TRANSPORTATION RESEARCH
CIRCULAR

Number 430 July 1994

Interchange Operations on the Local Street Side: State of the Art

TRANSPORTATION RESEARCH BOARD / NATIONAL RESEARCH COUNCIL

TRANSPORTATION
RESEARCH
CIRCULAR

Number 430, July 1994
ISSN 0097-8515

INTERCHANGE OPERATIONS ON THE LOCAL STREET SIDE: STATE OF THE ART

COMMITTEE ON HIGHWAY CAPACITY AND QUALITY OF SERVICE (A3A10)

Adolf May, Jr., Chairman
Wayne K. Kittelson, Secretary

Rahmi Akcelik	Douglas W. Harwood	Nagui M. Rouphail
James A. Bonneson	Michael Kyte	Ronald C. Sonntag
Werner Brilon	Joel P. Leisch	Alex Sorton
Kenneth G. Courage	John Morrall	Dennis W. Strong
Rafael E. De Arazoza	Barbara K. Ostrom	Stan Teply
Richard Dowling	Ronald C. Pfefer	Pierre-Yves Texier
Daniel B. Fambro	James L. Powell	R. Troutbeck
Ronald K. Giguere	William R. Reilly	Thomas Urbanik, II
Fred L. Hall	Roger P. Roess	John D. Zegeer

Richard A. Cunard, Transportation Research Board Staff

Subscriber category
IVA highway operations, capacity, and traffic control

Transportation Research Board
National Research Council
2101 Constitution Avenue, N.W.
Washington, D.C. 20418

PREFACE

This circular has been prepared as a volunteer effort by members of the Interchange Subcommittee of the Transportation Research Board (TRB) Committee on Highway Capacity and Quality of Service (A3A10), with contributions from participants at a conference session at the 1992 TRB Annual Meeting.

Following is a list of Interchange Subcommittee members:

James L. Powell (Chair), *Felsburg Holt & Ullevig*
James A. Bonneson, *University of Nebraska*
Michael Church, *California Department of Transportation*
Michael F. Holling, *JHK & Associates*
B. Kent Lall, *Portland State University*
Jim Lee, *Lee Engineering*
Joel P. Leisch, *CH2M Hill/Jack E. Leisch Transportation Group*
Scott J. Parker, *Edwards and Kelcey, Inc.*
Fred Rooney, *California Department of Transportation*
Thomas Urbanik, II, *Texas A&M University*

The Subcommittee gratefully acknowledges the assistance of Ms. Catherine Ollila of Felsburg Holt & Ullevig in the word processing for this circular.

TABLE OF CONTENTS

INTRODUCTION AND PURPOSE

INTRODUCTION AND BACKGROUND

Interchanges are the critical connections between roadways of the same or different functional classifications. Large turning volumes and changes in speed from acceleration/deceleration maneuvers place unique operational demands on the road system at these locations. This Circular deals primarily with interchanges involving roadways of different functional classifications, and the "local" street side refers to the lower classification facility (even though this facility may be an arterial or possibly even an expressway).

In the terminology of the AASHTO "Green Book" (*A Policy on Geometric Design of Streets and Highways,* 1990, American Association of State Highway and Transportation Officials), the Circular covers "service interchanges," i.e., those that involve a transition between facility types as contrasted with "system interchanges" that are typically freeways that involve free-flow transitions. Although there is some mention of system interchanges in this Circular, unless otherwise defined, the term interchange is used to refer to service interchanges.

The focus of this Circular is what happens on the local street in terms of operating characteristics. This includes: stop, delay, merge, and weaving behavior. Ramp intersections on these local streets may be either signalized or unsignalized. They are often closely spaced to one another or to adjacent intersections depending on the interchange type and road system characteristics. The close proximity of the intersections results in their operating as a single system.

While the operating characteristics of signalized intersections have been the subject of major research for many years, interchange operations on local streets have received significantly less attention. The current *Highway Capacity Manual* (HCM; Transportation Research Board Special Report 209) devotes an entire chapter to signalized intersections, but there is no discussion of operations at and near interchanges. Yet these junctions are often the critical intersections in their road networks and pose unique operating and safety concerns that complicate functional design, signal timing, and signing.

A good example of a unique operating characteristic is the delay that occurs on internal links of signalized interchange intersections. Because of the immediate upstream metering of traffic flows, these downstream delays are not subject to random arrivals, nor can they be characterized by a simple progression type. Thus, the signalized intersection delay equations given in Chapter 9 of the HCM are not fully applicable to interchange intersections operating under coordinated control.

In recognition of the knowledge gap related to interchanges, the TRB Committee on Highway Capacity and Quality of Service formed an Interchange Subcommittee. The charge was to deal with operating characteristics of interchanges plus up to one additional signalized intersection adjacent to each ramp terminal, for a total of up to four intersections. Although the typical case would be an interchange between a freeway and arterial -- arterial/arterial and expressway/arterial interchanges would be included as well. Again, the problem areas to be addressed are operation on the lower classification road or, more properly, the one with more impeded through or turning movements.

Merge/diverge and weaving characteristics on the free-flow roadway are concerns of the Freeway Subcommittee. New research on these characteristics is underway (NCHRP Project 3-37, *Capacity and Level of Service at Ramp-Freeway Junctions*), though an eventual goal is to integrate concerns on the free-flow facility with those on the impeded-flow. Similarly, other potential goals are to jointly consider geometric design, operational concerns, and incorporate safety as an explicit quality of service measure.

PURPOSE AND SCOPE

The purpose of this Circular is to provide a state-of-the-art review of interchange operations. Though major knowledge gaps remain, a good deal of theoretical and practical experience does exist. Only by evaluating and summarizing what is already known can intelligent choices be made regarding needed research and future directions.

Specifically, this Circular includes the following:

- Definitions and characteristics of service interchanges;
- A general search of the literature;
- Four papers presented at the 1992 TRB Annual Meeting conference session titled "Interchange Operations on the Local Street Side" sponsored by the Interchange Subcommittee. These four papers survey the problem generally; summarize computer software available for evaluating operation; discuss analysis

techniques practicing engineers have used; and present the results of a survey of state DOT's on interchanges; and

- Initial research problem statements on interchanges.

This Circular serves to highlight and define the interchange issues based on a review of current knowledge and understanding. From this base, major new initiatives can be pursued.

This document should serve as a useful source of information for those currently involved with the planning, design, construction and operation of interchanges. It also provides introductory material to the subject area.

SERVICE INTERCHANGE FORMS AND CHARACTERISTICS

INTRODUCTION

The at-grade intersections associated with service interchanges in an urban/suburban area are very important design and operational elements of a freeway and arterial network. They must be designed to function in harmony with the freeway traveled way, the ramps, and the arterial street. In many cases they control the capacity and operation of the system. Consequently in their design, the at-grade ramp terminals must have adequate lanes on each approach, appropriate channelization to facilitate turning, and sufficient storage lengths for queued left-turning and right-turning vehicles and through traffic. As well, these terminals require appropriate and properly placed traffic control, compatible and clear pavement marking and consideration for pedestrians, buses, plus related features.

While the same items must be considered in a rural setting, it is design and safety features that typically are the most important in rural interchange planning. The reason is that at most rural locations, volumes are usually low enough that they do not approach the physical load carrying ability of the facility, plus right-of-way often is not a serious constraint. The purpose of this chapter is to review basic interchanges forms mainly in settings where capacity and operations are prime concerns for further research and development. Thus the discussion is most pertinent to urban and suburban environments, in addition to those rural locations where volumes are relatively high, or potentially where right-of-way is seriously constrained.

INTERSECTION/INTERCHANGE FORMS

There are two basic forms of service interchanges in urban and suburban areas producing different intersection types, diamonds and partial cloverleafs (see Figures 1 and 2). These two basic forms include several variations that create six intersection types of primary interest. The six intersection types are those associated with 1) basic diamonds (conventional, compressed and tight urban); 2) the single point urban diamond; 3) the split diamond; 4) the Parclo A; 5) the Parclo B; and 6) the Parclo AB (2 quad).

DIAMOND INTERCHANGES

As demonstrated in Figure 1, there are five diamond interchange forms that produce different intersection and spacing types. The conventional diamond, compressed diamond, and the tight urban diamond have three approaches at each at-grade ramp terminal. The difference in design and channelization of these forms often depends upon location. The conventional diamond with intersections more than 800 feet apart is usually found in rural areas. Normally, stop control is used on the freeway exit ramp approaching each at-grade intersection. Capacity is usually not an issue. However, intersection sight distance and appropriate channelization to reduce the probability of wrong way movements are often the primary design and operational considerations.

The compressed diamond with intersections 400 to 800 feet apart usually is found in suburban areas. Channelization design for the intersection and cross road is important not only in facilitating turning movements and providing sight distance, but also in preventing wrong-way movements onto a ramp and, consequently, the freeway. The two intersections are almost always signalized, with or without interconnection.

The tight urban diamond, which primarily occurs in highly developed urban areas, is signal controlled. Because the two intersections are closely spaced (less than 400 feet apart), there usually is overlap in design elements—often left-turn lanes. Consequently the two intersections must be designed as a system. This is true not only of the geometrics, but also of the operational and capacity analyses and determination of signalization requirements. The signals for the two intersections should be interconnected, and signal timing needs to recognize the specific spacing/travel time relationship between intersections and make use of appropriate overlap phasing.

A single point (urban) diamond has one intersection with four approach legs of very different design and operational characteristics than the two previous diamond types. These are usually signalized with three basic phases: cross street lefts, cross street throughs, and off ramp lefts. Off ramp rights, which are typically unsignalized, operate smoothly with no opposing traffic during the cross street left turn phase and not the off ramp left turn phase.

The split diamond with one-way frontage roads results in two approach legs to each intersection. Signalization usually is used with the signals interconnected to optimize operation, using two or three phase operation if the cross streets are two-way roads. With one-way cross streets, signalization is two phase at all four intersections. The same two phase operation also applies to a three-level diamond, in which through

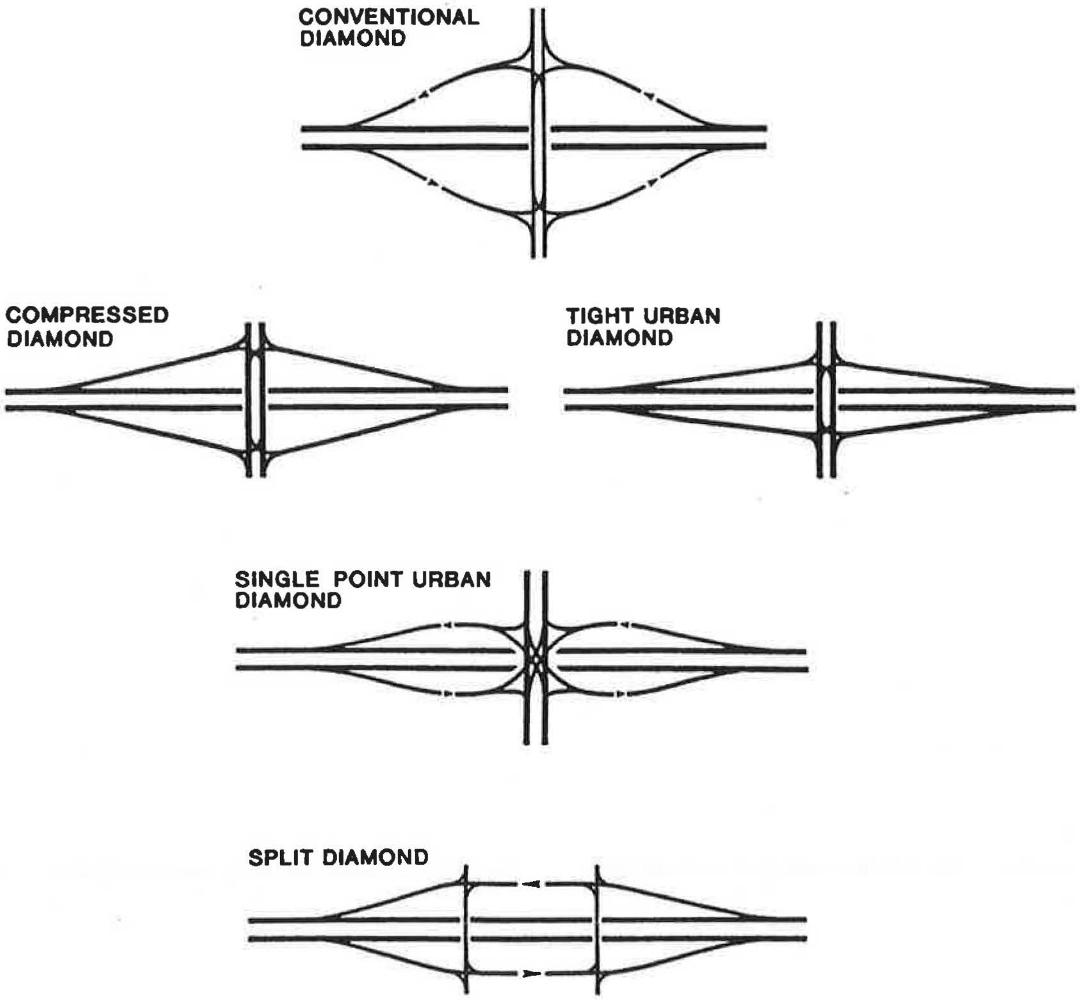


FIGURE 1 Diamond interchanges.

movements on both roadways are grade-separated. Split diamonds and three-level diamonds are not discussed further in this review, because they are less common than the other forms.

PARTIAL CLOVERLEAF INTERCHANGES

The other basic service interchange form is the partial cloverleaf, or parclo for short. It may be noted that full cloverleafs historically have been designed and built in several areas of the country. However, their use has been declining steadily in nearly all states for a variety of reasons, as discussed in Section 4, Chapter IV of this Circular. In keeping with this development, the following discussion covers only the parclo as a basic interchange form that is generally considered for new construction or reconstruction.

The three partial cloverleaf interchanges in Figure 2 are the most common forms in urban/suburban areas: Parclo A, Parclo B and Parclo AB. Although there are other parclo forms with one (see Figure 2), two or even three loops, the first three are the most basic types. In addition, the "2 quad" versions of Parclo A and Parclo B in Figure 2 are generally considered for new designs only where topographic or right-of-way constraints dictate their use. The reason is the large number of conflicting movements at the intersection areas, with potential for wrong-way movements onto ramps, unless intersections are carefully channelized. The 2 quad versions normally require three phase signalization, and

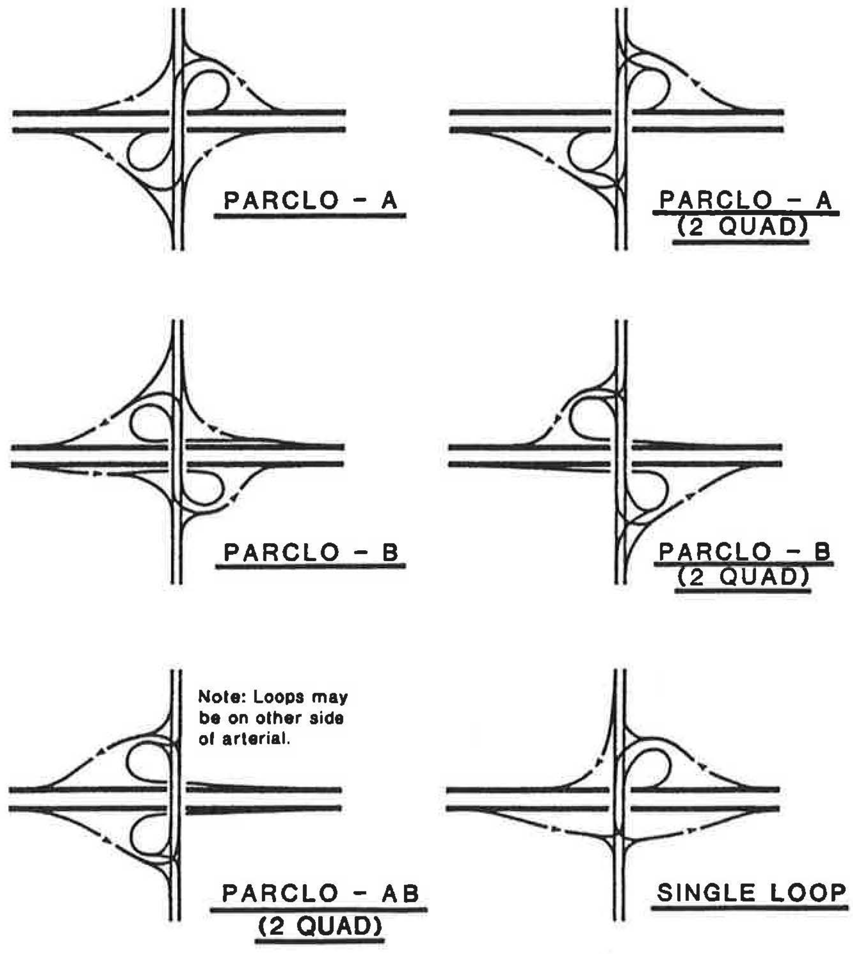


FIGURE 2 Parclo interchanges.

are discussed further only for the Parclo AB, for which the 2 quad configuration is more common.

The Parclo A has two intersections, both of which have three approaches and require two phase control when signalized. It should be noted that the only left turns are off the ramp onto the cross road. Generally the distance between intersections is 600 to 900 feet.

The Parclo B also has two intersections, but they have only two approach legs each. Left turns are made off the cross road onto the entrance ramp, requiring two phase signalization. The through movement adjacent to the left-turning queued traffic need not stop at the intersection. As with the Parclo A, the two intersections are generally 600 to 900 feet apart.

