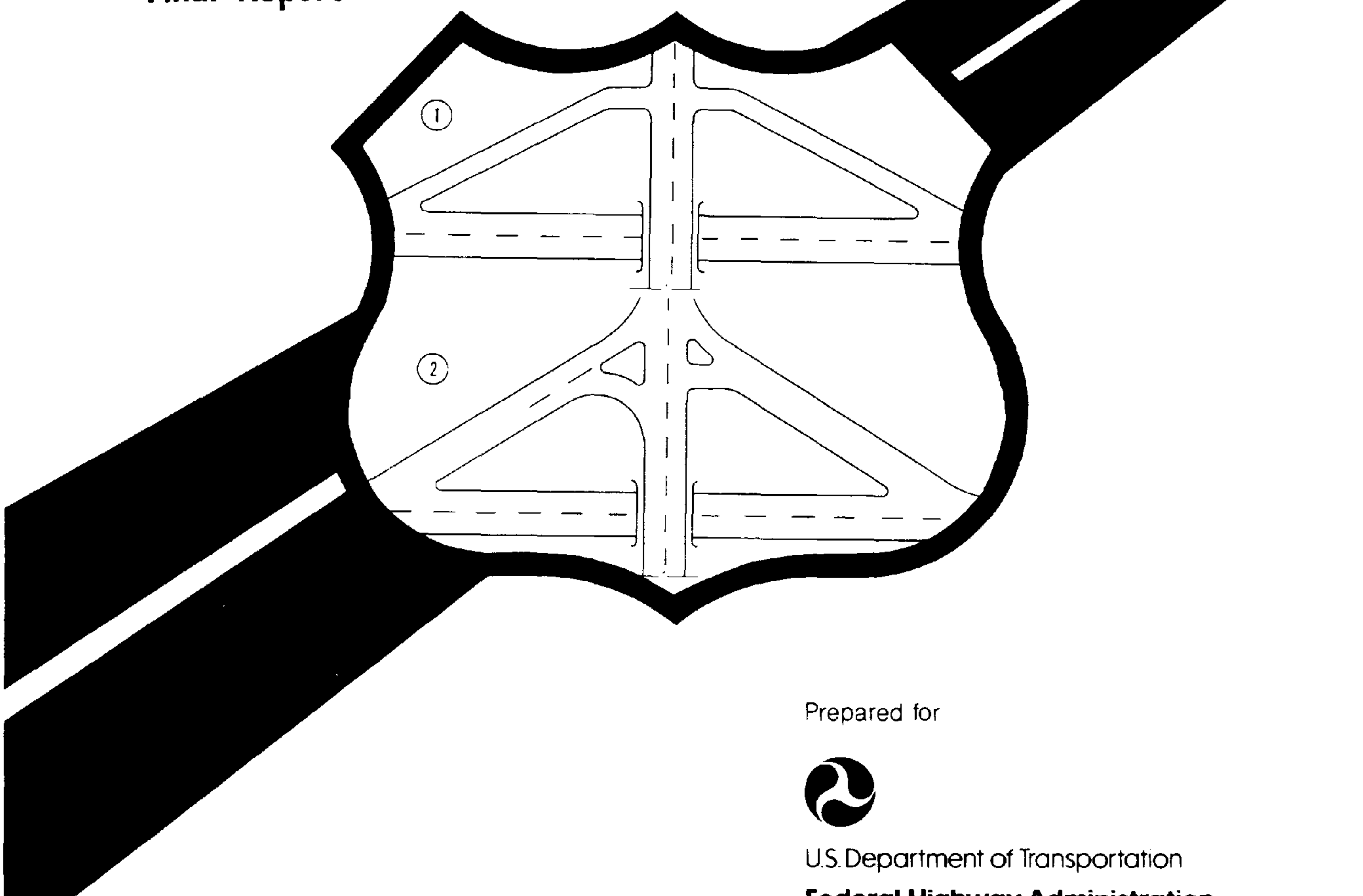
Report No. FHWA/RD-81/104

PROCEDURES AND GUIDELINES FOR REHABILITATION OF EXISTING FREEWAY-ARTERIAL HIGHWAY INTERCHANGES

Vol. II. Design Procedures for Rehabilitation of Freeway-Arterial Interchanges

May 1982

Final Report



Prepared for

U.S. Department of Transportation

Federal Highway Administration

Offices of Research & Development
Environmental Division
Washington, D.C. 20590

Document is available to the U.S. public through the National Technical Information Service Springfield, Virginia 22161

FOREWORD

This report summarizes the results of a research study which developed an interchange rehabilitation project design procedure which includes identification of safety or operational problems and identification and quantification of alternative improvement strategies. The report will be useful to highway design and traffic engineers concerned with rehabilitation of freeway-arterial highway interchanges.

The report presents the findings of Contract DOT-FH-11-9318, "Procedures and Guidelines for Rehabilitation of Existing Freeway-Arterial Highway Interchanges." The study is being conducted for the Environmental Division, Office of Research, Federal Highway Administration as part of Project 1J, "Improved Geometric Design," of the Federally Coordinated Program of Research and Development.

Sufficient copies of this report are being distributed to provide a minimum of one copy to each FHWA Regional Office, one to each FHWA division office and one to each State highway agency. A limited number of copies are available for official use from the Safety Design Group, Environmental Division, FHWA, HRS-43, Washington, D. C. 20590.

D. Solomon
for Charles F. Scheffey
Director, Office of Research
Federal Highway Administration

1. Report No. FHWA/RD-81/104	2. Government Accession No.	3. Recipient's Catalog No. PB83 190884
4. Title and Subtitle Procedures and Guidelines for Rehabilitation of Existing Freeway-Arterial Highway Interchanges Volume II--Design Procedures for Rehabilitation of Freeway-Arterial Interchanges		5. Report Date May 1982
		6. Performing Organization Code
7. Author(s) D. W. Harwood, J. C. Glennon and J. L. Graham		8. Performing Organization Report No. 4440-S
9. Performing Organization Name and Address Midwest Research Institute 425 Volker Boulevard Kansas City, Missouri 64110		10. Work Unit No. (TRAIS) FCP 31J4032
		11. Contract or Grant No. DOT-FH-11-9318
12. Sponsoring Agency Name and Address Federal Highway Administration Office of Research (HRS-43) Environmental Division Washington, D.C. 20590		13. Type of Report and Period Covered Final Report September 1977-November 1980
		14. Sponsoring Agency Code T-0515

15. Supplementary Notes

FHWA Contract Manager; George B. Pilkington II (HRS-43)

16. Abstract

This volume presents design procedures that can be used to improve existing interchanges with traffic operational or safety problems. The procedures are intended primarily for application to freeway-arterial interchanges located in urban and suburban areas. However, similar procedures would be appropriate for freeway-freeway interchanges and for interchanges located in rural areas. The report presents procedures for each step in the interchange rehabilitation process including identification of interchanges with traffic operational or safety problems; identification of specific deficiencies through engineering studies; identification of improvement alternatives; quantification of the effects of improvement alternatives; evaluation of alternatives and selection of the best; implementation of the improvement project; and, evaluation of the improvement project.

This volume is the second in a series. The others in the series are:

Volume Number	Report Number	Short Title
I	FHWA/RD-81/103	Executive Summary
III	FHWA/RD-81/105	Interchange Evaluations
IV	FHWA/RD-81/106	Research Report

17. Key Words Interchange, Design, Freeway, Traffic operations, Rehabilitation, Traffic safety, Reconstruction, Economic analysis	18. Distribution Statement No restrictions. Document is available to the public through the National Technical Information Service (NTIS), Springfield, Virginia 22161. There is a charge for copies ordered from NTIS.

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	171	

PREFACE

Efforts throughout this project were guided by a steering committee of design and traffic operations engineers from State highway agencies and consulting firms throughout the U.S. The members of this committee were Mr. Andrew J. Gazda, Illinois Department of Transportation; Mr. C. William Gray, Ohio Department of Transportation; Mr. Parker Hall, California Department of Transportation; Mr. Ronald E. Magahey and Mr. Aage G. Schroder III, Florida Department of Transportation; and Mr. Bernard Rottinghaus, Howard, Needles, Tammen and Bergendoff. The overall guidance and specific suggestions provided by this committee have contributed immeasurably to the results of this project. We also acknowledge the efforts of Mr. Kenneth E. Robertson, Michigan Department of Transportation, and Mr. Alan D. Kenyon, New York State Department of Transportation, who coordinated the data collection activities in their respective States. Finally, we wish to express our gratitude to the many individuals in each transportation agency, at both headquarters and local levels, who assisted in the data collection efforts.

