorporate these.

What clearly is needed is a thorough understanding of the traffic behavior and flow characteristics at the interchange. From there, a comprehensive model can be developed to capture the essential characteristics and provide appropriate performance measures. Once a detailed model is developed, fully validated from field data, and run for a variety of interchange conditions, it might be possible to develop planning techniques similar to those discussed here. Even in a planning situation, there probably should be mechanisms to vary key parameters (e.g., saturation flow rate), in order to consider the effect on performance. Along these lines, Research Problem Statement 1 in this Circular addresses basic characteristics of interchange area operations and the modelling of them. That research, which will be funded in 1994 under the National Cooperative Highway Research Program, should develop appropriate analysis techniques to address many of these concerns.

SURVEY OF STATE DOT PERSPECTIVES ON INTERCHANGES

Kent Lall, Portland State University; James L. Powell, Felsburg Holt & Ullevig; and Michael Church, California Department of Transportation

Introduction

The Interchange Subcommittee (Highway Capacity and Quality of Service Committee A3A10 Transportation Research Board) undertook to document the types of interchanges currently in operation in order to learn from the experience that has been gained from their performance. The ultimate objective will be to develop procedures for the determination of the capacity of interchanges similar to other highway elements embodied in the *Highway Capacity Manual*. This will be achieved through defining research objectives, preparing a scope of work and identifying funding sources, while building on existing experience. The survey was intended to identify knowledge gaps in both interchange characteristics and specific operational features, to help set priorities for future research and development. A related area is the need for new evaluation techniques for operations of other closely-spaced intersections near ramp terminals.

In the nomenclature used in the AASHTO "Green Book", the facilities under consideration are service interchanges, as opposed to system interchanges (i.e., freeway/freeway). Specifically, a service interchange is the grade-separated junction of a through roadway with a typically lower classification roadway, and includes the at-grade intersections between through roadway ramps and the lower classification roadway. Freeway/arterial interchanges are the typical configuration, though arterial/arterial interchanges are also included. The at-grade intersections are the primary focus here, and can be either signalized or unsignalized, even in urban areas.

The questionnaire/survey was sent to thirty-two traffic engineers in twenty six states, based on a list of AASHTO Traffic Engineering Subcommittee members (Highway Design Committee) and personal contacts through the Interchange Subcommittee. The goal was to reach primarily state level individuals from geographically diverse areas who were responsible for planning and operational functions. Twenty-four questionnaires were returned, which represents a 75% response rate (see Figure 1). A full text of the questionnaire is included as Appendix A.

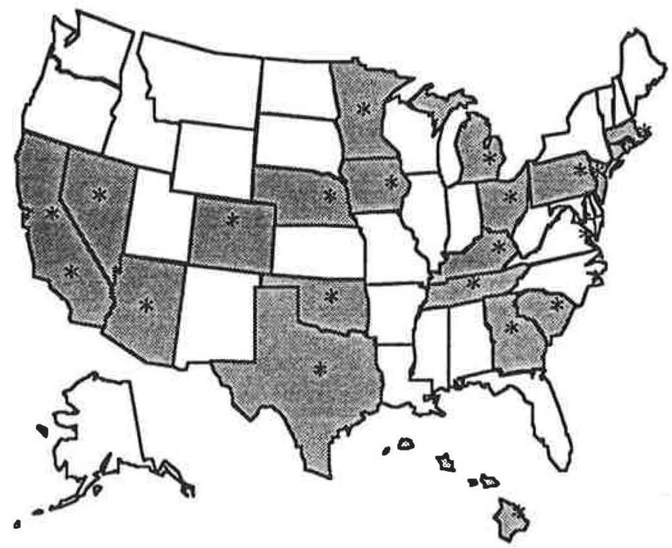

FIGURE 1 Distribution of questionnaire and responses.

Classification of Interchange Types In Use or Under Consideration

Based on a compilation of the survey results, it appears that the following interchange types are currently in operation (Question #1):

TABLE 1: INTERCHANGE TYPES IN USE

TYPE	NO. OF RESPONSES
Conventional diamond	24
Partial cloverleaf	23
Full cloverleaf	20
Trumpet	18
Split diamond	17
Directional	16
Tight diamond	15
Single point diamond interchange*	8
Three level diamond	6
Half diamond	2
Partial diamond	2
Semi-directional	2
Partial turban	1
Button hook	1
Slip ramp	1

*Also known as single point urban, urban diamond, or sometimes urban interchange.