The Parclo AB (2 quad) has two somewhat similar intersections, with each having three approach legs. The difference between the two intersections is the place where the vehicles turning left off the cross road onto the ramps are stored. At one intersection the left-turning vehicles are stored between the intersections, while at the other the left-turning vehicles are stored external to the interchange. Again, intersection spacing is 600 to 900 feet. Three phase signal operation is normally required at both intersections.

Historically, a number of Parclo ABs have been constructed and many are still operational. However, performance generally has not been good for several reasons. First, the left turn from the arterial at one of the intersections to go right on the freeway violates driver expectancy and can cause driver confusion. Second, without appropriate channelization wrong-way movements can occur. In some instances also, weaving between the intersections along the arterial is a problem. This interchange form is included here because it exists in urban areas and needs to be analyzed. It is not generally a desirable form for new construction, however, and should be considered only where physical or land ownership precludes other forms.

TABLE 1 CHARACTERISTICS OF URBAN/SUBURBAN SERVICE INTERCHANGES

Typical Characteristics	Interchange Type					
	Compressed Diamond	Tight Urban Diamond	Single Point (Urban) Diamond	Parclo A	Parclo B	Parclo AB (2 Quad)
Ramp Separation (feet) (Centerline-to-Centerline) (1)	400'-800'	Less than 400'	150'-250' (External Stopline-to-Stopline)	600'-900'	600'-900'	600'-900'
Typical Ramp Intersection Traffic Control	Two actuated, sometimes interconnected	One actuated One or two pretimed	3-Phase (one signal)	Two, 2-phase signals	Two, 2-phase signals	Two, 3-Phase Signals
Left-Turn Bay Geometry	Bay tapers overlapped, 150 to 300 ft. bay length	Parallel bays, if any; no bay taper	Usually 2 lanes external to intersections	On exit ramp only	Tapers generally do not overlap	No overlap
Signal Coordination (Ramp Terminal to Ramp Terminal)	Often needed but may require complex signalization, or difficult to obtain	Needed and easily achieved using phase overlaps	Not needed	Possible	Possible	Questionable
Applicable Volume Ranges	Moderate	Moderate to High	Moderate to High	Moderate to High	Moderate to High	Moderate
Bridge Width	Through lanes plus median, often part or all of left turn bay(s)	Through lanes plus both left turn bays, if provided	Platform above freeway (complex). Freeway over requires long span. (2)	Thru lanes on Arterial	Thru lanes plus median for left turns	Thru lanes plus left(s) for one intersection
Operational Experience	Acceptable-sometimes need for progression or interconnection a problem	Acceptable	Acceptable	Acceptable	Acceptable	Acceptable/poor potential safety deficiencies

(1) External stopline-to-stopline separation typically 30' to 60' more.

(2) Single Point needs to account for large radius of off-ramp left turn.

Each of the interchanges and intersections described are unique. However, every other diamond or parclo interchange form utilizes one or more of the basic intersections types embodied in the above.

COMPARISON OF OPERATIONAL AND DESIGN CHARACTERISTICS

The goal of this section is to give a general overview of the basic interchange forms, and to provide some initial understanding of their characteristics as a benchmark for further research. The focus is moderate–to–heavy volume interchanges, normally found in urban and suburban settings. Table 1 summarizes characteristics of the following interchange forms:

- Compressed Diamond,
- Tight Urban Diamond,
- Single Point (Urban) Diamond,
- Parclo A,
- Parclo B, and
- Parclo AB (2 quadrant).

BIBLIOGRAPHY

The following list of references is drawn from several sources that include:

• A search of the TRIS database on interchanges and interchange operations;
• Selected references identified by the TRIS entries; and
• Individual titles familiar to subcommittee member relating to the subject area.

Readers are invited to submit names of other pertinent works that may contribute to the subject area.

INTERCHANGE PLANNING & OPERATIONS BIBLIOGRAPHY

1. D.J. Hook and J. Upchurch. "Comparison of Operational Parameters for Conventional Diamond Interchanges and Single Point Diamond Interchanges." Paper Presented at the 71st Annual Meeting of the Transportation Research Board, January, 1992.

2. A. Sadegh, L.J. Pignataro, A.K. Bladikas, and M.S. Iqbal. "Operation of Weaving Areas under Non-Freeway Conditions." Paper Presented at the 71st Annual Meeting of the Transportation Research Board, January, 1992.

3. E.Y. Cheng. "Accident Analysis for Single-Point Urban Interchange." *Compendium of Technical Papers of the 61st Annual Meeting of the Institute of Transportation Engineers*. Institute of Transportation Engineers, Washington, D.C., 1991, pp. 29-32.

4. M.D. Izadi. "Traffic Control and Traffic Operation of Urban Interchange." *Compendium of Technical Papers of the 61st Annual Meeting of the Institute of Transportation Engineers*. Institute of Transportation Engineers, Washington, D.C., 1991, pp. 24-28.

5. C.J. Messer, J.A. Bonneson, S.D. Anderson, and W.F. McFarland. *NCHRP 345: Single Point Urban Interchange Design and Operation Analysis.* Transportation Research Board, Washington, D.C., December, 1991.

6. *Paired Intersections Project - Final Report.* Report No. WD TE91/017. Australian Road Research Board, Nunawading, Australia, November, 1991.

7. N.M. Rouphail and R. Akcelik. *Paired Intersections: Initial Development of Platooned Arrival and Queue Interaction Models.* Report No. WD TE91/010. Australian Road Research Board, Nunawading, Australia, July, 1991.

8. M.J. Poppe, A.E. Radwan, and J.S. Matthias. "Some Traffic Parameters for the Evaluation of the Single-Point Diamond Interchange." *Transportation Research Record 1303.* Transportation Research Board, Washington, D.C., 1991, pp. 113-124.

9. J.A. Bonneson. "Operational Efficiency of the Single-Point Urban Interchange". *ITE Journal*, Volume 62, No. 6. Institute of Transportation Engineers, Washington, D.C., June, 1992, pp. 23-28.

10. D.B. Fambro and J.A. Bonneson. *Application Guide for the Microcomputer Version of PASSER III-88.* Interim Report. Report No. FHWA/TX-90/478-2F. Texas Transportation Institute, College Station, Texas, April, 1990, 68 pages. (NTIS)

11. J.E. Copelan. *Prevention of Wrong-Way Accidents on Freeways.* Report No. FHWA/CA-TE-89-2. California Department of Transportation, Sacramento, California, June 1989, 95 pages. (NTIS)

12. J.P. Leisch, T. Urbanik II, and J.P. Oxley. "A Comparison of Two Diamond Interchange Forms In Urban Areas." *ITE Journal*, Vol. 59, No. 5. Institute of Transportation Engineers, Washington D.C., May 1989, pp. 21-27.

13. J.A. Bonneson and C.J. Messer. *A National Survey of Signal-Point Urban Interchanges.* Report No. FHWA/TX-88/1148-1. Texas Transportation Institute, College Station, Texas, Mar. 1989, 62 pages. (NTIS)

14. C.E. Lee, R.B. Machemehl, and W.M. Sanders. *TEXAS Model Version 3.0 (Diamond Interchanges).* Report No. FHWA/TX-90+443-1F. University of Texas, Austin, Texas, Jan. 1989, 224 pages. (NTIS)

15. J.L. Hibbard and J.C. Cline. "Application of Signal Timing Methodology to Suburban Arterials With Diamond Interchanges". *Compendium of Technical Papers: ITE 58th Annual Meeting.* Institute of Transportation Engineers, Washington D.C., 1988.

16. D.B. Fambro and J.A. Bonneson. "Optimization and Evaluation of Diamond Interchange Signal Timing". *Compendium of Technical Papers: ITE 58th Annual Meeting.* Institute of Transportation Engineers, Washington D.C., 1988. (ITE)

17. C.E. Wallace, K.G. Courage, D.P. Reaves, G.W. Schoene, G.W. Euler, and A. Wilbur. *TRANSYT-7F User's Manual.* Transportation Research Center, University of Florida, Gainesville, FL, October, 1988, 530 pages. (McTrans)

18. D.B. Fambro, N.A. Chaudhary, C.J. Messer, and R.U. Garza. *A Report on the Users Manual for the Microcomputer Version of Passer III-88.* Report No. FHWA/TX-88/478-1. Texas Transportation Institute, College Station, Texas, Sep. 1988, 132 pages. (NTIS)

19. L. Abbey and G.S. Thurgood. *Design and Operational Features of the Single-Point Urban Interchange.* Report No. UDOT-MR 91-003. Utah Department of Transportation, Salt Lake City, Utah, May, 1988, 100 pages (NTIS).

20. R.H. Oliver. "Relative Performance of Stop Sign Versus Signal Control at Diamond Interchanges." *ITE Journal*, Vol. 57, No. 10. Institute of Transportation Engineers, Washington D.C., Oct. 1987, pp. 17-24.

21. C.J. Messer and M-S Chang. "Traffic Operations of Basic Traffic-Actuated Control Systems at Diamond Interchanges." *Transportation Research Record 1114.* Transportation Research Board, Washington D.C., 1987, pp. 54-62.

22. F. Spielberg. "Some Thoughts on Suburban Transportation." *ITE Journal*, Vol. 57 No. 2. Institute of Transportation Engineers, Washington D.C., Feb. 1987, pp. 23-25.

23. T.R. Neuman. "Impacts of the 1984 AASHTO Design Policy on Urban Freeway Design." *Transportation Research Record 1100.* Transportation Research Board, Washington D.C., 1986, pp. 40-45.

24. D.W. Borchardt and EC-P Chang. *Alternative Analysis of X-Ramp and Diamond Ramp Designs.* Rept. No. TTI-2-8-84-335-1F. Texas Transportation Institute, College Station, Texas, Oct. 1986, 57 pages. (NTIS)

25. M-S Chang and C.J. Messer. "Volume Guidelines for Signalization of Diamond Interchanges." *Transportation Research Record 1010.* Transportation Research Board, Washington D.C., 1985, pp. 29-37. (TRB)

26. W.R. Stockton and S.M. Smith. *Guidelines for Identifying Improvements at Diamond Interchanges.* Rept. No. FHWA/TS-85/64+423-1F. Texas Transportation Institute, College Station, Texas, Dec. 1984, 102 pages.

27. M. Sargious and T. Tam. "Criteria for Signalized Simple Diamond Interchanges." *Proceedings of the 8th Annual Conference on New Directions in Transportation Engineering.* Institute of Transportation Engineers, Edmonton, Alberta Canada, 1983, 16 pages.

28. M. Sargious and T. Tam. "Vehicle Delay Reduction when a Signalized Intersection is Upgraded to a Simple Diamond Interchange." *ITE Journal.* Institute of Transportation Engineers, Washington, D.C., Oct. 1983, pp. 30-33.

29. *Diamond Interchange Program Users Manual.* Rept. No. FHWA-IP-80-4. Federal Highway Administration, Washington D.C., May 1982.

30. D.B. Tait, R.S. Hostetter, and D.F. Gould. *Remedial Measures for Problem Cloverleaf Interchanges.* Rept. No. FHWA-RD-78-87. Institute for Research, State College, Pennsylvania, May 1978, 145 pages. (NTIS)

31. *Control Strategies for Signalized Diamond Interchanges.* Rept. No. FHWA-TS-78-207. Federal Highway Administration, Office of Research & Development, Washington D.C., 185 pages, Apr. 1978.

32. C.H. Knapp, D. Ghosh, and R. Jain. "Simulation and Control of Traffic on a Diamond Interchange." *Transportation Research Record 644.* Transportation Research Board, Washington, D.C., 1977, pp. 132-137.

33. C.J. Messer, D.B. Fambro, and S.H. Richards. "Optimization of Pretimed Signalized Diamond Interchanges." *Transportation Research Record 644.* Transportation Research Board, Washington D.C., 1977, pp. 78-84.

34. D.B. Fambro, D.L. Putman, H.E. Haenel, L.W. Cervenka, and C.J. Messer. *A Report on the User's Manual for Diamond Interchange Signalization--Passer III.* Report No. FHWA-RD-77-T0253. Texas Transportation Institute, College Station, Texas, Aug. 1977, 74 pages. (NTIS)

35. J.M. Arey and W.L. Williams. "Report on Signal Control Strategies for Diamond Interchange". *Compendium of Technical Papers of the 47th Annual Meeting of the Institute of Transportation Engineers.* Institute of Transportation Engineers, Washington, D.C., 1977, pp. 123-135.

36. "Design and Control of Freeway Off-Ramp Terminals." *NCHRP Synthesis of Highway Practice Report No. 35*, Transportation Research Board, Washington, D.C., 1976.

37. C.J. Messer and D.J. Berry. "Effects of Design Alternatives on Quality of Service at Signalized Diamond Interchanges." *Transportation Research Record 538.* Transportation Research Board, Washington, D.C., 1975, pp. 20-30.

38. Technical Council Committee 6A-1. "Traffic Planning for Interchange Area Development." *Traffic Engineering.* Institute of Transportation Engineers, Washington, D.C., Sep., 1975, pp. 38-41.

39. *Real-Time Control System for Diamond Interchanges.* Federal Highway Administration, Washington, D.C., Dec. 1973, 22 pages.

40. J.A. Wattleworth. "A Capacity Analysis Technique for Highway Junctions." *Traffic Engineering.* Institute of Transportation Engineers, Washington, D.C., June, 1972, pp. 30-36. (Automated via FRIOP software distributed through *McTrans*).

41. P.K. Munjal, J.A. Nemeczky, and J.F. Torres. *Design Manual for Traffic Signal Control of Diamond Interchange Complexes - Volume 2.* Report No. FHWA-RD-73-35. Federal Highway Administration, Washington, D.C., June, 1972.

42. P.K. Munjal. "An Analysis of Diamond Interchange Signalization." *Highway Research Record 349.* Highway Research Board, Washington, D.C., 1971, pp. 47-64.

43. D.L. Woods. "Limitations of Phase Overlap Signalization for Two-Level Diamond Interchanges." *Traffic Engineering.* Institute of Transportation Engineers, Washington, D.C., Sep., 1969, pp. 38-41.

44. F.A. Wagner, D.L. Gerlough, and F.C. Barnes. *NCHRP 73: Improved Criteria for Traffic Signal Systems on Urban Arterials.* Highway Research Board, Washington, D.C., 1969.

45. C. Pinnell and J.H. Buhr. "Urban Interchange Design as Related to Traffic Operation - Part I and Part II." *Traffic Engineering.* Institute of Transportation Engineers, Washington, D.C., March and April, 1966.

46. S. Spitz. "Signalization of Diamond Interchanges." *Traffic Engineering.* Institute of Transportation Engineers, Washington, D.C., April, 1964, pp. 15-17.

47. C. Pinnell. "The Value of Signal Phase Overlap in Signalized Intersection Capacity." *Traffic Engineering.* Institute of Transportation Engineers, Washington, D.C., Dec., 1962, pp. 17-20.

48. C. Pinnell and D.G. Capelle. "Operational Study of Signalized Diamond Interchanges." *Highway Research Board Bulletin No. 324.* Highway Research Board, 1962, pp. 38-72.

49. L. Newman. "Traffic Operation at Two California Interchanges." *Highway Research Record 27.* Highway Research Board, Washington, D.C., 1963, pp. 14-43.

50. D.G. Capelle and C. Pinnell. "Capacity Study of Signalized Diamond Interchanges." *Highway Research Board Bulletin No. 291.* Highway Research Board, Washington, D.C., 1961, pp. 1-25.

51. O.T. Williams. "Some Considerations in the Design and Operation of Multi-Level Interchanges". *Presented at the Standards for Modern Highways National Conference.* Institute of Highway Engineers, London, England. Conference Paper, pp. 2-10.

52. *Highway Design Manual.* California Department of Transportation.

RAMP METERING BIBLIOGRAPHY: SELECTED REFERENCES

1. Y.J. Stephanedes, E. Kwon, and K. Chang. "Control Emulation Method for Evaluating and Improving Traffic-Responsive Ramp Metering Strategies." Paper presented at the 71st Annual Meeting of the Transportation Research Board, Washington, D.C., 1992.

2. J.L. Hibbard, J.D. Carvell, and B.G. Marsden. "An Overview of the Ramp Metering Subsystem for the Phoenix Freeway Management System." *ITE 1990 Compendium of Technical Papers.* Institute of Transportation Engineers, Washington, D.C., 1990, pp. 51-54.

3. *FREQ10PE - User's Manual.* Institute of Transportation Studies, University of California, Berkeley, CA.

4. *Ramp Metering Design Guidelines.* CalTrans Department of Transportation, Sacramento, CA, June 1989.

5. S. Yagar. "Predicting the Impacts of Freeway Ramp Metering on Local Street Flows and Queues." *Compendium of Technical Papers.* Institute of Transportation Engineers, Washington, D.C., 1989, pp. 509-512.

6. A. Hamad. *Evaluation of Ramp Metering Strategies at Local On-Ramps and Freeway-to Freeway Interchanges Using Computer Modelling Approach.* Ph.D. Dissertation. Michigan State University, 1987. (University Microfilms International)

7. C.W. Blumentritt, C. Pinnell, W.R. McCasland, D.W. Ross, and J. Glazer. *NCHRP Report 232 - Guidelines for Selection of Ramp Control Systems.* Transportation Research Board, Washington, D.C., 1981, 108 pages.

8. K. Ovaici, R.F. Teal, J.K. Ray, and A.D. May. "Developing Freeway Priority-Entry-Control Strategies." *Transportation Research Record 533.* Transportation Research Board, Washington, D.C., 1975, pp. 122-137.