This is the second volume of the four-volume final report. This volume presents recommended design procedures for interchange rehabilitation projects. Each step in the interchange rehabilitation process is reviewed and suggested procedures are presented. The other volumes of this set are Volume I, an Executive Summary of the other three volumes; Volume III, which presents evaluations of 40 interchange projects recently constructed by State highway agencies; and Volume IV, the research report, which provides an overview of all activities during the contract.

TABLE OF CONTENTS

TABLE OF CONTENTS (Continued)

List of Tables

List of Tables (Continued)

List of Tables (Concluded)

List of Figures

List of Figures (Continued)

List of Figures (Concluded)

I. INTRODUCTION

During the past 40 years, the United States has constructed an extensive system of multilane, limited-access freeways. The complete exclusion of driveways and at-grade intersections has had an important role in making freeways the safest and most efficient portion of the American highway system. Freeway-arterial interchanges have been constructed at intervals to provide access from the conventional highway system to the freeway system.

The importance of freeway-arterial interchanges as an element in our highway system cannot be overstated. Each freeway trip begins and ends at a freeway-arterial interchange. Because of the conflicting demands of entering, exiting and through traffic, most operational and safety problems of the freeway system are concentrated at interchanges. Operational and safety problems on the arterial street system are also common at or near freeway ramp terminals.

Redesign of inadequate interchanges can result in increased capacity, reduced delay and increased safety. However, the cost of remedy all existing operational and safety problems already exceeds the funds available to highway agencies for improvements and improvement needs are sure to grow. Therefore, the use of cost-effectiveness techniques is vital to assure that the limited funds available are invested optimally.

Because interchange rehabilitation is expected to play an important role in improving the traffic operations and safety of our highway system, Midwest Research Institute (MRI) has performed a study for the Federal Highway Administration entitled, "Procedures and Guidelines for Rehabilitation of Existing Freeway-Arterial Interchanges," under Contract No. DOT-FH-11-9318. The overall objective of the contract was to develop cost, safety and operationally effective geometric design procedures and guidelines by quantifying the effect of cost, safety and operational tradeoffs for the upgrading of existing freeway-arterial highway interchanges. The intent of these procedures and guidelines was to accommodate an increase in traffic volumes, maximize safety benefits and minimize costs.

The scope of the study was limited to consideration of freeway-arterial interchanges in urban and suburban areas. Throughout the study, the term "freeway-arterial interchange" has been interpreted as referring to all interchanges between a freeway and a street or highway with no control or partial control of access. Freeway-freeway interchanges were specifically excluded from the project scope. However, it is recognized that many of the project results are applicable to freeway-freeway interchanges as well as freeway-arterial interchanges, and rural interchanges as well as urban and suburban interchanges.

The recommended design procedures for rehabilitation of freeway-arterial interchanges are presented in this volume and its organization is based on the structure of the interchange rehabilitation process. The next section is a discussion of operational and safety problems at freeway-arterial interchanges. Section III is an overview of the interchange rehabilitation process that identifies each step of the process and indicates the sequential relationship between the steps. Section IV covers the first step in the interchange rehabilitation process, identification of interchanges with operational and safety problems. The investigation and definition of these problems through engineering studies is discussed in Section V and the identification of appropriate alternative solutions is addressed in Section VI. Section VII presents procedures to quantify the effects of the alternative solutions on traffic operations and safety. The consideration of these effects and other factors in selection of the best alternative is discussed in Section VIII; and, finally, Section IX presents procedures to evaluate the operational and safety effectiveness of completed interchange rehabilitation projects.

II. OPERATIONAL AND SAFETY PROBLEMS OF FREEWAY-ARTERIAL INTERCHANGES

Every highway agency that operates a freeway system has a continuing responsibility to identify and eliminate operational and safety problems that develop on the system, including those at freeway interchanges. The development of such problems over time may be inevitable, and highway engineers generally agree that operational and safety problems are often the combined result of three factors: trade-offs made in the original design, unanticipated growth of traffic volumes, and unanticipated changes in traffic patterns.

Improvements in geometric design standards and advances in highway safety technology have been continuous in recent years. However, many freeways and freeway interchanges built before the Interstate Highway System and some built in the early years of that system do not conform to current geometric design policies. Although the replacement of all of these facilities may be desirable, it is not possible within the funding allocations available to highway agencies. Furthermore, many "under-designed" freeways serve current traffic volumes adequately and do not experience high accident rates, so the available funds are often better spent on more critical operational or safety problems at other locations. The existance of these older or under-designed freeways does impose an obligation on highway agencies to monitor traffic operations and safety and to resolve any problems that develop.

The other major causes of operational and safety problems are unanticipated growth of traffic volumes and changes in travel patterns. Freeway interchanges are typically designed to serve traffic projected 20 years in the future, but such projections are, at best, uncertain. Current traffic volumes at many interchanges far exceed their design volumes. Reasons for underestimation of traffic volumes include (1) rapid growth of population and automobile travel within many metropolitan areas, (2) residential and commercial development of new areas on the urban fringe and (3) traffic volume projections based on extensive planned urban freeway systems that are now unlikely to be completed. Quite obviously, an interchange designed to serve a county road on the urban fringe will become inadequate if the surrounding land use changes and the county road is later developed into a suburban arterial street. Interchange problems will continue to develop in the future because traffic volumes in many metropolitan areas are expected to grow rapidly, despite the possible effects of the energy crisis.

The purpose of this report is not to establish design policies for interchanges. Accepted design policies are presented in AASHTO publications including A Policy on Geometric Design of Rural Highways-1965[3] and A Policy on Design of Urban Highways and Arterial Streets-1973.[4] Instead, the report presents an approach toward monitoring the operational and safety performance of interchanges, identifying problem locations and alleviating existing problems.

The recommended procedures are directed primarily toward geometric improvements (rehabilitation) of existing interchanges, although appropriate consideration is given to the role of traffic control and roadside safety improvements. It is recognized that geometric improvements are often constructed in conjunction with traffic control improvements (signalization, signing, etc.). In many cases, geometric improvements are made after low-cost traffic control improvements have failed to eliminate an existing problem. The recommended procedures are intended to apply to both traffic control and geometric improvements and, indeed, to provide a basis for comparison between them.

Many of the recommended procedures are already performed within highway agencies. The purpose of this report is to suggest alternative approaches to some procedures and, most importantly, to indicate an overall structure for the interchange rehabilitation process.

III. OVERVIEW OF INTERCHANGE REHABILITATION PROCESS

This section presents an overview of the interchange rehabilitation process. The objective of the process is to identify and correct traffic operational and safety problems at freeway-arterial interchanges. The six steps in the process are:

- Identify interchanges with operational and/or safety problems;
- Study problem locations and identify specific deficiencies;
- Identify improvement alternatives;
- Quantify effects of improvement alternatives;
- Evaluate alternatives and select the best; and
- Implement improvement and evaluate effectiveness.

Each step of the process is summarized here and discussed in more detail in a subsequent section.

A. Identify Interchanges with Operational and/or Safety Problems

The first step of the interchange rehabilitation process is to identify interchanges that have traffic operational or safety problems that are potentially correctable. The recommended approach is not a formal procedure, but does provide guidance for the use of accident surveillance systems and operational data to prepare a list of candidate interchanges for further study. The operational review procedures rely on field reviews of operational conditions, traffic volume counts, capacity analyses and citizen complaints. Greater emphasis is placed on a formal surveillance system for identifying safety problems than for operational problems, because safety problems are often more subtle and difficult to detect. Recommendations on accident surveillance procedures are based on a review of the systems employed by several state highway agencies.