9. J.H. Buhr, R.H. Whitson, K.A. Brewer, and D.R. Drew. "Traffic Characteristics for Implementation and Calibration of Freeway Merging Control Systems." *Highway Research Record 279.* Highway Research Board, Washington, D.C., 1969, pp. 87-106.

10. K.A. Brewer, J.H. Buhr, D.R. Drew, and C.J. Messer. "Ramp Capacity and Service Volume as Related to Freeway Control." *Highway Research Record 279.* Highway Research Board, Washington, D.C., 1969, pp. 70-85.

11. J.H. Buhr, W.R. McCasland, J.D. Carvell, and D.R. Drew. "Design of Freeway Entrance Ramp Merging Control Systems." *Highway Research Record 279.* Highway Research Board, Washington, D.C., 1969, pp. 137-149.

12. J.H. Buhr, L. Radke, B.M. Kirk, and D.R. Drew. "A Moving Vehicle Merging Control System." *Highway Research Record 279.* Highway Research Board, Washington, D.C., 1969, pp. 121-136.

13. L. Newman, A. Dunnet, and G.J. Meis. "Freeway Ramp Control--What it can and Cannot Do." *Traffic Engineering.* Institute of Transportation Engineers, Washington, D.C., June, 1969, pp. 14-21.

14. D.R. Drew. "Gap Acceptance Characteristics for Ramp-Freeway Surveillance and Control." *Highway Research Record 157.* Highway Research Board, Washington, D.C., 1967, pp. 108-143.

As an initial step to characterize the state-of-the-art in the interchange operations and help focus research, the Interchange Subcommittee hosted a Conference Session at the 1992 TRB Annual Meeting in Washington, D.C., titled "Interchange Operations on the Local Street Side." The four presenters covered a range of issues and concerns, briefly described as follows:

Dan Fambro (presenter) and Tom Urbanik of Texas Transportation Institute presented an overview paper on operational issues related to signalized interchanges. Though focused on diamond interchanges, issues of spillback, progression and levels of service were listed as pertinent to all interchange types.

Hobih Chen (presenter) of Viggen Corporation, plus Henry Lieu and Al Santiago of FHWA, provided an overview of various software packages that have been used to analyze interchange operations. Major packages discussed include HCS, SOAP, PASSER II and III, TRANSYT-7F, TEXAS, TRAF-NETSIM, and INTRAS.

Jim Lee of Lee Engineering presented a paper contrasting various techniques for evaluating diamond interchange operations from a planning perspective, starting with procedures first advocated 30 years ago. Not surprisingly, a variety of evaluation results occur in a sample application.

B. Kent Lall (presenter) of Portland State University, Jim Powell of Felsburg Holt & Ullevig and Michael Church of the California Department of Transportation (San Francisco) gave the results of a survey on interchanges and interchange operations from about one-half of the United States. Though a variety of responses came back, some common trends and themes emerged.

The discussion portion of the session involved the general audience in a discussion of future directions as well as perceived operational problems. The entire session along with the discussion portion was chaired by Jim Powell, chairman of the Interchange Subcommittee.

The papers presented represent the views and opinions of the authors, and have not been adapted as standards or procedures of the Interchange Subcommittee, or of the Committee on Highway Capacity and Quality of Service.

OVERVIEW OF SIGNALIZED INTERCHANGE OPERATIONS

Daniel B. Fambro and Thomas Urbanik II,
Texas A&M University

Introduction

The *Highway Capacity Manual* (HCM) (*1*) does not explicitly address analysis of signalized interchanges. In recognition of this shortcoming, the Transportation Research Board's Committee on Highway Capacity and Quality of Service established a subcommittee to consider the problem of analyzing the operation of signalized interchanges. This paper is intended to help identify the nature of the signalized interchange problem and begin formulating procedures for analysis.

There are a multitude of interchange configurations that could involve the use of signalized intersections. Generally speaking, the types of interchanges discussed in this paper would be those categorized by the American Association of State Highway and Transportation Officials in *A Policy on Geometric Design of Highway and Streets* (*2*) as service interchanges. These interchanges service the connection between freeways and arterial streets and include entrance and exit ramps connecting the freeway to an arterial street or parallel frontage road.

The interchanges discussed in this paper are represented by conventional two-level diamonds, split diamonds, three-level diamonds, and single-point urban (see Figure 1). Although these interchange forms are not all inclusive, they adequately illustrate the fundamental problems in analyzing signalized interchanges. It should be noted at this point that in addition to creating two closely spaced signalized intersections, these interchanges also create two intersections with higher percentages of turning traffic than typically occur at arterial street intersections.

The objective of this paper is to provide an overview of signalized interchange operations from both a technical and policy viewpoint. Specifically, this paper will discuss signalized interchange operational alternatives and issues, plus analysis and operational procedures. This paper, however, is not intended to provide answers to all signalized interchange questions. Rather, it is intended to identify a number of questions that need answers.

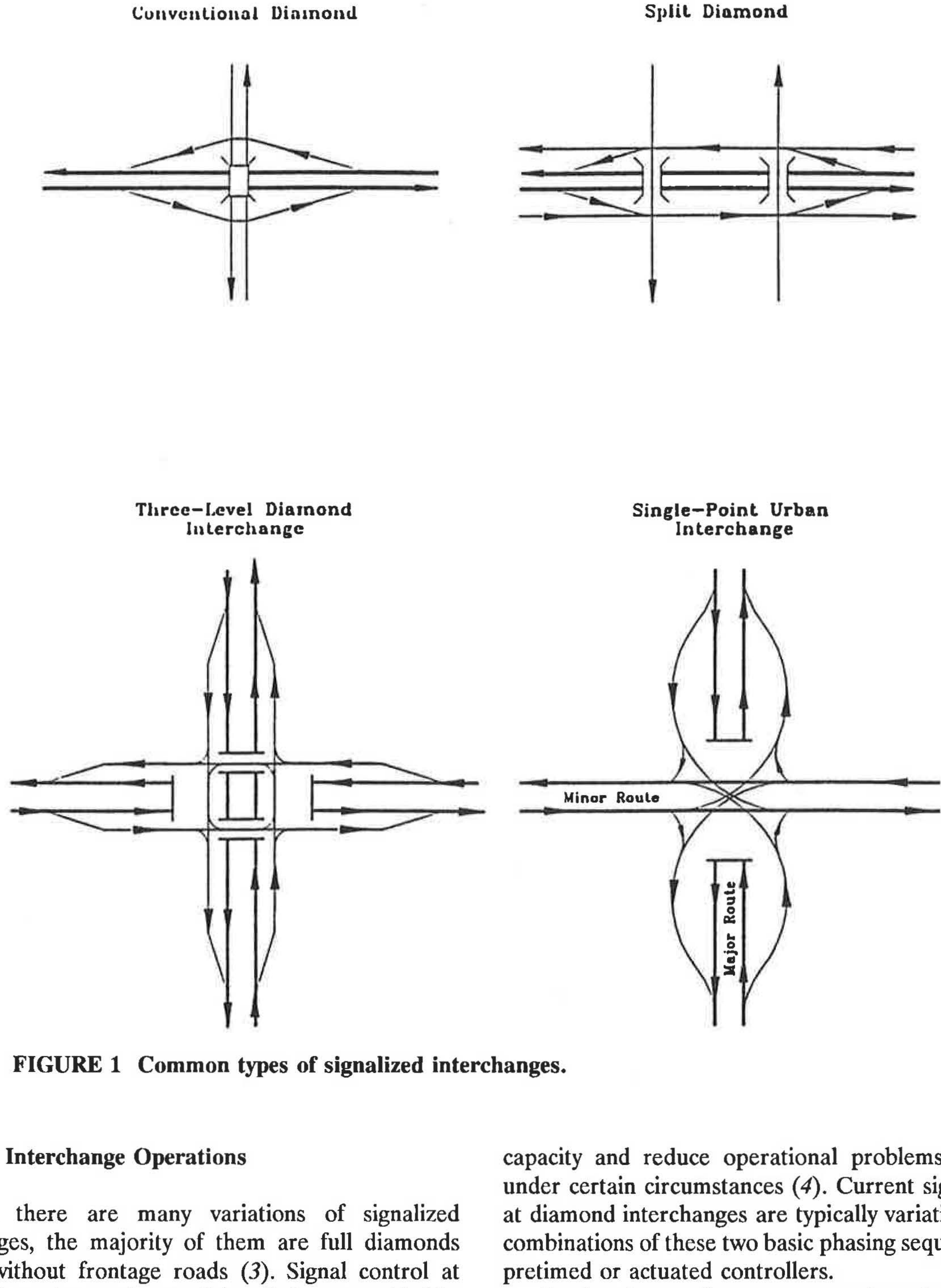


FIGURE 1 Common types of signalized interchanges.

Diamond Interchange Operations

Although there are many variations of signalized interchanges, the majority of them are full diamonds with or without frontage roads (*3*). Signal control at diamond interchanges has traditionally been provided by either a 3-phase pretimed signal sequence in which both off-ramp/frontage roads are released simultaneously (see Figure 2), or by two non-interconnected, full-actuated controllers with one controller at each intersection. The 4-phase, 2-overlap signal phase sequence (see Figure 2) developed by the Texas Transportation Institute in the late 1950's also has been used to increase interchange capacity and reduce operational problems and delay under certain circumstances (*4*). Current signal control at diamond interchanges are typically variations and/or combinations of these two basic phasing sequences using pretimed or actuated controllers.

Pretimed controllers are appropriate where a limited number of traffic patterns are found that repeat themselves on a daily basis. These controllers can be easily interconnected with adjacent signalized, controlled intersections. The basic phasing can be modified through changes in the split and offset if two pretimed controllers are used at the interchange (one at each cross street intersection) (*5*). Actuated controllers are

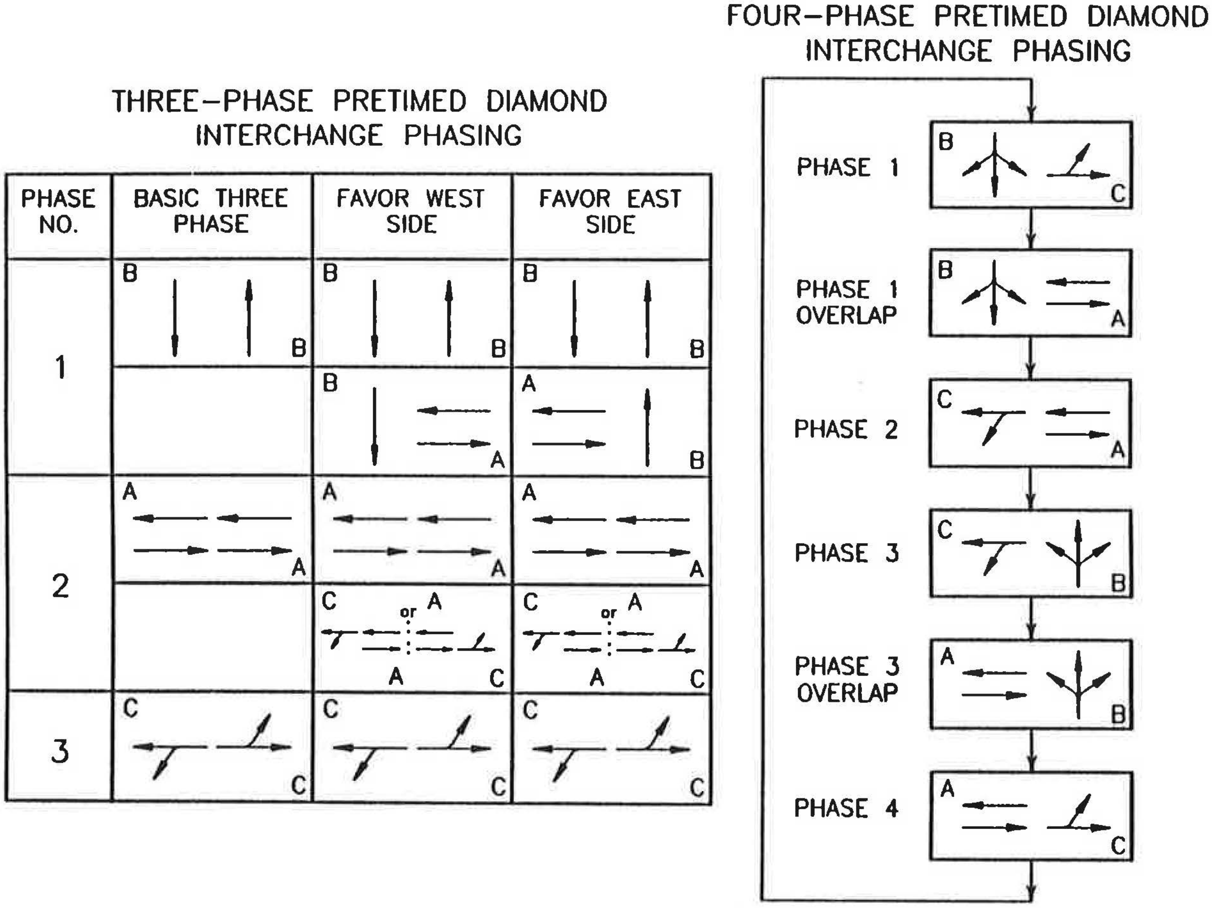


FIGURE 2 Common types of pretimed diamond interchange signal phasing.

appropriate where a large number of traffic patterns are required and these patters vary greatly on a daily basis. Because they are not easily interconnected with adjacent traffic signals, the primary usage of actuated controllers is at isolated diamond interchanges. When operating the 4-phase, 2-overlap signal phase sequence in an actuated mode, multipoint detection and detector switching logic on the ramp/frontage road is necessary to promote full utilization of the overlap phases and minimize the lost time associated with these phases (*6*).

The California Department of Transportation has developed a diamond interchange software program for the Model 170 controller unit which can provide either 3- or 4-phase actuated control strategies (*7*). Two standard NEMA full-actuated controller units also can be used to provide both 3- and 4-phase operation. The Texas Diamond Controller uses one NEMA full-actuated controller unit to provide both 3- and 4-phase operation at the same interchange (*8*). The change from one phasing operation to the other is made by a time clock or by external traffic responsive logic.

Split, offset, and cycle length determinations are additional considerations at a diamond interchange. The two intersections at the interchange can either be timed separately to minimize intersection delay (i.e., 3-phase control) or timed together to maximize interchange progression and thus minimize queue storage between the two intersections (i.e., 4-phase control). Neither method is universally better than the other (*9*) and each will probably result in different optimal cycle lengths (*6*). Left-turn lane requirements and type of protection (i.e., protected, protected/permitted, permitted) also must be considered. Recognizing this myriad of alternatives, the question arises: How do transportation engineers determine which strategy is most appropriate at any given signalized interchange?

Operational Issues

There are a number of operational issues that must be addressed in the analysis of a signalized interchange. First, traffic patterns at signalized interchanges are different from those at signalized intersections because of the higher percentage of turning movements at interchanges. Second, an analysis procedure for signalized interchanges must be flexible for a planning analysis and detailed for an operational analysis. Third, queue spillback considerations are extremely important at signalized interchanges because of the short spacing between intersections and the limited capacity for queue storage on some entrance and exit ramps. Fourth, the close proximity of adjacent intersections and/or access points may create weaving problems in the interchange area. In addition to operational issues with signalized interchanges, institutional issues between the freeway operator (usually the state) and the arterial operator (usually the city) may create problems in coordinating interchange operations. Each of these issues is discussed in the following sections.

Traffic Patterns

Because through traffic on one of the major roadways is grade separated, the primary movements at the two intersections of the interchange are left and right turns. This pattern is different from the pattern at signalized intersections where through movements tend to dominate. In fact, turning movement volumes and/or percentages at signalized interchanges often are two to four times greater than at signalized intersections.

In addition to higher turning movement volumes, the turning patterns change over time. For example, heavy left-turns during the morning peak at one of the intersections often result in heavy right-turns during the afternoon peak at the other intersection. This change means that in the morning peak multiple left-turn lanes may be needed at one of the intersections, but at other times of the day, additional through-right lanes may be needed. Unless the lane assignments can be changed by time of day, the turning lanes necessary to handle the peak traffic are under utilized during most of the day.

Planning versus Operations

As a result of the many signalized interchange forms, phasing alternatives, and changing traffic patterns, operational analysis procedures must be detailed and complex. Planning procedures, however, must be simple and allow flexibility because of the sensitivity to traffic patterns and the large number of unknowns at the planning stage.

Queue Spillback

Because of the limited distance between the two signalized intersections at an interchange, queue spillback potential between the two signals is especially critical. If the conditions (signal timing and intersection spacing) are such that the queue backs into the upstream signal, gridlock will occur. When this situation occurs, the ramp/frontage road signals may be green, but vehicles can not use the intersection because the interchange is blocked by stopped vehicles.

Queue spillback, however, is not confined to a between-signal problem. Queues can back up on an exit ramp and block the freeway's main lane if demand exceeds capacity at the exit ramp terminal. Queues also can back up on an entrance ramp and block the intersection if the demand exceeds the capacity at the entrance ramp terminal. In both cases, the potential for queue spillback must be carefully evaluated and take into account the potential consequences on adjacent facilities.

Weaving Areas

Traffic entering or exiting the freeway can create weaving problems on the arterial street or frontage road. If lengths between the interchange and adjacent signalized intersection on the arterial are not adequate, vehicles turning right from the ramp/frontage intersection and desiring to turn left at the next intersection on the arterial must weave across several lanes of arterial traffic. If intersection spacings are short or traffic volumes are heavy, weaving demands may contribute to congestion problems on the arterial.

Likewise, at interchanges with one-way frontage roads parallel to the freeway, traffic exiting the freeway to turn right at the frontage road arterial street intersection must weave across two or three lanes of frontage road traffic. If these distances are short, traffic volumes are heavy, or driveways are located within the weaving segment, operational problems may occur on the frontage road.

Institutional Issues

Different agencies are responsible for operating the signalized interchange and the adjacent signalized intersections (i.e., ramp/frontage road signals are typically controlled by the state and adjacent arterial signals are typically controlled by the local governments). This differing responsibility can cause problems due to different equipment, operational strategies, and traffic control objectives. For example, the state may be concerned with favoring vehicles exiting the freeway and not having them stop within the interchange. If so, they would adopt an operational strategy to accomplish this objective. The local government, on the other hand, may be concerned with progression along the arterial street and would desire to implement an operational strategy favoring through vehicles on the arterial and not having them stop at the interchange.

Analysis Issues and Procedures

The differences between the single-point urban and the other interchange forms highlight some of the difficulty in performing analyses of operational and geometric alternatives. The analysis of the single point urban using HCM procedures is relatively straightforward except for the selections of appropriate values for factors such as clearance interval and saturation flow rates. The selection of these factors is primarily a problem of limited field data reported in the literature. When it is necessary to compare a single-point urban to a conventional diamond, however, a variety of problems related to an equitable comparison using Chapter 9 of the HCM develop. The conventional diamond, which has two signals, requires separate analysis of each signal. However, the operational performance of the two signals cannot be considered in isolation. It is, therefore, necessary to consider the appropriateness of the HCM procedures to undertake a comparative analysis.

The HCM has two procedures that could be applied to signalized interchanges. The most basic analysis would be performed utilizing the signalized intersection procedures of Chapter 9 of the HCM. The only procedures for analyzing the system interaction of signals is using either the progression adjustment factors or the arterial analysis procedures of Chapter 11 of the HCM. The Chapter 11 procedures, however, are explicitly restricted to conditions with typical arterial street turning volumes. The limitations to signalized interchange analysis of the Chapter 9 procedures, including the progression adjustment factors, will be discussed later in this paper.

Qualitative Measures

Capacity and level of service are the basic components of an intersection operational analysis. The basic procedures are widely recognized. Capacity is reflected in the volume to capacity ratio (v/c ratio) and level of service is reflected in stopped delay. The primary measure of system effectiveness is the platoon ratio or the proportion of the volume arriving on green which reflects the quality of progression and the characteristics of the platoon.

Several important issues should be considered in the signalized interchange analysis. Two issues that need consideration and that are not explicitly a part of the current procedures are queue spillback and pedestrian effects. As mentioned previously, closely spaced intersections are subject to effects of downstream intersection spillback. Vehicular minimum timing requirements may not be sufficient to accommodate pedestrian minimum timing requirements. An additional issue is lost time, which include start-up losses, clearance time requirements, and phasing considerations. Each of these issues will also be discussed briefly.

Capacity Analysis Tools

The most fundamental consideration in interchange analysis is the capacity of the various lane groups and the geometry of the intersections. For simplicity, it is assumed that saturation flows are known or can be determined. The effective green time is variable; therefore, it is necessary to determine the appropriate value. Start-up and clearance values can be determined, although there is some question as to the appropriate values for single-point urban interchanges. A more difficult question involves the appropriate phase length and offset necessary for satisfactory system performance. This issue is especially important at signalized interchanges where turning movements are high.

Several computer models are available to analyze arterial streets and intersections. Simulation models offer the capability to evaluate alternative phasing patterns to minimize delay. The *Highway Capacity Manual* software (*10*) can be used for relatively quick analyses at specific locations or along arterial streets. The *Highway Capacity Manual* software uses a macroscopic, deterministic, off-line approach to evaluate traffic flow. The HCM procedures are useful in analyzing specific highway features. As traffic flow approaches capacity at a number of locations on a highway system, however, it is necessary to evaluate the roadways as a system. PASSER II-90 (*11*) is capable of analyzing isolated intersections as well as a series of signalized intersections. PASSER III-88 (*12*) is designed for diamond interchange analyses (i.e., two closely spaced intersections). NETSIM and TRANSYT-7F (*13*) also are capable of evaluating the operations along a series of intersections. Each model has unique qualities. Other computer models for intersection/interchange analysis also exist but are less widely used.

The previous summary of models was intended to indicate the range of alternatives that exist to the current HCM procedures. The development of computer models resulted in part from the recognition that sufficient green time on an approach does not necessarily translate into good operations. Stated another way, it is possible to optimize capacity at an individual intersection yet have an undesirable level of service or even a non-functional operation due to the inability of traffic to move due to spillback of the downstream queues. Capacity estimates must reflect realistic conditions, however no such measure of effectiveness exists in the HCM.

Level of Service

Intersection level of service is currently based on stopped delay. Delay is estimated from an equation having two terms: uniform delay and incremental delay. The uniform delay term assumes uniform arrivals while the incremental delay term accounts for random arrivals and cycle failures. A progression adjustment factor is used to account for platoons caused by upstream signal timing. Fambro, et al. discuss the progression factor considerations in the *Effects of the Quality of Traffic Signal Progression on Delay* (*14*). It is obvious that progression considerations affect delay at closely spaced intersections and should be taken into consideration when estimating delay.

For example, the timing plan for a typical conventional diamond interchange operating at high flow rates is likely to provide progression not only for the arterial movements but also for the ramp traffic turning through the interchange. Arrival patterns for these interior movements are platooned and delays caused by vehicles stopping within the interchange are likely to be minimal under a well designed phasing plan and unacceptable under a poorly designed phasing plan. The arrivals at the downstream signal are not likely to be random regardless of the origin of the traffic.

A further complication is the use of delay per vehicle as the measure of effectiveness when an analysis is performed for more than one intersection. Problems in comparing two alternatives exist even if two interchanges have equal numbers of vehicles, equal roadway lengths, and equal total delays. If the two interchanges have similar characteristics except that one has one signal and the other has two signals, the interchange with two signals will likely have less delay per vehicle. This discrepancy occurs because vehicles passing through more than one signal will be counted as two separate vehicles and the delay per vehicle is reduced because of the greater number of vehicles. Accounting for the total number of individual vehicles is only one issue when considering comparisons between alternatives with different numbers of signals. It should be noted that counting vehicles only once in a system of several intersections, however, does change the significance of the delay per vehicle criterion.

Queue Spillback

A fundamental assumption of most analyses is the nonexistence of spillback from downstream intersections. If the queue from a downstream intersection blocks a movement from an upstream intersection, effectively no capacity exists. This effect is pronounced especially in closely spaced (less than 600 feet between intersections) conventional diamond interchanges. Special timing plans are necessary to operate closely spaced diamond interchanges that are near capacity. These special phasing plans consider system effects of the two closely spaced intersections. The timing plans differ from those that would be implemented considering each intersection in isolation. The PASSER III computer model was developed explicitly to address this problem for diamond interchanges by constructing queue profiles for each of the internal movements.

Pedestrian Effects

Pedestrian flows may be minor, resulting in analysts ignoring their effects on operations. If pedestrian minimum timings exceed vehicle minimum timings, however, some consideration of pedestrian effects is warranted. Clearly, the level of service at an interchange is a function of pedestrian phasing and flows. The process of analysis is, however, complicated by the lack of guidance on how to consider such effects. This paper does not provide any guidance on how to conduct an analysis explicitly dealing with pedestrians, although this problem is clearly an area that merits further attention.

Operational Analysis Procedures

The most common method of analysis is based on the 1985 HCM procedures. This type of analysis is focused primarily on level of service based on stopped delay per vehicle. The procedure is limited obviously by the previously discussed issues. It is inappropriate to analyze individual intersections in isolation. The development of accurate progression adjustment factors could deal with some of the interaction issues as related to platoon dispersion. It would still be necessary, however, to address queue spillback potential.

An alternative to the HCM procedures is the use of computer models. A number of computer models were described earlier including PASSER III, which will be discussed only to further illustrate some of the issues and considerations. PASSER III is based largely on the delay equation in the HCM, however, it does use a delay offset analysis to address the effect of progression on the interior movements of a diamond interchange. Although PASSER III and other computer models are subject to some of the same problems as the HCM procedures, they do offer some improvements over an isolated intersection analysis. The advantages include measures of effectiveness in addition to stopped delay per vehicle, and the use of optimization to obtain the best feasible operations.

PASSER III, for example, is designed to find solutions that provide acceptable operations under the specific conditions of a conventional diamond. It does so partly through the incorporation of additional measures of effectiveness in the level of service analysis (i.e., three measures of effectiveness—average stopped delay, volume-to-capacity ratio, and storage ratio—are used to assess the interchange performance). Table 1 illustrates the measures of effectiveness and level of service threshold criteria used by PASSER III.

PASSER III also calculates level of service based on the total number of individual vehicles passing through the interchange.

Although PASSER III can address some of the identified problems, it is a special-purpose tool geared only to conventional diamond interchanges. It cannot, for example, evaluate a three-level diamond. TRANSYT-7F is a more flexible tool but does not explicitly address all spillback issues of the conventional diamond. It does, however, identify in the output the approaches with potential queue spillback problems. TRANSYT-7F also does not individually identify vehicles, so delay per vehicle is partly a function of the number of intersections.

Recommendations

The most critical issue in interchange analysis is an awareness by the user community of the limitations in the current *Highway Capacity Manual* procedures when applied to system problems such as signalized interchanges. Computer models offer the potential to reduce the number of problems associated with more complex analyses because they are generally more powerful tools than manual techniques. Additional measures of effectiveness such as volume to capacity and storage ratios, however, are needed to make the best use of these computer models. Furthermore, one should not assume that more sophistication eliminates all problems. Delay per vehicle is but one example where erroneous conclusions can be drawn if inappropriate comparisons are made between alternative interchange configurations.

Some recommendations can be made concerning the need for additional measures of effectiveness for signalized interchange analysis, as well as limitations in the measures of effectiveness currently being used. An analysis of closely spaced intersections (less than 600 foot spacing) should be based on three measures of effectiveness applied in the following order: storage ratio, volume-to-capacity ratio, and vehicular delay. If a level of service F condition (which needs to be defined for storage ratio) is calculated for either storage ratio or volume-to-capacity ratio for any movement and/or phase, no further calculations are made and the overall interchange level of service is reported as level of service F. If storage ratio and volume-to-capacity ratio for all movements and/or phases are acceptable, then vehicular delay can be used to determine overall interchange level of service.

Average stopped delay per vehicle should be limited in its application to a single intersection unless the number of individual vehicles in the interchange is known. That is, comparison between interchange alternatives with different numbers of intersections cannot be made fairly unless the number of individual vehicles in the interchange is known. Therefore, system or total interchange delay is a more appropriate measure of effectiveness when comparing interchange alternatives.

References

1. *Highway Capacity Manual.* Transportation Research Board, Washington, D.C., 1985.

2. *A Policy on Geometric Design of Highways and Streets.* American Association of State Highway and Transportation Officials, Washington, D.C., 1990.

3. "Signal Control Strategies for Signalized Diamond Interchanges." Report No. FHWA/IP-80-5. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, December 1980.

4. Woods, D.L., "Limitations of Phase Overlap Signalization for Two-Level Diamond Interchanges." *Traffic Engineering.* Vol. 39, No. 12 September 1969, pp.38-41.

5. Guyton, J.W., Yarbrough, J.W., and Arey, T.M. "Control Strategies for Signalized Diamond Interchanges." Report No. FHWA/TS-78-206. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, April 1978.

TABLE 1 PASSER III LEVEL OF SERVICE CRITERIA FOR OPERATIONAL MEASURES OF EFFECTIVENESS AT SIGNALIZED DIAMOND INTERCHANGES

Measures of Effectiveness	Level-of-Service					
	A	B	C	D	E	F
Volume-to-Capacity Ratio[a]	<.60	<.70	<.80	<.85	<1.0	>1.0
Average Vehicular Delay[b]	<6.5	<19.5	<32.5	<52.0	<78.0	>78.0
Interior Storage Ratio[c]	<.05	<.10	<.30	<.50	<.80	>.80

[a] "Guide for Designing and Operating Signalized Intersections in Texas."
[b] Total delay (1.3 times stopped delay).
[c] Average queue length per cycle (veh) divided by the available queue storage (veh).

6. Messer C.J. and Chang, M.S. "Traffic Operations of Basic Traffic Actuated Control Systems at Diamond Interchanges." Transportation Research Record 1114. Washington, D.C.: Transportation Research Board, National Academy of Sciences, 1988, pp. 54-62.

7. "Diamond Interchange Program User's Manual." Report No. FHWA/IP-80-4. Washington, D.C.: Federal Highway Administration, U.S. Department of Transportation, December 1980.

8. Haenel, H.E., Kosik, A.H., and Marsden, B.G. "Innovative Uses of Traffic Responsive Control in Texas." Proceedings of the 1983 Engineering Foundation Conference on "Traffic Monitoring and Control Systems." Henniker, New Hampshire, July 1983, pp 95-112.

9. Messer, C.J., Fambro, D.B., and Richards, S.H. "Optimization of Pretimed Signalized Diamond Interchanges." Transportation Research Record 644. Washington, D.C.: Transportation Research Board, National Academy of Sciences, 1977, pp 78-84.

10. *Traffic Software Integrated System User's Manual 2.0.* U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1989.

11. Chang, E.C.P. and C.J. Messer. "Arterial Signal Timing Optimization Using PASSER II-90 - Program User's Manual." TTI Report No. 467-3F, Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 1990.

12. Fambro, D.B., N.A. Chaudhary, C.J. Messer, and R.U. Garza. "A Report on the User's Manual for the Microcomputer Version of PASSER III-88." TTI Report No. 478-1, Texas Transportation Institute, The Texas A&M University System, College Station, Texas, 1988.

13. *TRANSYT-7F User's Manual.* U.S. Department of Transportation, Federal Highway Administration, Washington, D.C., 1988.

14. Fambro, D.B., E.C.P. Chang, and C.J. Messer. "Effects of the Quality of Traffic Signal Progression on Delay." NCHRP Report No. 339, National Research Council, Transportation Research Board, Washington, D.C., September 1991.

INTERCHANGE OPERATIONS: SOFTWARE EVALUATION TECHNIQUES IN USE

Hobih Chen, Viggen Corporation; Henry Lieu and Alberto Santiago, FHWA

Introduction

Serving as the interface point between two intersecting facilities (freeway/freeway, freeway/arterial, or arterial/arterial), an interchange provides an environment that allows vehicles to perform weaving and merging maneuvers safely and smoothly when they move from one facility to another. Since the weaving/merging activities cause disturbance to the traffic flow, a poorly designed and/or operated interchange can easily become a traffic bottleneck. In spite of its importance, interchange planning and operational analysis have received much less attention than other components of the freeway system, judging from the methodologies and analysis models available. The formation of the Interchange Subcommittee under the Transportation Research Board Committee on Highway Capacity and Quality of Service is a first step in the right direction to address this issue and recognize the need for developing methodologies and modelling tools for analyzing the unique operating and performance characteristics of interchanges.

The objective of this paper is to review the existing software evaluation techniques for interchange operations analysis, to identify unique issues related to interchanges and their immediate operating environment, and to provide recommendations for future developments.

Interchange Analysis

The analysis of interchanges can be categorized into two types: planning/design analysis and operational analysis. Planning/design analysis is conducted when designing a new interchange or converting an existing interchange from one form to another. The analysis usually starts with the study of the warrant for a new interchange, followed by the selection of an appropriate interchange design from various alternatives. In general, the selection is based on at least the following considerations (*1,4*):

- Geometry: right-of-way, turning angle, open pavement area, street widths, drainage, lighting, etc.;
- Traffic Demand: on- and off-ramp traffic patterns;
- Safety: sight distance, clearance, allowable travel speed, traffic conflicts, etc.;
- Structures: bridge designs, span lengths, retaining walls; and
- Construction cost and user benefit, including environmental factors.

On the other hand, typical purposes of an operational analysis are to determine whether an existing interchange meets current or future traffic demand, and to improve its operation through signal optimization or minor geometric changes such as restriping to add a left-turn bay, etc. Unlike the planning/design analysis, the operational analysis is mainly concerned with traffic operations and capacity aspects of the interchange, which are also the focus of this paper.

Survey of Current Practice

To date, there are many computerized traffic models available for evaluating freeway or surface street networks. Since interchanges are composed of interconnecting freeway and/or surface street sections, current practice generally is to evaluate each component separately: freeway models for the freeway section, and surface street models for the cross street. There are some software models that can evaluate integrated systems consisting of both freeway and surface street networks. However, the purpose of these integrated models is to include the on-ramp and off-ramp traffic into consideration, not to examine the performance of the interchange itself. None of the models are designed especially to address the complex traffic signal timing and vehicular operations at interchanges, such as weaving/merging/diverging maneuvers.

As part of this study, a survey was conducted to get a glimpse of the type of software models that have been used in evaluating traffic operations at interchanges or intersections near freeway junctions. Questionnaires were sent to members of the TRB Freeway Operations Committee and of the new Interchange Subcommittee of the TRB Highway Capacity and Quality of Service Committee. Twenty-four (*24*) responses were received, the results of which are summarized in Figures 1 and 2.

Seven software models were listed in the questionnaire: HCS, PASSER-II, PASSER-III, TRAF-NETSIM, TEXAS, TRANSYT-7F, and INTRAS. All of which are developed either by the Federal Highway Administration or the states, though basic development of the TRANSYT model was by the British Transport and Road Research Laboratory. Each member was asked to select the models used for interchange analysis as well as the name of any other tools that they have used. Except for two members who have no experience with any software tool, all members specified more than one model. As can be concluded from Figure 1, popular software models such as TRANSYT, HCS, PASSER, and NETSIM are still the most frequently used tools in evaluating interchanges. The "Other" category covers those models that received less than three votes (i.e., INTRAS, FRESIM, FREQ, TEXAS, etc.)