B. Study Problem Locations and Identify Specific Deficiencies

Engineering studies for identifying specific deficiencies at problem locations are presented. A set of basic studies including physical inventories, on-site observation, traffic volume counting, accident tabulations and summaries, and collision diagrams are recommended for each problem location. Supplementary engineering studies, suitable for investigating specific types of operational and safety problems, are also discussed. The recommendations identify both the objective of each type of engineering studies and the sources that can be consulted for the detailed procedures.

C. Identify Improvement Alternatives

A critical step in the interchange rehabilitation process is the identification of alternative solutions. It is important that all feasible alternative solutions be considered by the engineer, lest the best alternative be missed. A series of charts have been developed to relate identified operational and safety problems to potential solutions. Using the appropriate chart, the designer can quickly identify a set of solutions that are potentially applicable to the particular interchange configuration and problem under consideration. Additional solutions developed by the engineer should also be considered.

D. Quantify Effects of Improvement Alternatives

Procedures are provided to quantify the effects of improvement alternatives on travel time, vehicle operating costs and accidents. Travel time and vehicle operating costs are quantified through the procedures of the AASHTO Manual on User Benefit Analysis for Highway and Bus Transit Improvements-1977,[2] which have been reorganized to specifically address the analysis of interchanges. Safety effectiveness estimates are based on the literature, effectiveness evaluations of interchange rehabilitation projects made during this contract, and engineering judgment. The potential importance of air pollution and noise analyses to some interchange decisions is stressed and it is recognized that other factors, which cannot be quantified, will also influence interchange rehabilitation decisions.

E. Evaluate Alternatives and Select the Best

The guidelines for evaluation of alternatives encourage the use of analytical techniques to compare alternatives, although they recognize that the choice between alternatives rests heavily on the engineer's judgment. The net return method--a conventional engineering economic analysis technique--is recommended to examine trade-offs between factors that can be quantified in monetary terms. Factors included in the net return analysis are: construction costs, travel time (delay) costs, vehicle operating costs, accident costs, and other costs. Non-monetary factors are considered on the basis of engineering judgment. The designer is urged to record the advantages and disadvantages of each alternative design in a formal report or memorandum to fully document the selection of an alternative.

F. Implement Improvement and Evaluate Effectiveness

The final step of the interchange rehabilitation process is to implement the selected improvement project and, subsequently, to evaluate its effectiveness. The objective of an effectiveness evaluation is to compare the actual effects of the project with its predicted effects. Techniques for both operational and safety effectiveness evaluations are suggested. Feedback from the evaluation of completed projects will enable the anticipated effects of planned projects to be more accurately quantified in the future.

IV. IDENTIFY INTERCHANGES WITH OPERATIONAL OR SAFETY PROBLEMS

The first step in the interchange rehabilitation process is to identify existing interchanges with operational and/or safety problems. This step requires a systematic review of the operational and safety performance of all interchanges under the jurisdiction of a highway agency to identify those in need of improvement. The objective of this step is not to establish formal improvement priorities, but rather to select a set of interchanges for further investigation as candidates for improvement.

A. Operational Problems

A variety of operational problems are found at freeway-arterial interchanges. Delays to motorists result when the traffic volume using any element of an interchange approaches or exceeds the capacity of that element of the interchange, even for a short period of time. Delays not only increase travel time for motorists but also increase consumption of our increasingly scarce fuel supplies. Operational problems can occur in any portion of an interchange, although recent experience shows that the most common locations for operational problems are in the vicinity of the crossroad ramp terminals.

1. Measures of operational performance: Two formal measures of operational performance are most commonly used to quantify interchange operational performance: level of service and delay. The following discussion briefly describes these measures and their use in problem identification. Specific procedures for quantifying these measures are provided in Section VII and Appendix B.

The most commonly used analytical procedure for evaluating traffic operations in interchange areas is the 1965 Highway Capacity Manual (HCM).[17] This procedure is used to assess the Level of Service in interchange elements from traffic volume and geometric data. Under this concept, traffic operations are rated on a qualitative scale from Level of Service A, representing free flow conditions, to Level of Service F, representing forced flow conditions. The methods of defining these Levels of Service vary depending on the interchange element under consideration. For example, Levels of Service for uninterrupted flow on mainline freeways and arterial streets are based on both operating speed and volume-to-capacity ratio (V/C), while for interrupted flow on a signalized intersection approach they are based on the "load factor."* Still other definitions apply to

* The proportion of signal phases continuously and completely utilized by traffic.

weaving areas and to merging and to diverging areas at ramp terminals. A 1974 report by Leisch provides expanded procedures for consideration of traffic service on ramps, multiple weaving, etc.

A major effort is currently underway to develop a revised and expanded HCM. The completion of this effort is still several years away, but tentative procedures for analysis of at-grade intersections, freeways and weaving areas have been presented in Transportation Research Circular 212, "Interim Materials on Highway Capacity,"[40] published in January 1980. The Level of Service concept has been retained as the basis for traffic operational analysis, but major changes in the procedures to determine the Level of Service have been proposed. For example, a critical movement analysis technique is suggested to evaluate the Level of Service for an entire at-grade intersection rather than treating each approach separately, as in the 1965 HCM. When the revised HCM is completed and published, its use is recommended. In the meantime, the interim procedures may be used on a trial basis.

Delay measures are another promising approach to assessment of operational performance. Most attempts to quantify delay have addressed the issue of delay at signalized intersections. An analytical model of delay at fixed-time signals was developed by Webster[44] and has been widely used in the development of signal control strategies. The operational analyses of intersection delay presented in Section VII of this report are based on Webster's model. A variation of this model has been applied to the estimation of delay for traffic-actuated signals by Courage and Papapanou.[12] Field procedures for delay measurement at signalized intersections have been recommended by JHK and Associates.[32]

2. Traffic volume counting: An effective traffic volume counting program is needed within a highway agency for several reasons. The basis for most analytical estimates of traffic operational measures, as well as qualitative assessments of operational problems, are the traffic volumes for the site in question. Traffic volumes for the morning and evening peak hours are usually of greatest importance in dealing with operational problems. Average daily traffic (ADT) volumes are needed for analysis of safety problems and for planning purposes.

Every highway agency has a traffic volume counting program to supply the basic data needed for operational evaluations of the freeway system. However, many volume counting programs concentrate on mainline freeway counts and do not count interchange ramps. This is unfortunate because a counting strategy based on ramp volumes can determine mainline volumes as well. Figure 1 illustrates two alternative strategies for counting traffic volumes on freeways. The upper portion of Figure 1 illustrates traffic volume counting in both directions between interchanges on the mainline freeway. The lower portion of Figure 1 illustrates traffic volume counting on each interchange ramp. The mainline traffic volumes at each interchange are adjusted by subtraction of off-ramp volumes and addition of on-ramp volumes. The mainline freeway should be counted every fifth or sixth interchange to provide a check on the derived volumes. While this approach involves approximately twice as many counts, the need to count across all mainline lanes is minimized and the data obtained is of maximum utility to interchange analyses.

Traffic volumes on the arterial crossroad are also critical to many interchange analyses. Unfortunately, crossroad volumes are often unavailable in highway agency files if the crossroad is under the jurisdiction of a local agency.

It is recommended that routine traffic volume counting procedures and records systems include the determination of both ramp and crossroad volumes, especially in urban and suburban areas. At the very least, crossroad volumes should be obtained from local agencies and included in State records systems. Both average daily traffic and morning and evening peak hour volumes should be included as a minimum. On the other hand, turning movement counts at ramp terminals are very costly to conduct and should be done only as needed in response to identified problems.

3. Recommendations for interchange operational surveillance: While every highway agency has developed procedures for identifying safety problems, there is no established procedure to identify operational problems. Due to massive data requirements, it is probably infeasible to perform a formal evaluation of the capacity and Level of Service in both peak hours for every element in every interchange. Fortunately, locations with operational problems are usually obvious to observers in the field, so it is feasible to focus attention on selected interchanges. Several States have developed procedures for "operational inventories" of their freeway system. For example, Missouri has measured travel time, delay and fuel consumption during peak periods using an instrumented vehicle. While such inventories are usually focused on the mainline freeway lanes, they could easily be adapted to identification of ramp and crossroad operational problems as well.