In addition to specifying the models used, each member was also asked to identify the specific analysis that the models were applied to. The following ten areas of analysis were identified according to the survey data:

- capacity,
- interchange type,
- intersection spacing,
- progression,
- queue analysis,
- signal timing,
- spillback,
- ramp metering,
- weaving on cross roads, and
- weaving on freeways.

Figure 2 shows the number of members who selected each of these specific areas. Of these, capacity, weaving, and progression on surface streets appear to be the most popular areas of interest.

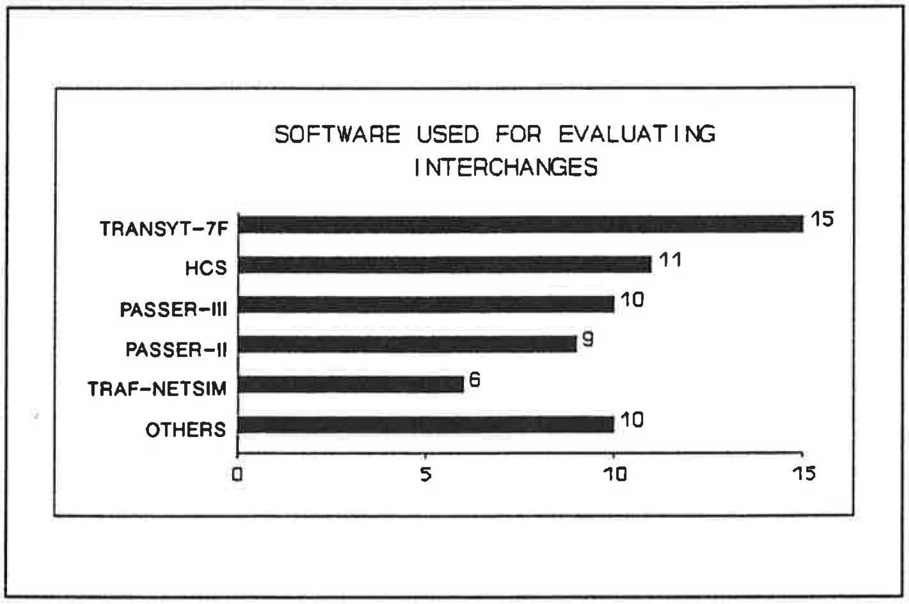


FIGURE 1 Software used for evaluating interchange performance.

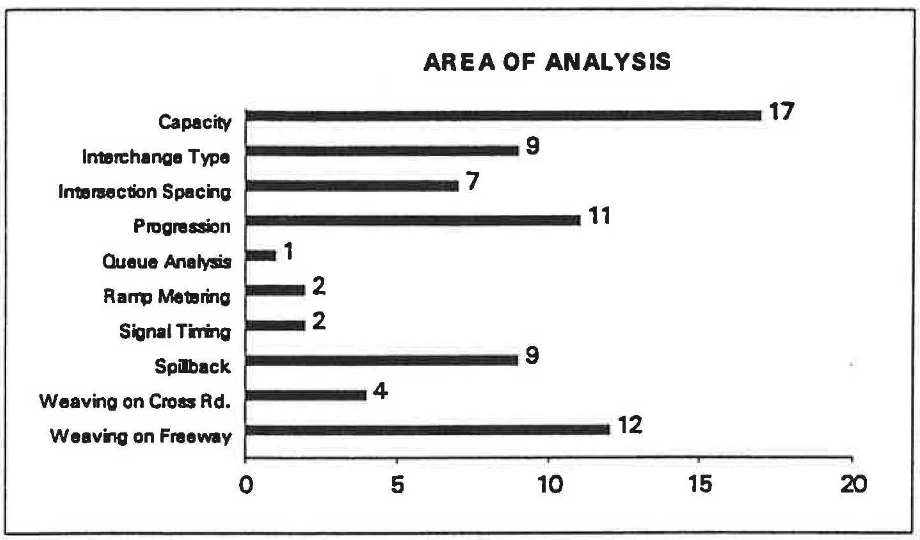


FIGURE 2 Area of analysis.

Software Requirements for Interchange Evaluation

In order to evaluate properly traffic performance at interchanges, the computerized model should be capable of dealing with the unique and complex characteristics. These major characteristics can be identified as:

1. *Weaving Maneuvers*: Complex and heavy weaving, merging, and diverging maneuvers are the unique characteristics of traffic operations at the interchange area. Regardless of the type of interchange, weaving occurs on the freeway when on-ramp vehicles try to merge into the mainline traffic or when freeway vehicles try to move to off-ramps. For clover and directional type interchanges, weaving occurs on the surface street when on- or off-ramp vehicles interact with surface street through traffic. Weaving on arterials to access crossing arterials can occur at all interchange types. The disturbance effect of these vehicular operations can significantly hamper capacity and safety on the surface street.

2. *Closely-Spaced Intersections at Diamond Interchanges and Vicinity*: The unbalanced traffic pattern caused by the lack of a through movement for both off-ramps, coupled with the short distance between the ramp/arterial intersections, results in a unique traffic operation that cannot be treated simply as a regular two-node surface street network. These two closely-spaced intersections require coordinated operation using a special three or four-phase signal timing plan, in order to achieve maximal throughput and ensure safety. Often adjacent intersections on the arterial are closely spaced as well near all interchange types, not just diamonds.

3. *Spillback*: Another critical design element for closely spaced interchange and arterial intersections is the vehicle storage capacity for the traffic movements on the arterial. Because of the close spacing, spillback can occur very easily if the signal timing plan is not compatible with the vehicle storage areas. Spillback disrupts traffic flows, degrades performance, and could easily cause gridlock along a significant portion of the network. Spillback can also occur onto the freeway as the result of excessive queues forming on off-ramps, or onto the arterial as the result of queues occurring on on-ramps due to their inability to enter the freeway.

4. *Ramp Metering*: In order to maintain smooth traffic flows on the freeway, many agencies have started implementing ramp metering control, both pretimed and actuated, on freeway on-ramps to regulate the surface traffic entering the freeway. The analysis software should be able to model this ramp metering control and address the effects of ramp metering on interchange performance.

Computer Models For Interchange Analysis

The main objective of this paper is to make the user aware of the availability of computer models which are suitable for analyzing traffic operations at interchanges. Use of these computer software tools will assist engineers in developing and evaluating alternative improvement scenarios at interchange areas.

Traffic analysis software currently being used by traffic engineers generally can be categorized according to the aspect of traffic operations with which they deal or the manner in which they model traffic operations: *analytical*, *optimization*, and *simulation*. It should be noted that these categories can overlap. A general review was performed to identify existing computer models that could be applied to interchange analysis either directly or indirectly. The review focused on public domain software models developed by the FHWA or the states. Those models identified are described below.

Analytical Models

Analytical models are the software implementations of analytical equations or procedures. The purpose of developing such models is to automate the analysis process, thus minimizing human errors and improving efficiency. The Highway Capacity Software (HCS), a computerized form of the standard 1985 *Highway Capacity Manual* (HCM; TRB Special Report 209), is the only model that fits into this category.

The HCS was developed in modules with each module corresponding to a chapter in the HCM. In addition to the signalized and unsignalized intersection analysis which can be applied to the ramp/arterial intersections, there are two modules which can be applied to interchange analysis: Chapter 4 for weaving areas, and Chapter 5 for ramps. The weaving analysis procedure was developed for freeways, but can be used with caution to approximate the effects of weaving on surface streets. The methodology relates the level of service in weaving areas to the average running speeds of weaving and non-weaving vehicles. The parameters used in the capacity analysis include total volumes, weaving volumes, length of weaving area, and total number of lanes at weaving areas.

For ramp capacity analysis, the methodology establishes the ramp capacity as a function of merge volume for on-ramps, diverge volume for off-ramps, freeway volume, and ramp configurations. The analysis also takes into consideration the upstream and downstream volumes and distance between ramps. Because ramps are analyzed independently of other highway components, the effects of spillback from ramps into freeways or surface streets can not be modelled by the HCS.

Optimization Models

Optimization models are used to determine the "best" signal timing plan for the *signalized* intersections in an urban network. In general, the models reach their decisions by achieving certain system objectives such as maximizing progression or minimizing delay. The signal timing plan is optimized by determining the best combination of timing parameters such as cycle length, split, offset, and phase sequence within the constraints of the optimization criteria (i.e., progression, delay, etc.). The networks that can be optimized include isolated intersections, linear networks (arterial), and grid networks. Four optimization models, SOAP, TRANSYT-7F, PASSER-II, and PASSER-III are reviewed in this paper.

Signal Optimization Analysis Program (SOAP) is a signal optimization and evaluation program which determines optimal phasing and timing for any isolated intersection based on analytic formulas of operations. It also allows the user to analyze existing or pre-determined timing and to evaluate a wide range of intersection signal design alternatives. The intersection can be controlled by either a pretimed or an actuated controller. The input data required by SOAP include geometric configurations, signal timing data, traffic volumes, headway, and capacities. The program can also generate Measures of Effectiveness (MOE's) such as traffic delays, stops, fuel consumptions, and left-turn conflicts.

TRAffic Network StudY Tool (TRANSYT) 7F is a macroscopic network simulation and optimization model that represents traffic flows in the network as histograms over small time increments. It can determine signal timing (cycle, split and offset) for a coordinated network of up to 50 intersections, both signalized and unsignalized, based on a user defined "performance index" (typically a weighted sum of stops and delays). Traffic control is fixed-time, two to seven phases (including pedestrian movements) with fixed sequential phasing, though actuated operation can be approximated. Priority lanes may be designated for buses. A TRANSYT network is structured on a link-node basis. There are 23 input card types available to describe the network configurations, traffic data, signal timing, and parameters controlling the optimization process.

The major outputs produced by TRANSYT are:

- Performance table generated for each intersection and the entire network. The table shows link volumes, saturation flow, degree of saturation, total travel time, delay time, stops, fuel consumption, maximum back of queue, and green times;
- An optimized signal timing table with splits and offset;
- Flow profiles graphically showing the arrival and departure flow patterns; and
- Time-space diagrams and performance measures for any number of routes desired.

Progression Analysis and Signal System Evaluation Routine (PASSER) II is designed to determine optimum progression (maximum bandwidth) along an arterial street considering various multi-phase sequences. The program can handle arterials with up to twenty intersections. The signal timing parameters that can be optimized include cycle length, split, offset, phase sequence (for the arterial), and progression speed.

Basic inputs required by PASSER-II include turning volumes, saturation flow rates, and minimum green times for each movement at every intersection, distances between intersections, average link speed, queue clearance intervals, and permissible phasing sequence. The program produces optimum cycle length, bandwidth, average speed, and signal timing (phase sequence, split, offset), plus performance measures generated from analytic relationships (degree of saturation, delay, stops and fuel consumption). It also generates a time-space diagram showing the progression bands for both outbound and inbound directions, with optimum progression speed.

PASSER-III is an extended version of PASSER-II designed to assist traffic engineers in analyzing fixed-sequence signalized diamond interchanges, pretimed or actuated. Different phasing patterns are permitted including all combinations of "leading" and "lagging" greens, plus the commonly used "4-phase with overlap" pattern. The program is designed to evaluate the interchange performance under existing traffic and signal conditions or to optimize the interchange performance by calculating the best phase sequences, green splits, offsets, and cycle lengths. The effects of queue build-up between interchange intersections are considered explicitly. In addition, the program can evaluate the effectiveness of various geometric design alternatives, e.g., lane configurations, U-turn lanes, and channelization.

The data required for interchange analysis include geometric descriptions, desired phasing pattern(s), cycle length, overlap, queue capacities, movement volumes, and capacities. Outputs are optimal timing designs, similar MOE's as described in PASSER-II, and time-space diagrams.

Simulation Models

Simulation models provide a safe and cost-effective way to evaluate various traffic improvement scenarios before the actual implementation. In terms of design, simulation models can be macroscopic (which represent traffic in aggregate bunches or platoons), or microscopic (which process each vehicle individually). Microscopic models, though requiring more computing time and resources to run, can represent vehicles more realistically than the macroscopic models. Microscopic models theoretically are more responsive to different traffic strategies and can also produce more accurate MOE's and provide enough flexibility to test various combinations of supply and demand. Macroscopic models, on the other hand, often do not have the sensitivity and resolution required to study detailed traffic and geometric changes associated with interchange design. For example, it could be very difficult to use a macroscopic model to evaluate the spillback effect within a diamond interchange when the intersection spacing is reduced, say from 450' to 400'. In this paper, only the microscopic models will be reviewed.

Traffic EXperimental and Analytical Simulation (TEXAS) is designed to perform detailed evaluations of traffic performance at single, isolated intersections. Vehicle and driver characteristics are all treated stochastically in the program. The model is useful in evaluating the effects of roadway changes, changes in driver and vehicle characteristics, intersection control, lane channelization, and operational effects of signal timing plans. The latest version of TEXAS has been enhanced to include new features specifically for diamond interchange analysis.

The data requirements include detailed intersection geometrics, traffic patterns, volumes, signal timing, and vehicle and driver characteristics. The output includes intersection performance MOE's, vehicle interaction MOE's, and animated graphics files for reviewing simulation results pictorially.

TRAF-NETSIM is one of the component models in the TRAF system. NETSIM is an interval-scanning microscopic simulation model for surface street networks. The traffic stream is modelled explicitly according to car-following theory; each vehicle on the network is treated as an identifiable entity. This approach allows the program to simulate the detailed, vehicle-specific traffic processes so that most conditions experienced on an urban traffic environment can be realistically described. As far as microscopic traffic simulation, TRAF-NETSIM constitutes the state-of-the-art.

An extended version of TRAF-NETSIM currently is being developed for the FHWA. The objective is to include some of the latest developments in traffic signal controller functions and to expand the capabilities of the model such that vehicular movements within intersections and grade-separated interchanges can be simulated. A safety-related MOE, traffic conflicts, is being developed as well. Traffic conflicts occur when a vehicle is forced to take some action (alter its speed, trajectory) to avoid a collision with another vehicle. These new features should be helpful particularly in analyzing the weaving and merging of traffic at interchanges and will be able to simulate traffic operations for general types of interchanges ranging from simple underpass/overpass interchanges to complicated ones.

Two graphics postprocessors, ANETG and SNETG, which allow users to view the TRAF-NETSIM simulation results pictorially, are also being revised accordingly. The revised graphics software will allow traffic engineers to review and evaluate traffic performance within intersections and grade-separated interchanges through various graphics displays, both static and animated.

INtegrated TRAffic Simulation (INTRAS) is a vehicle-specific, interval scanning simulation program designed to represent realistically traffic and traffic control in a freeway and surrounding surface street environment. Although INTRAS has been developed mainly for use in studying freeway incident detection and control strategies, it is an ideal tool for studying urban corridors. The surface street model in INTRAS is patterned after the logic of an early version of the NETSIM simulation model (UTCS-1) and allows the user to study the interaction between surface streets with either other surface streets or freeway ramps. Provision is made for the modular inclusion and referencing of specially coded subroutines to model traffic responsive signal control. Ramp metering and freeway traffic diversion procedures are also included.

INTRAS is being enhanced currently by FHWA to allow the model to change the signal timing plans and/or the ramp metering rates between sub-intervals and to add the capability of simulating areawide traffic-responsive ramp metering schemes, with or without queue override features. With these new features, users will be able to evaluate traffic performance at interchanges, both isolated and integrated, by simulating a variety of traffic operations and signal coordination strategies.

Summary of Software Capabilities

The capabilities of the eight models reviewed above are summarized in Table 1 in terms of the analysis areas to which they can be applied.

Except for "*Interchange Selection*," which is a critical element in the planning/design analysis, and perhaps "*Intersection Spacing*," analysis areas identified in the table are components of the operational analysis. In other words, most of the capabilities offered by the existing software models are geared to operational analysis rather than planning/design analysis. The following areas deserve further discussion.

Interchange Selection

In their current form, the application of software models in this area is limited to evaluating the traffic operation

TABLE 1 SOFTWARE MODEL CAPABILITIES

	HCS	SOAP	T7F[1]	PSR-2[2]	PSR-3[3]	TEXAS	NETSIM[4]	INTRAS
Ramp Capacity Analysis	X							
Interchange Selection			X			X	X	
Intersection Spacing			X	X	X	X	X	X
Progression				X	X			
Ramp Metering								X
Signal Timing Optimization		X	X	X	X			
Queue Analysis			X		X	X	X	X
Spillback			X		X	X	X	X
Weaving on Crossroads	X						X	
Weaving on Freeways	X							

Note: [1]TRANSYT-7F [2]PASSER-II [3]PASSER-III [4]TRAF-NETSIM

aspects of various interchange designs. None of the models are comprehensive enough to evaluate geometric properties such as sight distance, right-of-way or cost. Because TRAF-NETSIM, TRANSYT-7F, PASSER II and SOAP can generate fuel statistics, their output can be used to quantify user benefits when conducting benefit-cost analysis. TRAF-NETSIM also can provide emission statistics.