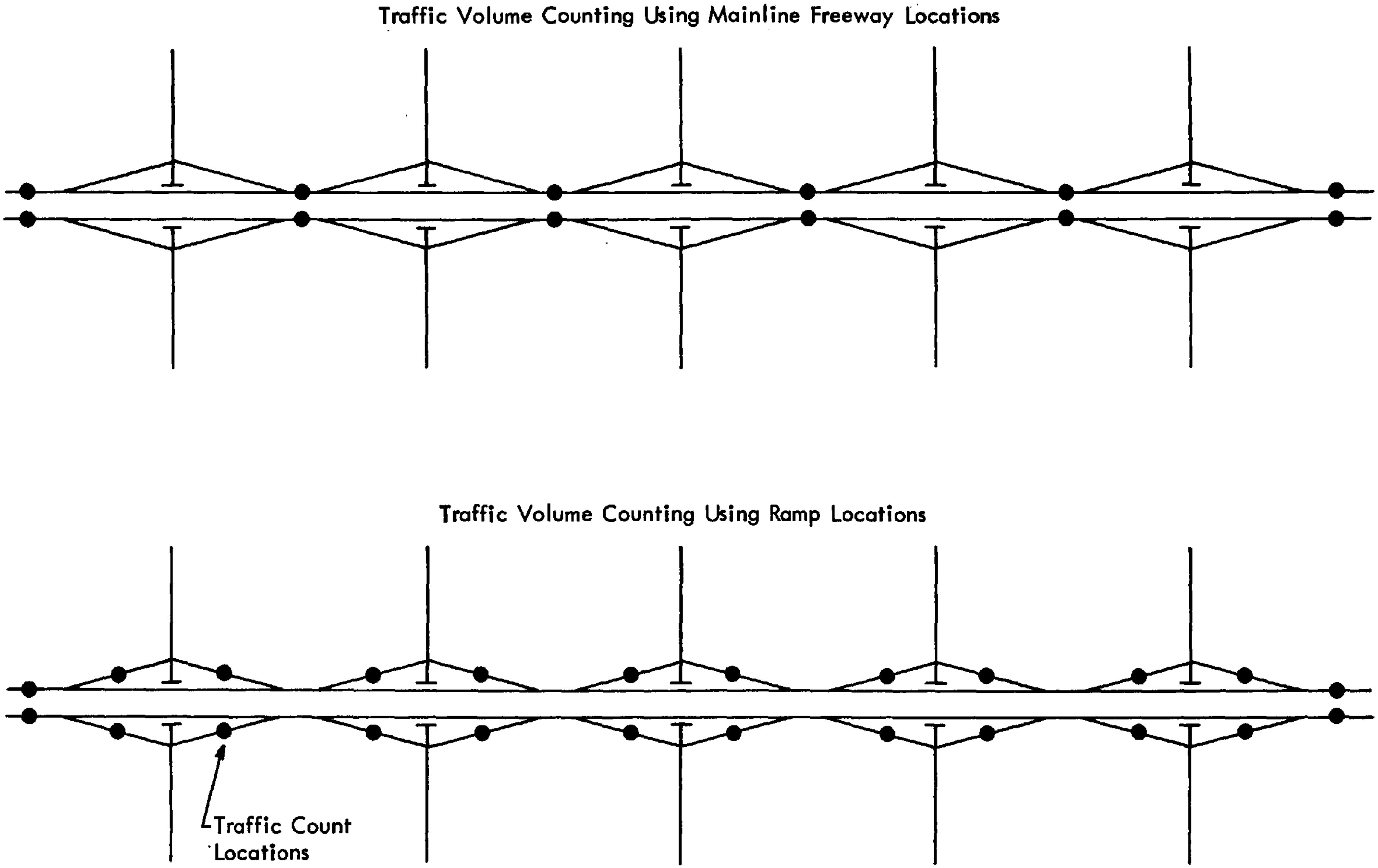


Figure 1 - Traffic Volume Counting Strategies for Urban Freeways.

Every highway agency should review each interchange periodically and select those with the most serious operational problems as improvement candidates. While this process will remain somewhat subjective, objective measures such as delay should be utilized to the greatest possible extent. The traffic volume data base can be used in the process to identify locations with traffic volumes obviously in excess of capacity and locations with rapid growth rates where existing problems will be magnified.

B. Safety Problems

Interchange safety problems can be identified by a combination of formal and informal techniques, as is the case with indentification of operational problems. Greater reliance should be placed on formal surveillance systems for safety problems than for operational problems, because safety problems are often more subtle and difficult to detect.

Computerized accident records systems provide the most powerful and effective means of reviewing the entire highway system under the jurisdiction of an agency to identify locations with safety problems. Most State highway agencies have had accident records systems for a number of years and are continuously refining these systems to improve their accuracy and expand their capabilities. However, many accident records systems lack the completeness, accuracy and timeliness needed for effective accident surveillance. Additional development is often necessary to assure that the accident data base is complete and accurate, and that appropriate software is developed to identify locations with potential safety problems. Furthermore, freeway-arterial interchanges are particularly difficult to consider in accident surveillance because they often connect roadways of different jurisdictions and functional classes and because many separate roadways and ramps are located within a small area. Finally, certain kinds of safety problems, such as single vehicle run-off-road accidents in interchange areas, cannot always be addressed with accident records systems because such accidents are often unreported.

For these reasons, the identification of safety problems should be approached with a combination of formal and informal techniques. The informal techniques could include observations of traffic operations; on-site inspections of roadways, shoulders and roadside areas for tire marks and damaged hardware; and investigation of complaints from local jurisdictions and from the public. The use of formal traffic conflicts and erratic maneuver studies to identify problem locations is not recommended, because it would not be feasible to conduct such studies at every interchange. However, traffic conflicts and erratic maneuver studies are discussed in Appendix B, as a technique for investigating safety problems identified by some other means.

The remainder of this section is devoted to a discussion of computerized accident surveillance systems, with emphasis on their application to interchange areas. Increasing reliance can be placed on computerized accident surveillance, as systems are improved in future years. However, some reliance on manual and informal techniques will be necessary, whatever advances in computer systems are made.

1. Measures for safety problem identification: The two primary measures for safety problem identification are accident frequency and accident rate. The accident frequency is the actual number of accidents at a location, while the accident rate is a ratio between the accident frequency and the exposure to traffic volume. Different forms of accident rate are used for different types of locations. For a highway section--e.g., a mainline freeway or arterial crossroad section--the accident rate is defined as:

$$R = \frac{(N)\ (10^6)}{(D)\ (ADT)\ (L)}$$

where R = Accident rate (accidents per million vehicle-kilometres or per million vehicle-miles),
N = Number of accidents in a given time period,
D = Length of time period (days),
ADT = Average Daily Traffic on section (vehicles), and
L = Length of section (kilometres or miles).

The most common form of accident rate is the accident rate per million vehicle-kilometres or per million vehicle-miles, as defined above. The accident rate per hundred million vehicle-kilometres or per hundred million vehicle-miles is also used by some agencies.

For an intersection or a ramp, the accident rate is usually defined as:

$$R = \frac{(N)\,(10^6)}{(D)\,(ADT)}$$

where R = Accident rate (accidents per million vehicles),
N = Number of accidents in a given time period,
D = Length of time period (days), and
ADT = Average Daily Traffic on intersection or ramp (vehicles).

For an intersection, ADT represents the sum of the traffic volumes entering the intersection for a 24-hour period. The intersection accident rate is also applicable to a crossroad ramp terminal. For a ramp, ADT represents the traffic volume on the ramp. The accident rate for the interchange as a whole can also be expressed as accidents per million vehicles. In this case, ADT represents the traffic volume entering the interchange on both freeway and crossroad approaches.

2. <u>Location reference methods</u>: A key element in computerized accident surveillance is the location reference method used to identify accident locations. Three basic types of location reference methods are in use today: the Route Number - Accumulated Distance method; the Link-Node method; and the Coordinate method.[15] Figure 2 illustrates the application of these three methods to a typical diamond interchange. The following discussion identifies the strengths and weaknesses of each method and the suitability of their use in interchange areas. All highway agencies currently express accident locations in units of miles or feet; however, the concepts of each method described below are equally applicable to accident locations expressed in metres or kilometres.

a. <u>Route number - accumulated distance method</u>: The location reference method most commonly used by State highway agencies is the Route Number - Accumulated Distance method. This method is not well suited for use in interchange areas, but has been adapted by incorporation of additional location data for ramp accidents and has been employed successfully by many agencies. In this method, accidents are identified by a route number or control section number and the distance from a known reference point such as the beginning of the route or a county line. The milepost systems employed by many agencies are examples of the Route Number - Accumulated Distance method. The route number, milepost and direction of travel of involved vehicles are sufficient to identify the location of accidents on the mainline freeway and on the arterial crossroad (if the crossroad is a State highway).

In some highway agency accident records systems, all ramp accidents at an interchange are coded to a single location (usually the milepost of the crossroad structure) and the accident experience of individual ramps or ramp terminals can be determined only by manual means. This type of system is generally unsatisfactory for automated interchange accident surveillance, and a system for identifying the ramp on which each accident occurred is needed.

Two systems for locating ramp accidents are illustrated in Figure 2. In System I, each ramp is identified by the route number and the location of the gore point where the ramp leaves the mainline freeway. A

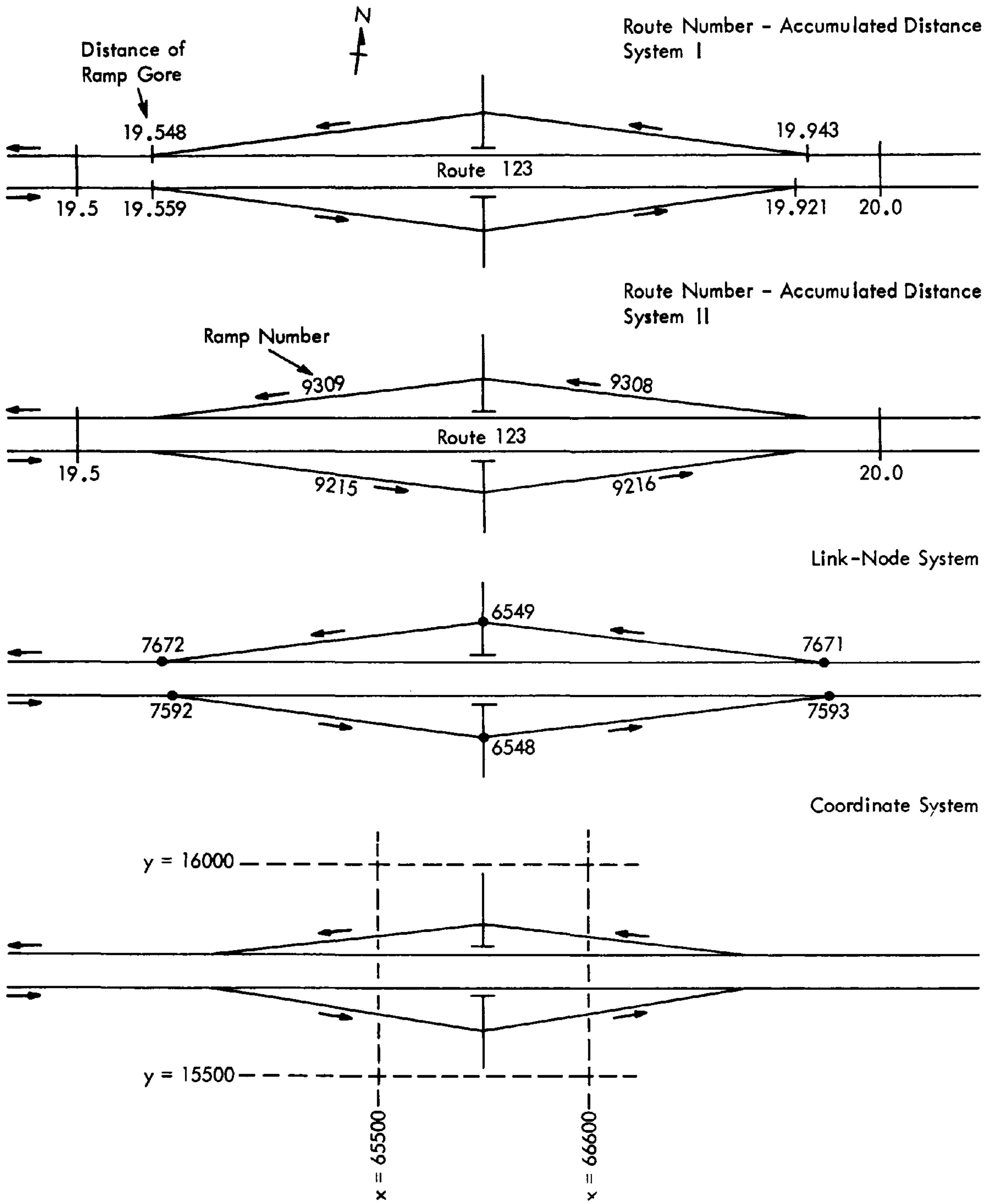


Figure 2 - Examples of Location Reference Methods for Interchange Areas.

special data item in the computer system identifies ramp accidents so that they cannot be confused with mainline freeway accidents at the same location. This system is employed by the California Department of Transportation, the Michigan Department of State Highways and Transportation, and many other agencies. In System II, each ramp is identified by a route number and a unique ramp number. This system is employed by the Illinois Department of Transportation. In System I, an accident on the eastbound off-ramp would be designated as a ramp accident on Route 123 at location 19.559. In System II, the same accident could be identified as a Route 123 accident on Ramp 9215.

In either system, an additional code can be used to identify the portion of the ramp on which an accident occurred. A typical set of ramp location codes are those employed by the California Department of Transportation:[10]

Code	Location	Definition
1	Ramp Exit	Accidents related to terminal at end of ramp (freeway end of on-ramps; crossroad and of off-ramps).
2	Ramp Body	Accidents on the ramp not related to either terminal.
3	Ramp Entry	Accidents related to terminal at beginning of ramp (crossroad end of on-ramps; freeway end of off-ramp).
4	Non-State Route	Accidents on non-state crossroad related to the ramp.

Accidents on the arterial crossroad in the interchange area are handled in a variety of ways. If the crossroad is a State highway, then accidents are identified by the route number and location on the crossroad. If the crossroad is not a State highway, most systems do not include accidents unless they occur at the ramp terminal. For example, accident data for the intersection of a frontage road and a non-State crossroad would not be included in the computerized surveillance system, even if the intersection were only 30 m (or 100 ft) from a ramp terminal.

b. Link-node method: The Link-Node method is used by only a few agencies, but the concept it employs is extremely well suited for use in interchange areas. In this system, each accident is located at a node or on a link defined by two adjacent nodes. The nodes are important point locations such as intersections, ramp terminals, bridges and jurisdictional boundaries. Each node is identified for recordkeeping purposes by a node number which--alone or in combination with a county code--is unique within the highway system.

Figure 2 illustrates the application of the Link-Node method to a diamond interchange. Accidents related to one of the four freeway ramp terminals would be located by the appropriate node number: 7592, 7593, 7671 and 7672. Accidents at the crossroad ramp terminals would be located at nodes 6548 and 6549. Accidents on the mainline freeway between the ramp terminals would be located on links 7592 - 7593 and 7671 - 7672, while ramp accidents would be located on links 7592 - 6548, 6548 - 7593, 7671 - 6549 and 6549 - 7672. The system may also include the distance of the accident location along the link from one of the nodes, allowing individual accidents to be located exactly. The Maine Department of Transportation uses this kind of system, and the New York State Department of Transportation has a similar system under development.

c. Coordinate method: In the Coordinate method, accident locations are referenced to an established grid system such as Universal Transverse Mercator Coordinates or State Plane Coordinate systems. Each accident is located by two coordinates. The major advantage of a coordinate system is the ease with which the accident surveillance system can be adapted to mechanical plotting equipment. The major disadvantage is that, with existing maps, accidents can be located to an accuracy of only 150m (or 490 ft). With the many roadways and ramps that can be present in an interchange area within a space of 150m, it is obvious that additional location information similar to that used in the milepost and link-node methods must be supplied for interchange accidents. Figure 2 demonstrates that both mainline freeway roadways, the arterial crossroad and all four ramps could be included within a single grid in a 150m coordinate system. Thus, the coordinate method is not readily adaptable to interchange accident surveillance.

d. Recommended method: It is recommended that highway agencies use either the Route Number - Accumulated Distance method or the Link-Node method in accident surveillance systems used for freeway interchanges. The system should have the capability to identify the ramp or other interchange element in which each accident is located.