Considerable care should be taken when selecting appropriate software models to determine the type of interchange most applicable to the conditions being studied. For example, Leisch, et al. (*2*), compared the operational characteristics between two interchange forms: the single-point urban interchange (SPUI), and compressed diamond interchange (CDI) using TRANSYT-7F. The results showed that CDI's are more efficient than the SPUI's except for the case when both left turns on the cross streets are heavy and balanced. However, the CDI received a much less enthusiastic endorsement from one recent study. One reason could be that queues in the TRANSYT-7F model are "vertical." That is, the adverse effects of spillback from left-turn bays or from short through lanes within the interchange area are not represented properly by TRANSYT-7F in terms of, for instance, effects on upstream saturation flow rates. On the other hand, microscopic simulation models such as TRAF-NETSIM normally have car-following behavior algorithms built in that will decrease the upstream saturation flow rate in recognition of standing or slow moving downstream queues.

Ramp Metering

The capability of modelling ramp metering control is important when studying spillback from the ramps onto the surface street system. Even though INTRAS is the only model that currently has this feature, FHWA is integrating NETSIM and FRESIM, a microscopic freeway simulation model in the TRAF family, to give users this capability in a much more realistic fashion.

Weaving on Crossroads

Both the HCS and TRAF-NETSIM can be used to analyze the weaving behavior on the crossroad in a limited way. The HCS has a chapter on freeway weaving which can be extended to weaving analysis on crossroads. However, further study is needed to determine the appropriateness of this application. TRAF-NETSIM can simulate weaving, to some extent, by treating the weaving points as yield sign-controlled intersections. The extended version of TRAF-NETSIM that FHWA is developing currently will have weaving modelling built in; this new version will treat the interchange area as an integrated area. In addition to modelling traffic interactions, the model will allow users to specify the origin-destination pattern for every traffic movement through the area, such that the amount of weaving should be represented reasonably. The ability to simulate an interchange with graphic display capabilities should make the new TRAF-NETSIM a comprehensive model.

Summary/Recommendations

Existing software evaluation techniques for interchange operations analysis were reviewed and summarized in this paper. The results of a survey were also reported. There is no doubt that interchange analysis is important, yet the availability of good computerized models is limited; features offered by existing models are confined to certain analysis areas.

As part of the survey mentioned above, members of the TRB Freeway Operations Committee and Interchange Subcommittee were asked to identify the desirable features that should be provided, but are currently missing, from existing models. Their responses are summarized below.

1. Establish a methodology for the integrated capacity analysis of interchanges. The methodology should allow traffic engineers to compare different interchange forms. The analysis should yield a level of service index which reflects the geometry and traffic of the interchange area as a whole. Once the methodology is developed, an analytical software model, similar to HCS, will follow naturally.

2. For diamond interchanges, guidelines should be developed to relate the spacing between the two intersections to the traffic demand and turning patterns. Such guidelines will help traffic engineers to evaluate the performance of diamond interchanges, and in selecting between the CDI and SPUI forms. TRAF-NETSIM is one tool to help determine the relationship between traffic demand and the storage capacity between intersections.

3. Develop a comprehensive model for aiding the planning/design and interchange selection process. It is desirable to have a software model that can evaluate interchange geometric properties in addition to the traffic operations aspects of interchanges. Even though models like INTRAS or FRESIM allow users to enter superelevation and pavement friction, and the new NETSIM will generate safety-related MOE's, more features are needed in order to evaluate geometric properties satisfactorily. Ideally, the traffic engineer should be able to design the interchange and then use a performance model to evaluate the design in an interactive and user-friendly manner.

References

1. "Design and Operational Features of the Single Point Urban Interchange," Utah Department of Transportation, UDOT-MR-91-003, Final Report, June 1991.

2. Leisch, J., Urbanik, T., and Oxley, J., "A Comparison of Two Diamond Interchange Forms in Urban Areas," *ITE Journal*, May 1989.

3. Fambro, D.B., Bonneson, J.A., "Application Guide for the Microcomputer Version of PASSER III-88,", Report No. FHWA/TX-90/478-2F, Texas State Dept. of Highways & Public Transportation, Transportation Division, Austin, TX., April 1990.

4. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*. Washington, D.C., AASHTO, 1990.

5. Transportation Research Board, Highway Capacity Manual, TRB Special Report 209. Washington, D.C., TRB, 1985.

DISCUSSION

Jim Powell, Session Moderator

This paper has presented a good overview of the software available for evaluating interchange operations and desirable extensions to this software. The authors have touched on most of the critical issues including: 1) the need to distinguish planning/design analysis from operational analysis; 2) operational complications such as arterial weaving and spillback; and 3) the alternate types of software tools—analytic, optimization and simulation.

One important aspect that should be discussed further has to do with use of microscopic simulation models such as NETSIM. Such models have powerful capabilities that capture many real world interactions. At the same time, a potential user needs to understand basic concepts of Monte Carlo simulation, the stochastic nature of modelled processes and the variability of the results.

A program such as NETSIM utilizes statistical distributions to model key aspects of traffic behavior, for example, driver type, and thus characterize such things as aggressiveness, car-following logic and lane changing behavior. Other distributions such as headway and deceleration profiles are used similarly by NETSIM. Inherently assumed is that the underlying distribution (e.g., mean, variance and shape) is well known for a given characteristic. That is, it has been verified in original model development, or has been input by the user based on observed field data. This is important when considering, for example, weaving behavior on arterials near interchanges, because weaving is dependent on complex interactions among a variety of vehicle and driver types under varying geometric and traffic control conditions.

As an example most of the NETSIM model was calibrated originally around 1973 using data primarily from Washington, D.C., and its ability to replicate real world conditions was very good for peak hour conditions, but not as good under less disciplined off-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.

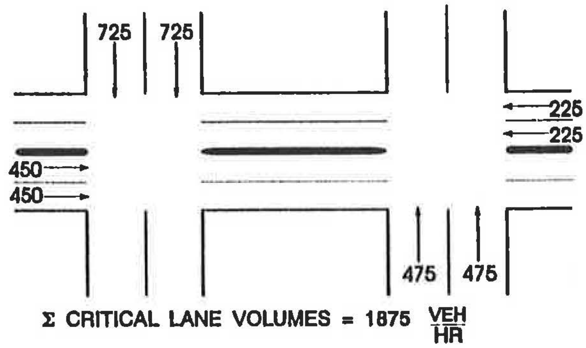


FIGURE 1 Capelle/Pinnell example critical lane volumes.

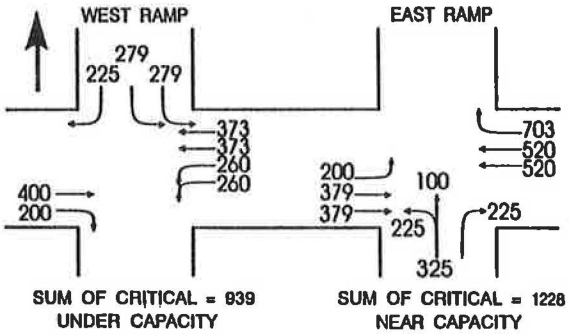


FIGURE 2 2010 projected pm peak hour volumes example analysis: planning method.

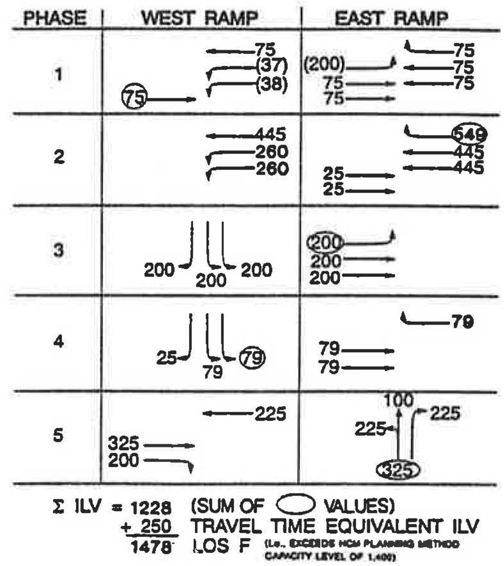


FIGURE 3 2010 projected pm peak hour volumes examples analysis: Church method.

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.

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

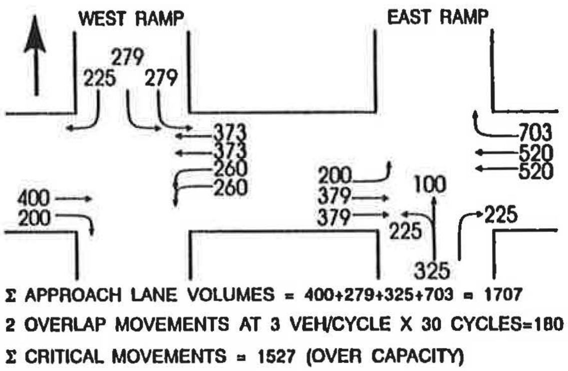


FIGURE 4 2010 projected pm peak hour volumes example analysis: Lee method.

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

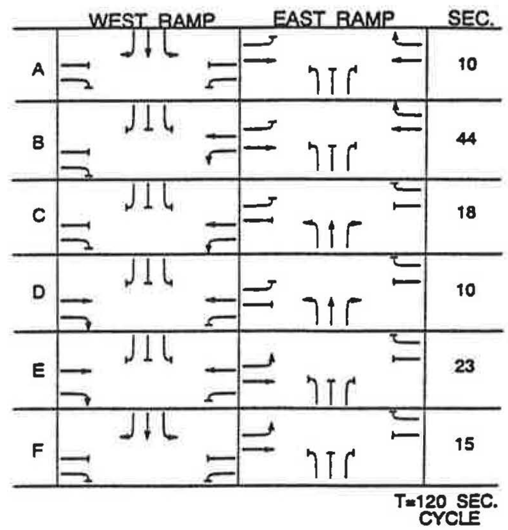


FIGURE 5 Four phase with overlap phasing used in hook modified operational analysis.

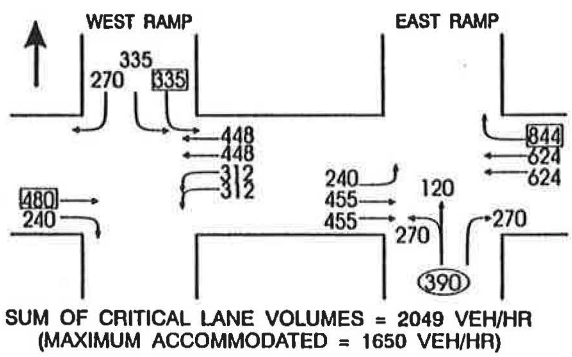


FIGURE 6 2010 projected pm peak hour volumes example analysis: Capelle and Pinnell method.

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.

References

1. Donald G. Capelle and Charles Pinnell. "Capacity Study of Signalized Diamond Interchanges". In *Highway Research Bulletin 291*, HRB, National Research Council, Washington, D.C., 1961, pp. 20-31.

2. David J.P. Hook and Jonathan Upchurch. "Comparison of Operational Parameters for Conventional Diamond Interchanges and Single Point Diamond Interchanges". Paper presented at 71st Annual Meeting, TRB, Washington, D.C., January, 1992.

3. *Highway Capacity Manual*, Special Report 209, Transportation Research Board, National Research Council, Washington, D.C., 1985.

4. *Operations Analysis, A Self-Study Manual, Volume 1 - Intersection Analysis,* Draft Report, Highway Operations Branch, Caltrans-District 4, San Francisco, CA, August 10, 1990.

5. *Operations Analysis, A Self-Study Manual, Volume 2* - Interchange Analysis, Draft Report, Highway Operations Branch, Caltrans-District 4, San Francisco, CA, October 25, 1991.

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.

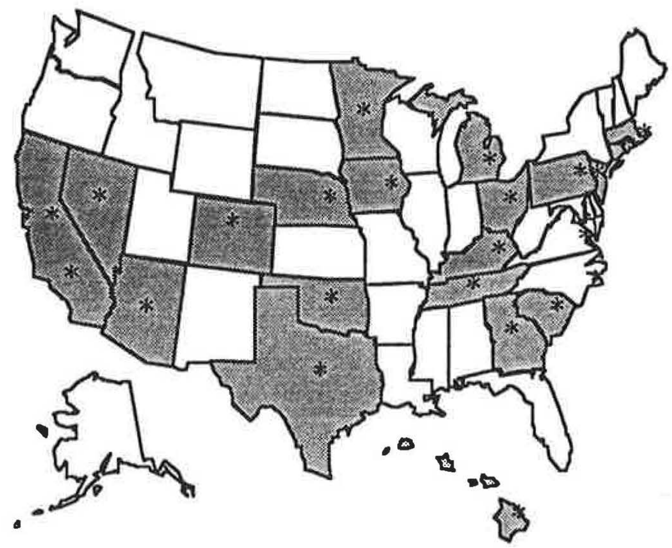


FIGURE 1 Distribution of questionnaire and responses.

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.

It should be noted that a large number of states (*16*) indicated use of directional interchanges, despite the fact that directionals are normally system interchanges, not service interchanges that were the survey focus. Apparently respondents did not make this distinction in completing both Questions #1 and #3.

Of the interchanges currently in operation, the most "common" types (see Question #1) appear as follows:

TABLE 2: MOST COMMON INTERCHANGE TYPES IN USE

TYPE	NO. OF RESPONSES
Conventional diamond	19
Partial cloverleaf	16
Tight diamond	8
Full cloverleaf	6
Directional	4
Trumpet	3
Split diamond	1
Three level diamond	0
Other	0

Thus the two most common types appear to be conventional diamond, followed by partial cloverleaf. Tight diamond occupies a distant third place. It comes as no surprise that these types also dominate what is considered for new construction or reconstruction as indicated by the following responses (Question #3):

TABLE 3: INTERCHANGE TYPES CONSIDERED FOR NEW CONSTRUCTION OR RECONSTRUCTION

TYPE	NO. OF RESPONSES
Conventional diamond	22
Partial cloverleaf	17
Directional	14
Tight diamond	12
Single point diamond	10
Split diamond	10
Trumpet	10
Full cloverleaf	9
Three level diamond	3
Semi-directional	2
Button hooks	1
Diamond-type ramps	1
Partial turban	1
Diamond with flyover ramps	1

Conventional diamond leads the candidate list in new construction or reconstruction followed by partial cloverleaf. Directional interchanges appear to have gained popularity in the new construction/ reconstruction, but probably only when a system interchange is under consideration, as discussed following Table 1. Single point diamond interchanges, as expected, are also frequently being considered. Split diamond and trumpet are being considered about as frequently as the tight diamond or full cloverleaf. It is of note that the full cloverleaf has dropped a good deal from future consideration, especially compared to the list of interchanges in use (i.e., from third place in Table 1 to eighth place in Table 3). This point will be discussed further.

The factors that are considered in the selection of interchange type for new construction/reconstruction are listed as follows (Question #4):

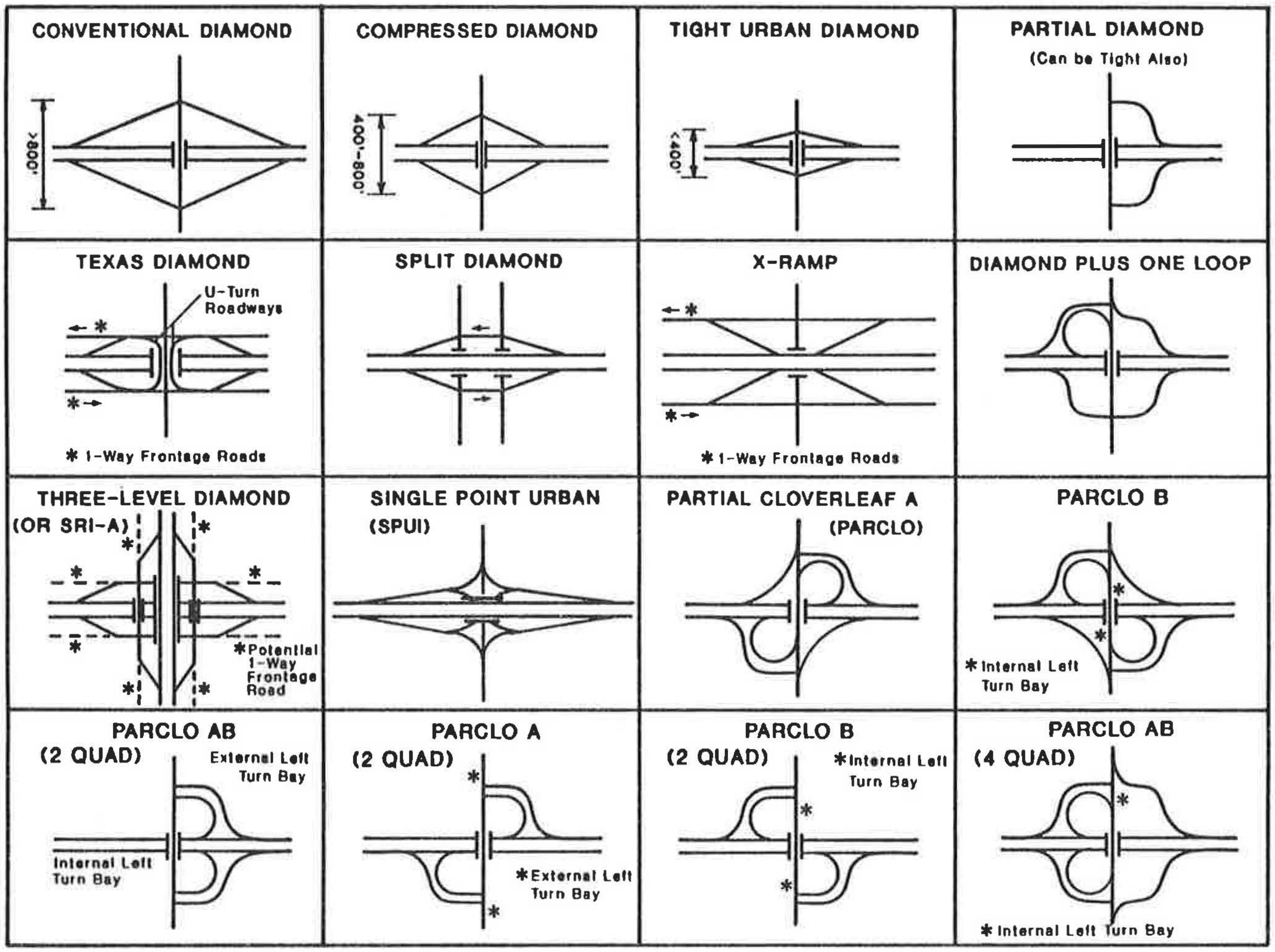
TABLE 4: FACTORS CONSIDERED IN SELECTING INTERCHANGE TYPE

FACTORS	NO. OF RESPONSES
ROW	24
Environmental/Socioeconomic	24
Operations	23
Cost	23
Topography	21
Design Features	20
Constructability	20
Other	5

As expected, a large number of factors weigh about equally in the choice of interchange type.