3. Capabilities of Accident surveillance systems: Most States with computerized accident records systems have developed software to review the accident experience of elements of the highway system and identify locations with potential safety problems. The capabilities of two State systems--California and Michigan--are reviewed here to illustrate the state-of-the-art of accident surveillance in interchange areas. Both systems are among the most advanced in the nation, but differ in their approach to some basic aspects.

a. <u>California</u>: The California Department of Transportation operates an accident surveillance system known as TASAS (Traffic Accident Surveillance and Analysis System) for all State highways.[9] The system includes both an accident file and a file of highway characteristics (including geometrics and traffic volumes) that can be used together in accident analyses to calculate accident rates or identify accidents associated with specific geometric features.

The TASAS system has the capability to identify locations with accident rates significantly higher or lower than the expected accident rate for the location. This identification is performed by a statistical comparison of the actual number of accidents (N_A) for a given time period with the expected number of accidents (N_E) corresponding to the expected accident rate. The types of locations considered by the program are highway sections 0.16 to 0.80 km (0.1 to 0.5 miles) in length, intersections and ramps. Thus, an interchange would be considered as a series of highway sections and ramps rather than as a single unit. The expected number of accidents for an interchange ramp would be calculated as:

$$N_E = \frac{(R_E)(D)(ADT)}{10^6}$$

where N_E = Expected number of accidents for ramp,
R_E = Expected accident rate for ramp (accidents per million vehicles),
ADT = Average Daily Traffic for ramp (vehicles), and
D = Number of days in study period.

Expected accident rates have been established and incorporated in the program for only two categories of ramps: urban ramps and rural ramps. This limitation obviously reduces the reliability of the system. Expected accident rates for specific ramp types, such as diamond ramps, loop ramps, buttonhook ramps, etc., are currently being developed by the State.

The following criteria are used to determine whether the actual accident rate for a location is significantly higher or lower than the expected value:

Confidence Level	Significantly Higher If	Significantly Lower If
80%	$N_A = N_E + 1.282(N_E)^{½} + 0.5$	$N_A = N_E - 1.282(N_E)^{½} + 0.5$
85%	$N_A = N_E + 1.440(N_E)^{½} + 0.7$	$N_A = N_E - 1.440(N_E)^{½} + 0.7$
90%	$N_A = N_E + 1.645(N_E)^{½} + 0.8$	$N_A = N_E - 1.645(N_E)^{½} + 0.8$
95%	$N_A = N_E + 1.960(N_E)^{½} + 1.0$	$N_A = N_E - 1.960(N_E)^{½} + 1.0$
99%	$N_A = N_E + 2.576(N_E)^{½} + 1.4$	$N_A = N_E - 2.576(N_E)^{½} + 1.4$

The criteria are based on the two-tail t-test. Accident frequencies are assumed to follow a Poisson distribution; the test actually uses the normal distribution, with a correction factor (0.5, 0.7, etc.) added to approximate the Poisson distribution.

The program user selects the confidence level to be used and the length of highway secitons to be considered. The program then conducts the statistical tests for all locations on the State highway system for 5 periods: the latest 36 months, latest 24 months, latest 12 months, latest 6 months and latest 3 months of accident data. The program identifies a location with high (or low) accident experience if the number of accidents for any of the 5 periods exceeds the criteria presented above. These multiple time periods allow the system to detect both persistent, long-term problems and newly developed problems. Both total accident rate and fatal and injury accident rate are tested. The output of the program is a printout which identifies each location with high (or low) accident experience and identifies the criterion under which it was selected. The program identifies locations with potential safety problems, but does not attempt to establish improvement priorities.

b. Michigan: The Michigan Department of State Highways and Transportation has recently performed a study to establish priorities for interchange improvement.[30,31] While the procedures used for this study were not part of a formal surveillance system, similar procedures can be used in the MIDAS (Michigan Dimensionalized Accident Surveillance) system that is currently under development. This study differs from the California approach in several ways. First, the Michigan study was based on the accident experience for the entire interchange area rather than for individual ramps. Second, the statistical criteria for identifying high-accident interchanges were determined directly from the data set rather than established *a priori* from accident data for a previous year. Finally, a more complex system of cutoff values was used to select a set of critical interchanges with both high accident frequencies and high accident rates. The objective of the Michigan study was to select a group of priority interchanges based on high fatal and injury accident experience without regard to interchange configuration. However, the same procedure could be employed to establish priorities for each individual interchange configuration.

A stepwise procedure was used to identify priority interchanges from among the 621 freeway interchanges in the State. It was decided that priority interchanges should be those with injury accident frequencies both significantly higher than average injury accident frequency and at least twice the average injury accident frequency. Eighty-one (81) of the 621 interchanges exceeded both these criteria. Thirty-three (33) of these 81 interchanges were classified as critical interchanges because they had injury accident rates above the average injury accident rate for the entire group of 81 interchanges. Table 1 gives the actual numerical criteria used as cutoff values in this study.

The result of the study was a list of the 33 critical interchanges arranged in descending order of injury accident rate. However, it was not intended that the interchange with the highest injury accident rate have priority over the second highest, etc. Rather, each of the critical interchanges will be considered further in Phase II of the study to determine which of these 33 interchanges have the greatest potential for cost-effective improvements.

4. Recommendations for interchange accident surveillance: This section presents recommendations for computerized accident surveillance systems based on the current state-of-the-art and advances being made in California, Michigan and other states.

a. An accident surveillance system should have access to files of current and historical geometric and traffic volume data, as well as accident data. An appropriate organization for a surveillance system is presented in Figure 3.

b. The system should include all reportable accidents on the approach to and within each interchange, including the mainline freeway, ramps and arterial crossroad. It is generally necessary to choose a fixed distance from the ramp terminals along both the mainline freeway and arterial crossroad at each interchange for inclusion in the accident surveillance system. Suggested limits are 300 m (or 1,000 ft) or more along the mainline freeway and 30 m (or 100 ft) or more along the arterial crossroad. Accidents on the crossroad should be included even if the crossroad is maintained by another agency, because the crossroad is at least as likely a source of operational and safety problems at freeway-arterial interchanges as the mainline freeway, and, therefore, deserves complete consideration.

TABLE 1

4-STEP PROCEDURE FOR IDENTIFYING CRITICAL INTERCHANGES IN MICHIGAN STUDY

Step	Measure Used	Definition of Criteria Used	Numerical Criteria Used
1. First checkpoint	Number of injury accidents	Significantly greater than average injury accident frequency, as determined by Poisson distribution test at 97.5% Level of Confidence	28 injury accidents
2. Second checkpoint	Number of injury accidents	Twice the average injury accident frequency	37.24 injury accidents
3. Select first cutoff	Number of injury accidents	Highest of 2 checkpoints	37.24 injury accidents
4. Second cutoff (critical accident rate)	Injury accidents per million vehicles	Average rate for interchanges exceeding first cutoff	0.597 injury accidents per million vehicles

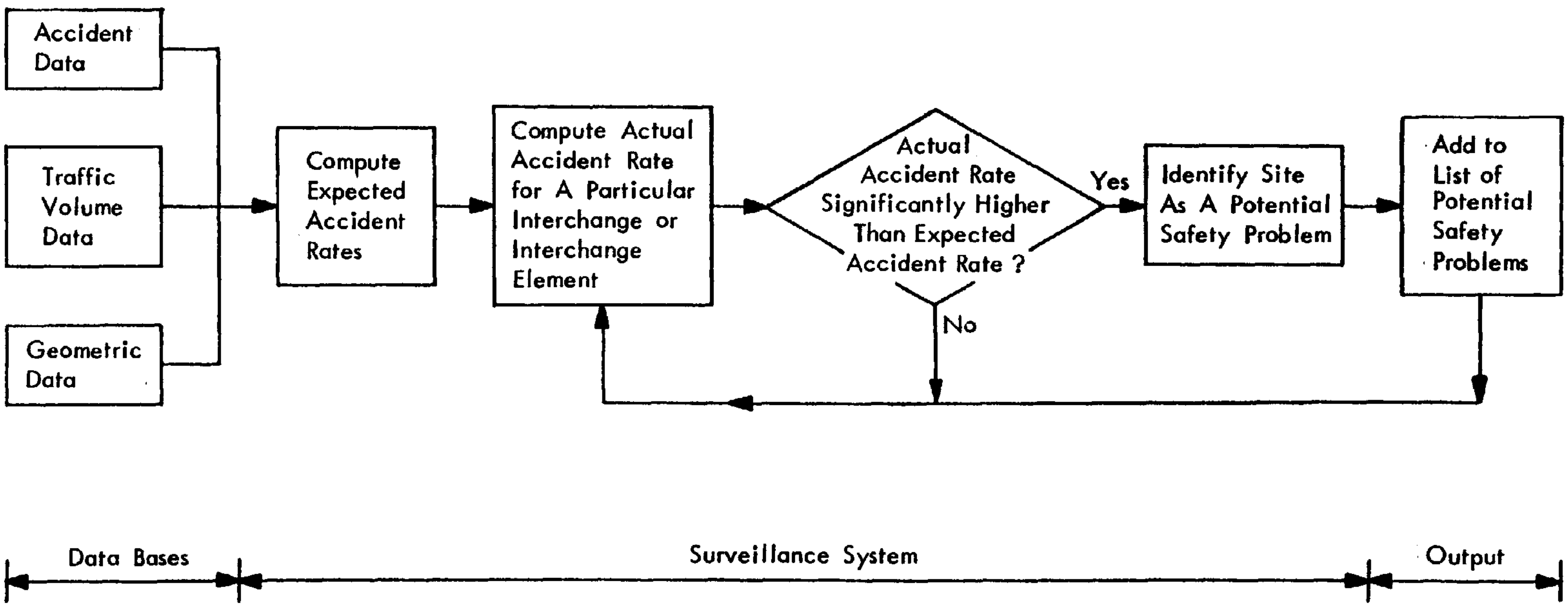


Figure 3 - Flow-Chart of an Accident Surveillance System.

c. The system should review the accident experience of individual portions of the interchange and the interchange as a whole. This is a combination of the California and Michigan approaches described above. The portions of the interchange to be reviewed should include:

- Mainline freeway;
- Arterial crossroad;
- Crossroad ramp terminals; and
- Individual ramps.

d. Identification of interchanges with safety problems should be based on a statistical comparison of actual accident rates with expected accidents rates. Additional criteria such as a minimum accident frequency may also be included.

e. Expected accident rates should be established for the following elements of interchanges and overall interchange configurations, at a minimum:

Interchange Elements	Interchange Configurations
Mainline Freeway (by number of lanes)	Full Diamond Half Diamond
Arterial Crossroad (by number of lanes)	Full Slip-Ramp Diamond Parclo A
Diamond Ramp	Parclo A/4-Quad
Loop Ramp	Parclo B
Outer Connection Ramp	Parclo B/4-Quad
Buttonhook Ramp	Parclo AB
Slip Ramp	Parclo AB/4-Quad
Directional or Semi-Directional Ramp	Full Cloverleaf Full Cloverleaf (less 1 loop)
Other Ramps	Full Cloverleaf (with collector-distributor road)
	Directional

These ramp types and interchange configurations are illustrated in Figures 4 and 5. Figure 4 presents typical ramp types in freeway-arterial interchanges and Figure 5 presents typical freeway-arterial interchange configurations. Other categories, such as directional ramps, should be added as needed to suit the particular highway system to be considered. Depending on the configuration of the surveillance system, expected accident rates can be calculated by the program or determined a priori from data for the previous year.

f. It may also be desirable to establish expected accident rates for specific categories of accidents. For example, the review of night accident rates and wet-pavement accident rates can be used to identify potential locations for application of specific countermeasures such as improvement of lighting or skid resistance.

g. An interchange should be identified as having a potential safety problem if the accident rate for the entire interchange or any portion of the interchange is significantly higher than the expected accident rate. Statistical signifigance should be based on the approximate Poisson confidence limits currently used by California (see previous section) or on exact Poisson confidence limits, as tabulated in may statistics texts such as Fryer.[14]

h. The surveillance system should identify ("flag") interchanges with potential safety problems for further investigation. However, the surveillance system results should not be used to rank interchanges in priority order. Implementation priorities should include consideration of the cost and anticipated effectiveness of solutions, as well the magnitude of the safety problem.