Operational Experience

The literature provides little information on the subject of interchanges, particularly with respect to operational experience on the local street side. This survey deals with basic aspects of interchanges such as configuration, operational characteristics and evaluation techniques to aid in design and construction of key elements in future projects. It is important to define operational experience by looking at the interchange area comprehensively with the surrounding street system. Almost every interchange is unique and all are influenced by external factors. Traffic engineers appear to favor providing the motorist as consistent a driving experience as possible in terms of



NOTE: ADAPTED FROM MICHIGAN DOT & AASHTO

FIGURE 2 Basic interchange types *(continued on next page).*

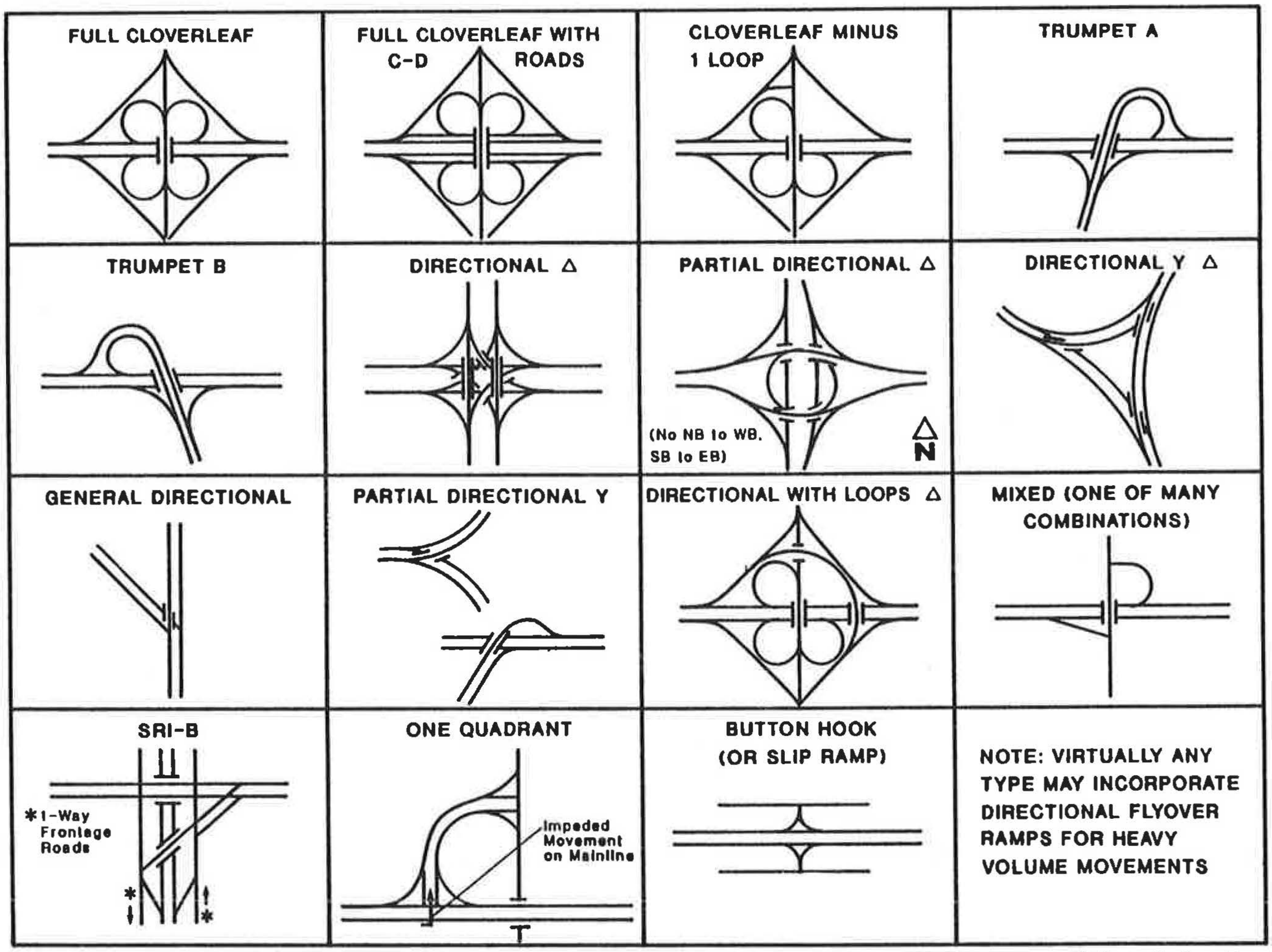
configuration to meet driver expectancy. There also appears to be an effort to eliminate weaving or at least provide sufficient lengths to accommodate it. Operational experience of commonly used interchange types is described below, based on summaries of the qualitative and descriptive portions of Questions One and Two. The responses are grouped by interchange type.

It should be noted that this survey predated the description of distinct types of diamond interchanges discussed earlier in this Circular. The definitions used here are not consistent with that discussion, rather the discussion below generally distinguishes between a tight diamond with very closely spaced ramps (under 250 ft.) and diamonds with wider spacing. These definitions did not seem to cause confusion among survey respondents, though there clearly is a need for uniform terminology when discussing interchanges.

For reference, Figure 2 illustrates a variety of basic interchange types. This figure is based on a graphic provided by the State of Michigan.

Conventional Diamond

Arizona indicates the conventional diamond as the most common type of interchange, which has a high motorist familiarity. California recognizes it as the basic type and favors it because it can be designed for expansion and adding loops generally at a minimum cost. However, it requires good signal synchronization to be flexible with respect to traffic demand. Connecticut lists this as the preferred type. The conventional diamond appears as a standard in Georgia as it meets driver expectancy. It is best used in conjunction with an arterial divided by a median. Problems are reported to occur at high volumes or if driveways and side-streets are in close proximity.



Δ Normally Considered a Systems (Freeway-to-Freeway) Interchange

FIGURE 2 Basic interchange types *(continued)*.

Iowa reports the conventional diamond as the most understood type and economic in design, and considers it efficient and safe unless high volumes exist. Kentucky favors it as the best type for its operations. It is the most common type in Nebraska in rural areas, where the operation is typically unsignalized. Nevada conveys that ninety percent of the interchanges in the state are conventional diamonds. Problems are reported if adjacent intersections are closely spaced or high volumes exist, where it leads to storage and capacity difficulties.

Ohio links its choice of type and configuration of interchange to the volume. For low-to-medium volumes, conventional diamond is the standard. A cloverleaf is favored for medium-to-high volume. However, weave problems are reported with this type at very high volumes, in which case a "super diamond" with extra lanes is the recommended type. Oklahoma indicates the conventional diamond is the most common and the most understood interchange. At some locations there are on/off ramps to/from two-way frontage roads, where problems have been reported. Backups have occurred onto freeways sometimes.

South Carolina recommends the conventional diamond as the most common type. If there are heavy left turns at the location, double left turn lanes are generally used. Tennessee, in rural areas, uses 600 ft to 1000 ft spacing between ramps. Loops are added instead of signalization if heavy turning traffic arises. Drivers appear to understand the operation well and few problems are reported.

Based on the responses, the following common features emerge for the conventional diamond:

• Most understood by drivers regarding proper lane positioning;
• Most efficient and safest design, and usually has minimum construction cost;
• Provides low level of service at high volume locations, but it is adaptable to a wide range of traffic volumes, sometimes by adding loops to make a partial cloverleaf; and
• Restricted left turn storage length on the cross road sometimes may create a problem, adversely affecting ramp operation. Possible remedies include double left turn lanes or full length (ramp–to–ramp, side–by–side) left turn lanes. Traffic operations improve if cross road is divided by a median.

Overall, the conventional diamond appears to be the basic design against which to compare other types. Diamonds seems to enable provision of a high standard of alignment and treatment of turning maneuvers, and they are also seen as adaptable to a wide range of traffic volumes.

Tight Diamond

California uses tight diamond interchanges in locations which are heavily developed and/or if the freeway is either elevated or depressed, but feels it provides the least capacity and sometimes has storage and sight distance problems. The sight distance problems can occur due to the presence of support piers. Storage and capacity problems are also reported by Minnesota and spacing problems are reported by Kentucky.

Pennsylvania reports that intersections at tight diamonds are usually unsignalized, where sight distance problems have been reported sometimes. When used with signalized intersections, storage problems have been noted.

The respondents' experience with the tight diamond are summarized as follows:

• May be appropriate if right–of–way is greatly constrained;
• There can be storage and capacity problems for both the left turns and the through movements when traffic volumes are high. Back–to–back left turn lanes can be troublesome; and
• The presence of crest vertical curves and bridge rail can sometimes limit sight distance.

Overall, the impression left by respondents is that the tight diamond is limited by basic capacity and storage constraints. It is worthwhile to note that the recent research and discussion on interchange types (*Single Point Urban Interchange Design and Operations Analysis*, by C.J. Messer, J.A. Bonneson, S.D. Anderson and W.F. McFarland, NCHRP 345, 1991) states that as long as good signal timing is employed, the capacity of a tight (urban) diamond interchange should be similar to that of a single point urban interchange (pp. 47-48). Apparently, many practicing engineers are unaware of the signal timing concepts appropriate to tight diamonds embodied, for example, in the PASSER III program (see Bibliography, References *10*, *16*, *18* and *34*). An important requirement for tight diamonds is that the signals be timed to assure no internal stacking and spillback problems. Two or three basic phase plans, depending on volume conditions, can be used to coordinate signal operations to meet the requirement.

Regarding the broader question of capacity of tight diamonds relative to single point diamonds, the literature unfortunately has been confused by inappropriate before–and–after comparisons. Frequently, basic lane capacity has been added to an after case along with a change in form, making it nearly impossible to compare interchange forms alone. Research Problem Statement 1 in this Circular should provide tools to address the question fully.

Single Point Diamond (Urban) Interchange (or SPUI)

Arizona reports single point diamond interchanges becoming commonplace, particularly in the Phoenix area. Single points are favored for handling heavy volumes with a minimum right-of-way. No storage problems are reported and it appears to provide good capacity with a minimum number of signal phases. Kentucky appears to have had success with their use so far, and more of this type are being built. Kentucky also reports requiring minimum right of way and no storage problems.

Tennessee reports increased construction of single points lately, experiencing good capacity and storage relative to tight diamonds. The design also appears flexible to changing or time-varying traffic demands. Tennessee's experience also indicates that it costs less in right–of–way dollars than tight diamonds, although construction costs are often higher.

Colorado indicates favorable experience with a small number of single points, observing good delay and capacity performance. The state notes that following an "initiation" period, drivers have adapted well, and likes the fact that the number of signals is reduced. Nebraska also is experimenting with them and feels positive with their first such design.

The following observations based on the responses can be made about the single point diamond interchange:

• May be appropriate if right–of–way is greatly constrained;

• Accommodates large turning movements (with large left turning radii); see Bibliography, Reference 9 for further discussion;

• Eliminates left turn storage within the interchange, and reduces the number of signals or signal phases; and

• May increase the capacity of the cross road.

Partial Cloverleaf (Parclo)

It can be noted that the survey did not distinguish between Parclos types. Responses covered a range of Parclos illustrated on Figure 2 and discussed in Sevice Interchange Section of this Chapter.

California finds the partial cloverleaf utilizing a loop on-ramp (Parclo A) provides the best local service. The Parclo provides high capacity and is sometimes used with unsignalized intersections in California. However, free-flow right onto arterials have sometimes been eliminated due to pedestrian concerns. Connecticut reports that in their use of (2 quad) Parclo B's, drivers sometimes are confused by on/off ramps at the point of entry from the arterial (see Figure 3). Hawaii also favors Parclo's.

Georgia finds the application very useful for dealing with heavy left turn volumes, though it does result in occasional weaving problems. This design is also sometimes required due to topographic constraints (e.g., an immediately adjacent river or railroad). Iowa also uses a Parclo when right-of-way requirements or topography suggest it and it is also a favored design where high left turn volumes are encountered. Iowa indicates that placement of signs is sometimes made more complex due to right side and left side entrances onto the freeway, and recommends avoiding weaving sections.

Nebraska uses a Parclo A where topography or right-of-way requires it. Driver expectancy on the freeway is felt to be violated with a Parclo B, apparently due to concern with two separate egress ramps. New Jersey uses partial and full cloverleafs exclusively, especially on intersections along major arterials or expressways. To avoid left turns at signalized intersections between two arterials, New Jersey often uses a "jug handle" (converting all turns to right turn maneuvers). There are weave concerns, including weaves involving nearby driveways on arterials. Oklahoma reports that in their use of partial cloverleafs, they find weave problems to be a function of signal spacing.

South Carolina finds that the Parclo can help eliminate some problems associated with heavy left turns, but suggests avoiding the Parclo AB due to arterial weaving. Tennessee favors avoidance of weaves and also reports driver confusion on half clovers similar to Connecticut's experience noted earlier.

Colorado notes the fact that Parclo intersections typically operate with simple two-phase signalization. The state is currently converting several diamond and full cloverleaf interchanges to partial cloverleafs.

The following characteristics appear to emerge regarding the Parclo:

• The Parclo provides a good design when right-of-way is restricted in one or two quadrants or when left turn volume has a significant impact on the operation of the basic diamond intersections;

• Weaving sections can be created by adjacent loops. These should be avoided, or provided adequate length to complete the maneuver; and

• Motorists' confusion can occur if the on ramps and off ramps meet at one point on the cross road (i.e., in a 2 quad configuration) and are not properly channelized. At some locations, signing can be somewhat complex if drivers are unfamiliar with the layout.

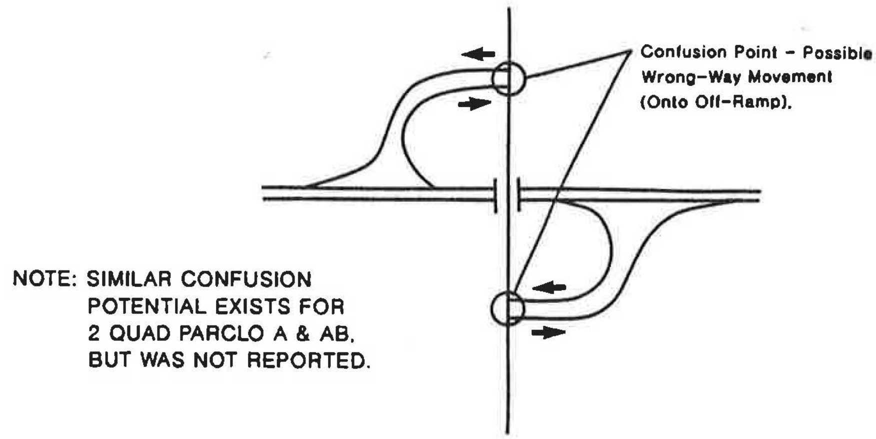
The at-grade confusion problem is discussed earlier in this Circular, where it is indicated that 2 quad Parclo A's and B's are considered in new designs only where topography or right-of-way require them.

Full Cloverleaf

Both California and South Carolina report weaving problems associated with a full cloverleaf. California's experience indicates that it can handle high volumes, however, sideswipe and rear end accidents have been reported. Also problems are noted for left turns at the downstream intersection due to the need to weave across arterial through lanes. Iowa would not use a full cloverleaf unless a collector-distributor road is provided, and does not see it as a local service interchange.

In the same vein, Kentucky is eliminating use of the full cloverleaf because of weaving problems. Massachusetts similarly is eliminating its use due to merge conflicts. Nebraska reports that increasing traffic volumes lead to capacity and weaving problems. As noted previously, New Jersey uses full and partial cloverleafs exclusively, especially at intersections between major arterials and expressways. However, concerns remain about weaving, including with nearby driveways on arterials. Tennessee also observes that weave problems exist unless a collector-distributor road is provided. Colorado considers the full cloverleaf a "dinosaur."

2 Quad Parclo B



Standard Parclo B (4 Quad) Eliminates Problem (where Topography/ROW permit):

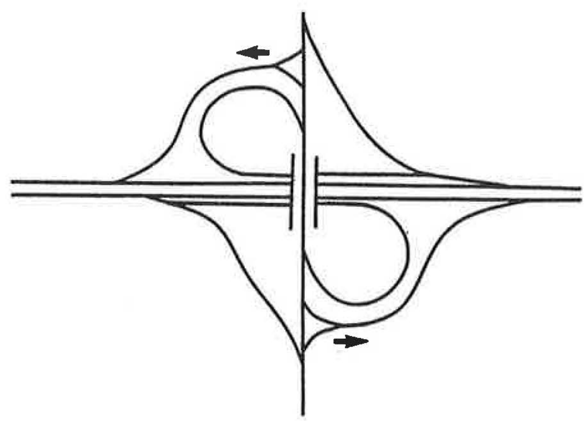


FIGURE 3 Reported parclo confusion.

Some general observations about the full cloverleaf are as follows:

- Maximum right-of-way is typically required;
- A full cloverleaf often presents weaving problems on the freeway, local street or both, particularly during peak hours, thus reducing capacity; and
- It is a good practice with this form to include collector-distributor roads along the freeway to reduce the weaving problem.

Trumpet

Arizona uses trumpet type interchanges in rural areas. Hawaii indicates that their engineers favor the directional features of the trumpet.

Other

Arizona reports that it occasionally uses turban (a form of directional) interchanges. Georgia also uses the directional type for freeway-to-freeway interchanges. A flyover is sometimes used on ramps to/from a major arterial.

General Comments/Responses

Georgia reports that weaving cross-over problems exist if traffic needs to make a left turn at nearby downstream intersections after exit maneuvers. Minnesota similarly indicates that closely spaced intersections cause problems. Minnesota is also interested in ramp metering implications and high occupancy vehicle (HOV) lane accommodations, such as by-pass lanes. Texas favors a consistency in design and provision of adequate lengths for weaves and merges. Virginia reports one case of a single point diamond with three other closely spaced (300' to 650') signalized intersections that works very well.

Analysis Techniques and Concerns

Question #5 elicited information on how operations were considered in selecting a new interchange type or evaluating an existing one. A summary of the techniques used for analysis/evaluation is given as follows:

TABLE 5: OPERATIONAL ANALYSIS METHODS

ANALYSIS TECHNIQUE	NO. OF RESPONSES
HCM	23
Software Programs	13
Critical lane	12
Other	6

More specific information was compiled from the responses to determine what software is being used in analysis or evaluation, as follows:

TABLE 6: OPERATIONAL ANALYSIS SOFTWARE

ANALYSIS TECHNIQUE	NO. OF RESPONSES
Highway Capacity Software (HCS)	21
Passer II & III	4
TRAF-NETSIM	1
TRANSYT-7F	1

It would appear that the Highway Capacity Software is being used primarily. It is interesting to compare these results with those of Figure 1 in the Chen, Lieu and Santiago paper earlier in this Circular chapter. There, TRANSYT-7F came out as the most common software for evaluating interchanges. The differences probably lie in the survey groups; the Chen survey queried TRB committee members (Freeway Operations and the Interchange Subcommittee) who tend to be research oriented, while this survey covered practicing state engineers.

In addition to the occasional use of other software, the following analysis concerns or techniques were mentioned: driver expectancy, weaving, merging, and lane balance; storage length (specifically using 1967 Jack Leisch technique); and benefit versus cost analysis.

In response to Question #7 asking if interchange operations were a major area of research need, only five reported in the affirmative. The wording of this question (asking to identify persons to contact for funded research) may have discouraged a more positive response.

TABLE 7: MAJOR RESEARCH NEED ON INTERCHANGES?

RESPONSES	NO. OF RESPONSES
Yes	5
No	9
Did not respond	10

Overview

It is appropriate to combine some of the survey planning/operations responses with our own observations regarding interchange planning, design and operations, as follows.

Major items that need to be addressed in interchange planning include:

- Travel demands: volumes, origin/destination patterns, vehicle classifications (with differing vehicle acceleration/deceleration characteristics), and pedestrian considerations;
- Driver characteristics: work load/stress, gap acceptance behavior, merging and weaving behavior;
- Functional characteristics: laneage (e.g., free right versus dual right turn; dual left turn provision), signage and pavement markings; and
- Signal timing and operational MOE's: delay and spillback, and their relationship to signal optimization.

These items should be considered together with the many design and environmental aspects of interchanges.

Specific performance characteristics that should be considered in evaluation include:

- Capacity and level of service:
  Ability to handle changing travel demands,
  Number of lanes and distance before lane drop,
  Signalized vs. unsignalized operation, and
  Signal coordination and resulting throughput along a series of closely spaced intersections (ramps and adjacent intersections);

- Safety:
  Driver expectancy,
  Signing, and
  Sight distance;
- Storage and spacing:
  Ramp intersection spacing and spacing to adjacent intersections and
  Access control along the cross road; and
- Weaving:
  Between freeway off ramp and on ramp, along the arterial and
  Between off ramp and next downstream intersection on the arterial, to make a turn.

Evaluation may be a part of initial planning for a new interchange, or it may relate to review or problem identification for an existing interchange. Many of these items cannot be considered separately from design and cost issues.

For the future, current trends and recent developments suggest areas for additional research:

- Ramp metering accommodation, with potential interconnection to interchange signals;
- HOV accommodation (e.g., by-pass lane treatment), to service transit needs; and
- Integration with advanced traffic control concepts associated with Intelligent Vehicle Highway Systems (IVHS).

Overall the topic of service interchange operations is an evolving field in which a good deal has been learned, but a good deal more needs to be understood and put into practice.

Acknowledgements

Assistance provided by Khaled Mudarres & Kostaman Thayib, both graduate students at Portland State University, in the preparation of this paper is greatly appreciated.

APPENDIX A: SURVEY FORM

Interchange Subcommittee
Committee on Highway Capacity and Quality of Service
Committee A3A10 Transportation Research Board

The Interchange Subcommittee is undertaking to document the types of interchanges currently in operation and learn from the experience that has been gained from their performance. The ultimate objective is to develop procedures for the determination of the capacity of interchanges similar to other highway elements as embodied in the Highway Capacity Manual. While we define the research objectives, state the scope of work and identify funds for any research, we expect to build on existing experience. This questionnaire is brief with a view to limiting your time and effort in completing it, yet it may call for certain coordinating effort on your part within your organization. Any effort you spend will be a very useful contribution to the advancement of knowledge in this area.

Interchanges *here are meant to be the at-grade junction of a through roadway, usually a freeway, with a lower classification road and can be either signalized or unsignalized. Arterial/arterial interchanges are also included. The operational focus is on the lower classification road and its intersections with the on/off ramps, and not on the merge/diverge features of the through roadway.*

1. What type of interchanges are currently in operation for your agency? Please check types below, and put an asterisk next to the three most common types.

☐ Conventional Diamond
(Ramp spacing greater than 250 ft.)
☐ Tight Diamond
(Ramp spacing less than 250 ft.)
☐ Split Diamond
☐ Partial Cloverleaf (One, two or three quadrant)
☐ Full cloverleaf
☐ Trumpet
☐ Three level Diamond
☐ Directional
☐ Other (Please list):

On the back of this sheet, please briefly summarize your operational experience (that is, operations on the lower classification road) with the three most common types in use. If you so choose, please feel free to describe your operational experience with types other than the three most common in use.

2. For the types of interchanges in use, are there any design or operational characteristics that you feel are unique? Please briefly summarize and supply related sketches, figures or photographs.

____
____
____
____
____

3. What interchange types are you considering for any new construction or reconstruction?

☐ Conventional Diamond
(Ramp spacing greater than 250 ft.)
☐ Tight Diamond
(Ramp spacing less than 250 ft.)
☐ Split Diamond
☐ Partial Cloverleaf (One, two or three quadrant)
☐ Full cloverleaf
☐ Trumpet
☐ Three level Diamond
☐ Directional
☐ Other (Please list):

4. In interchange construction or reconstruction, what factors do you consider in selection of type? Check those that apply.

☐ Right–of–Way
☐ Cost
☐ Operations
☐ Design features (for example, design speed)
☐ Constructability
(for example, disruption/construction difficulty)
☐ Topography
☐ Environmental/Socio-Economic Impacts
☐ Other (Please list):

Do you have a procedure for considering and trading off the factors? If so, please briefly describe (or supply a sample application).

____
____
____
____
____
____

5. How do you consider operations in selecting a new interchange type or evaluating existing conditions? Please check off the analysis techniques you use.

☐ Critical lane
☐ Highway Capacity Manual
(with or without modifications to account for interchange characteristics)
☐ Software programs (Please list):
☐ Other (Please list):

If possible, include document(s) that outline your approach. We are particularly interested in special considerations related to interchanges, for example, spill back between ramps or onto the freeway, signal timing, weaving on the arterial or interactions with adjacent intersections.

6. State the three most important aspects of interchange operations (from your point of view) that we need to understand and analyze better.

____
____
____
____
____
____

7. Does your agency consider interchange operations a major area of concern justifying new research attention?

____

If yes, please supply the name/title/phone number of person or persons we should contact regarding the possibility of funded research or contributed manpower.

____
____
____
____
____
____

8. Please provide any further comments regarding interchanges.

____
____
____
____
____

9. Name/address/telephone no. of person(s) responding to this survey.

(Note: This is a condensed version of the survey.)

DISCUSSION: NEEDED RESEARCH

Following the presentations, each of the presenters was asked to identify the most critical research needs in both short-range and long-range perspectives. Following is a summary of their comments.

Dan Fambro identified six areas of needed research or clarification, including:

- Establishment of criteria for level of service determination; most likely more than one measure will be needed, i.e., more than delay only;
- The potential need for other metrics besides, or in addition to, level of service when comparing or evaluating alternative interchange forms. In other words, a complete framework that includes level of service, safety, cost and environmental factors at a minimum, when considering interchange alternatives;
- The need to better define and validate storage ratio as a level of service measure, as used in the PASSER III program;
- Better understanding and characterization of weaving on arterials near interchanges;
- More thorough research into progression effects at interchanges, since with limited platoon dispersion, progression/timing effects will be at a peak; and
- Guidance to analysts regarding values of lost time and saturation flow rate for use at various interchange forms.

Hobih Chen took a somewhat different tack:

- The need to look at adaptive signal control technology for use at closely-spaced interchange intersections; FHWA is conducting research to develop self-calibrating strategies and algorithms that will optimize performance in a traffic responsive mode. These may be useful at interchanges.

Jim Lee suggested the following:

- The need for a comprehensive capacity analysis technique for diamond interchanges and
- Better methods for comparing and evaluating basic interchange forms; he mentioned the tendency in Arizona to regard the single point urban interchange as the best in all circumstances, when this may not be the case. Economic analysis should be an important part of the evaluation technique.

B. Kent Lall mentioned some of the same ideas as Dan Fambro and Jim Lee:

- Focus on performance and level of service measures, though in a set order of priority, i.e., perhaps conventional diamonds first, parclos second, etc., and then on to less common interchange types.

RESEARCH PROGRAM

FUNDED RESEARCH

Based on the previous materials of Chapter I-IV, the Interchange Subcommittee has identified initial research needs and submitted research problem statements to the full Committee on Highway Capacity and Quality of Service. Subsequently in 1992, a research problem statement entitled "Capacity Analysis Techniques for Interchange Ramp Termini" was selected for funding by the National Cooperative Highway Research Program (NCHRP), to begin in FY 1994. Following is an outline of the research, which will probably run for a two to three year period. One of the more critical elements in the highway system is the interchange. The interchange can be characterized by numerous conflict points, high traffic demands, and a high percentage of turning traffic. Conflicts among the higher volume movements are typically grade separated while lower volume movements are served at-grade. To maximize major road operations, many maneuvers that tend to generate conflicts and delays (such as stopping, turning and weaving) are designed to occur on the minor road. Unfortunately, the relatively close spacing of the ramp termini combined with the high volume of interchanging traffic demand tend to cause operational problems such as high delays, poor minor road progression, long queues, and, in some cases, queue spillback between adjacent ramp intersections.

This research will develop capacity analysis techniques applicable to basic interchange configurations. The techniques will cover only those interchanges having one or two signalized ramp/minor road junctions, such as the single-point urban interchange (SPUI), conventional diamond, tight diamond, Parclo A, Parclo B, and Parclo A-B. The goal should be to describe basic operations and develop level of service measures similar to those for signalized intersections in the HCM, though with explicit consideration of spillback.

The research will be accomplished through the following tasks: (1) Literature Review—Major related research has been completed or is underway and will first be reviewed (e.g., PASSER III model, current Australian Road Research Board work on "paired intersections" and current New Jersey research on arterial weaving). Traffic flow models, weaving/merging behavior and level of service measures are a few of the review targets. Weaving should be considered just downstream and upstream of the interchange intersections, as well as between them. Coordination with other HCM procedures and IVHS technologies should be included also. (2) Data Collection—Field data will be collected utilizing video taping to encompass delay, lane utilization/weaving/merging and platoon behavior. Though focused on capacity, these data could later be used for interchange safety studies also. (3) Model Development and Validation—Based on Tasks 1 and 2, develop and validate a model of interchange operations using a combination of theoretical and empirical modeling. Identify an appropriate measure of level of service. Accuracy and ease of application should be goals, with due consideration to computer modeling techniques. Validate and refine the model based on the Task 2 database. (4) Documentation and Dissemination—Following final review and testing by selected users, document the techniques and incorporate text and software procedures into the HCM.

ADDITIONAL RESEARCH NEEDS

The initial research will address basic capacity and performance, as outlined above but it is anticipated that further research needs will evolve from the initial effort. One area already identified is the need for signal timing methods applicable to a range of interchange forms and configurations. The methods should incorporate not only the interchange signals, but adjacent closely spaced intersections as well, with which timing must also be coordinated.

Following is a general problem statement addressing this issue. The effort should not begin until the basic capacity research for ramp termini is well underway.

Problem Statement

At present, optimization of signal timing at interchanges is only partially understood, with the result that timing procedures are somewhat limited and operations are not as efficient as they could be. Since heavy traffic volumes typically turn and change travel paths at interchanges, even small timing in efficiencies can result in large delay and stop penalties. Conceptually, the problem is more complicated than traditional progression optimization strategies in which there are typically one or two primary flows with linear travel paths. At interchanges, there can easily be three or even four nearly equal major flows, some of which will have a turn maneuver (e.g., left off a ramp) in the travel path.

Another frequent characteristic is the presence of one or more adjacent closely spaced intersections for which spillback is a major operational and safety concern. Basic operational performance (e.g., saturation flow rate and capacity) can be a function of spillback, yet few if any optimization packages today acknowledge this fact. Spillback also can affect mainline operations on the through roadway, due to off ramp queues. Similarly, ramp metering may cause spillback from on ramps into the ramp terminals, but most ramp metering systems do not recognize or adjust for this possibility.

Interchanges are critical to freeway corridor efficiency and safety. At these major nodes, minimizing delay should be a primary goal of IVHS technology, yet systematic timing techniques are lacking. Similarly, IVHS diversion strategies around congestion will rely on optimized signal control, say, for example, it is known that an interchange is about to receive a 40% increase in off-ramp volume, due to closure or restriction at an adjacent off ramp. Quick response in timing adjustments will be a key to success in such a case.

Objective

The main objective of this research should be to develop a comprehensive model to optimize interchange area signal operations, based on combined evaluation of stops, delays, spillback potential and progression opportunities. The problem area should encompass the interchange traffic signal(s) plus up to two closely spaced adjacent intersections (for a system maximum of six signals). Ideally, evaluation should use a weighted sum of the performance measures, with relative weights being variable by the user. In addition, ramp meter timing should be incorporated in recognition of the mutual impacts between mainline and interchange operations.

Because of the complexity of the problem, it is envisioned that software techniques will be the ultimate result. However, the emphasis in this research should be on understanding mechanisms and behavior, and then formulating algorithms that account for essential characteristics. Examples are weaving effects and upstream saturation flow rates when facing a downstream queue. Although at least prototype optimization software should be developed, final software might best be left to programmers who could incorporate the results in specific traffic signal controllers and systems.

Scope of Work

Following is a prospective work program.

Phase 1, State-of-the-Art Review and Data Collection
Conduct a comprehensive literature review/appraisal. Included should be on-going research on ramp termini capacity (NCHRP 3-47) and on "paired intersections" by the Australian Road Research Board (R. Akcelik), PASSER III research (Texas Transportation Institute) and TRANSYT-7F work (University of Florida), plus strategic/tactical work completed under FHWA contract DTFH61-89-C-00006, "Coordinated Operation of Ramp Metering and Adjacent Traffic Signal Control Systems" (Turner-Fairbank Center). Parallel developments on the Advanced Traffic Controller (ATC) should also be examined. While assessing the state-of-the-art, pertinent knowledge gaps need to be identified.

Field data collection should cover a variety of interchange operating conditions and traffic control scenarios, to characterize techniques in practice and to compile a good base of "before" data. Both video tape data and comprehensive time/delay study data should be covered. Time/delay studies should cover major traffic movements through the interchange area, and not just through movements on the arterial roadway. Included should be selected interviews with practicing engineers, since there likely are empirical techniques that work, even if practitioners only intuitively understand why.

Phase 2, Optimization Tool Development
Based on Phase 1, develop the underlying mechanisms and algorithms to optimize interchange area intersections. Optimization should include cycle length, phase sequence, splits and offsets for each of one to six intersections in and adjacent to the interchange. As discussed in the above Objective section, alternate performance index formulations should be built in, and the evaluation portion should reflect the unique interchange environment (high turning volumes, closely spaced intersections). Ramp metering interactions should be included, though the goal is not to model merge behavior on the major facility in detail. The model should capture essential traffic flow characteristics and quickly identify the best combination of timing parameters, recognizing potential use in IVHS.

For Phase 1 study sites, conduct new "before" studies as needed, apply the model to several cases, implement the timings and conduct "after" field studies. Based on the results, refine and further validate the model.

Phase 3, Documentation and Dissemination
Document the data sources, algorithms and model format developed by the project. Included should be procedures that users should always undertake, at least initially, to calibrate to local drivers and driving conditions. After testing through selected users, make written documentation and initial software available through public sources. The results should provide a firm basis for developing a fully operational control package, for immediate or longer term (i.e., IVHS) use.