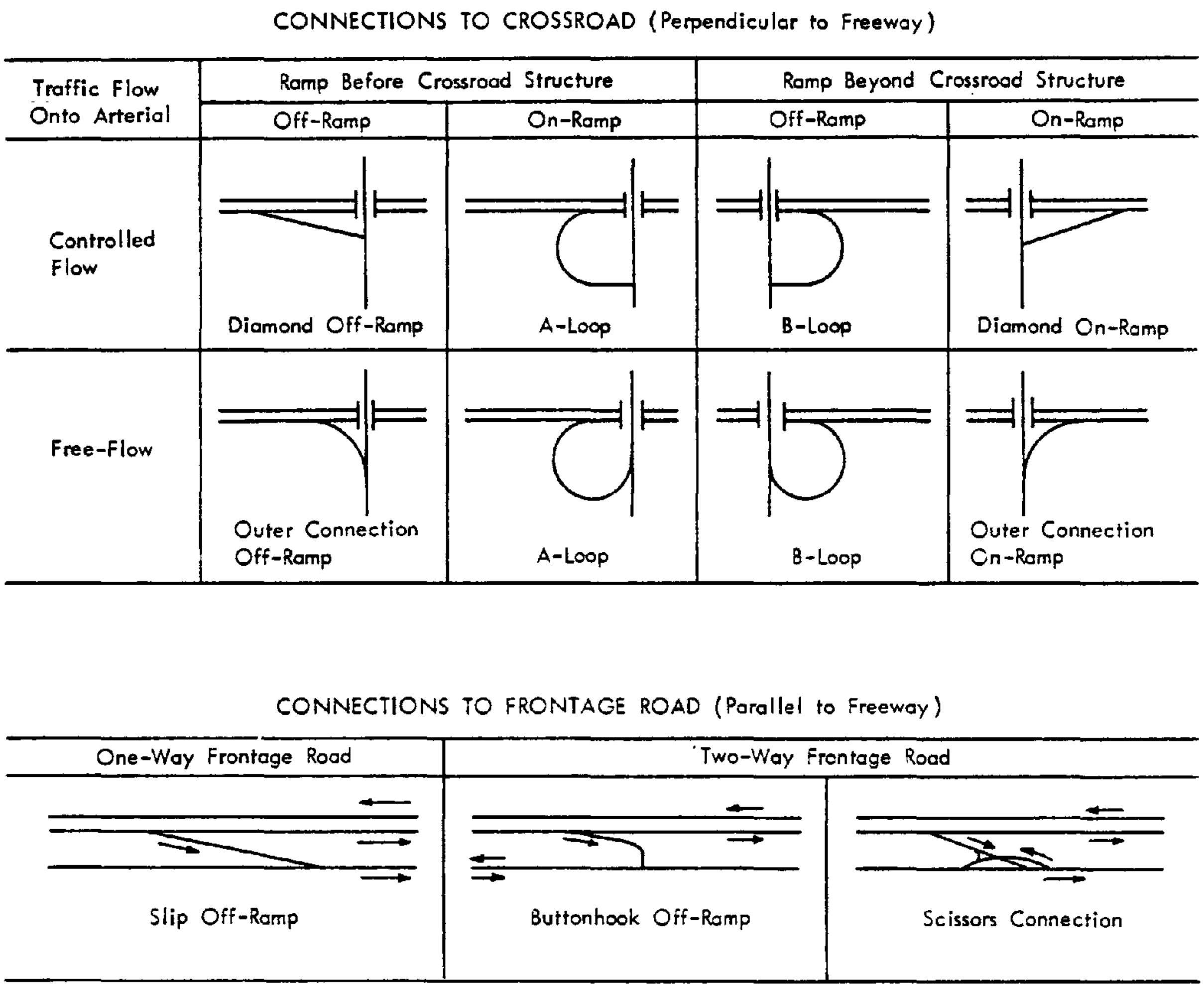


Figure 4 - Typical Ramp Types in Freeway-Arterial Interchanges

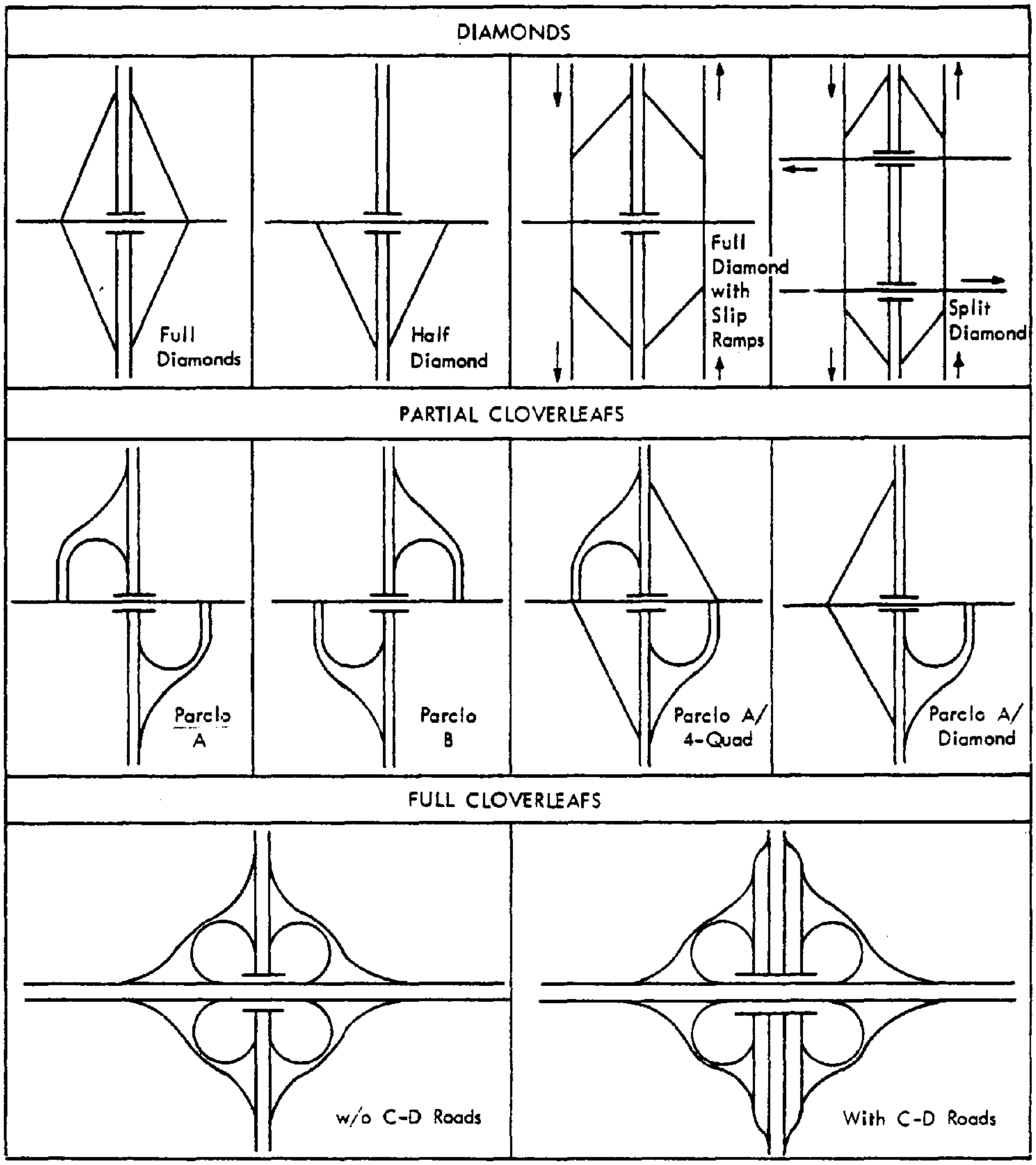


Figure 5 - Typical Freeway-Arterial Interchange Configurations

V. STUDY PROBLEM INTERCHANGES AND IDENTIFY SPECIFIC DEFICIENCIES

Once a set of interchanges with operational and/or safety problems has been identified, the next step in the interchange rehabilitation process is to study each interchange to determine the nature of the operational or safety problem(s) and, to the extent possible, identify the cause of the problem. The goal of the investigation should be to identify the basic cause of the operational or safety problems at the interchanges, although complete identification of the symptoms of the problem may also be valuable. This investigation, analogus to a medical diagnosis, is accomplished through established engineering study methods. These methods are commonly divided into three broad categories: physical inventories, operational studies and safety studies. Not every engineering study is suitable for any given location or problem. Instead, a set of basic studies have been defined that should be performed at each problem interchange. The basic studies define the existing operational and safety conditions in much greater detail than does the systemwide review of conditions in Section IV. As more is learned about the nature of an existing problem, supplementary studies that help to isolate the cause of the problem or identify an appropriate solution may be needed.

This section describes the basic engineering studies that should be considered for use to study operational and safety problems of freeway-arterial interchanges and also identifies supplementary studies that may be appropriate. The supplementary studies are discussed further in Appendix B. Most of the basic and supplementary engineering studies are presented in detail in standard reference books such as the Transportation and Traffic Engineering Handbook[19] and the Manual of Traffic Engineering Studies.[18] Other sources that concern specific engineering studies are referenced in the appropriate discussion.

A. Basic Engineering Studies

Basic physical inventories, operational studies and safety studies should be performed at each problem interchange to determine the nature and the magnitude of the existing operational and safety problems. Procedures for these studies are discussed below.

1. Physical inventories: An initial step in the investigation of any interchange-related problem should be to document the physical conditions at the site. The geometrics of the interchange are needed as the basis for locating all data collected about the interchange and, in many cases, the review of the geometrics may suggest potential operational and safety problems at the interchange. And, of course, a physical description of the interchange, surrounding development and topography will be needed to select and evaluate alternative improvements.

In the engineering study stage, the best available physical description of an interchange is usually the set of construction or as-built plans for the original construction or the most recent reconstruction of the interchange. The plans should include the mainline freeway, ramps and arterial crossroad within the interchange area. Details of ramp terminal and frontage road or other adjacent interchanges are desirable. Profiles of the interchange roadway and ramps are needed where sight distance, grade or drainage is a problem. Signal plans, pavement marking plans and signing plans are usually useful also. Formal surveys and signing inventories are not required at this stage, but often are later in the design process.

2. Traffic operational studies: The basic traffic operational studies to be performed include on-site observation of traffic operations and traffic volume counts.

a. On-site observations: On-site observation by a highway engineer is recommended for each problem interchange. Although data collection for most engineering studies can be accomplished by trained technicians, the value of visits to the interchange by the engineer cannot be overemphasized. The ability to learn from the existing situation and eliminate deficiencies from the revised design is a major advantage for the designer of a rehabilitation project.

The timing of visits by the engineer should be carefully selected. While most operational problems occur during the morning and evening peak periods, safety problems often occur during off-peak periods and at night as well. Traffic volume data and accident summaries, both discussed below, can aid in the appropriate timing of field visits. Repeated visits during several stages of the interchange study process may also be useful.

Two different forms of observation should be employed. First, the engineer should drive through the interchange from all approaches to observe conditions from the driver's point of view. Then, several good vantage points should be chosen to observe traffic and identify unusual behavior. Table 2 lists symptoms of operational problems that should be noted and physical inventory data that may be conveniently collected during an on-site visit. The list of physical inventory parameters in Table 2 includes only items that are not usually available on existing construction plans; these items can be noted on the construction plans in the field. Many engineers also take photographs of geometric or operational problems for later review in the office and possible use in reports or public hearings.

TABLE 2

BASIC FIELD OBSERVATIONS USED TO STUDY PROBLEM LOCATIONS

Operational Problem Symptoms	Physical Inventory Parameters (to supplement construction plans)
1. Length of vehicle queues	1. Sight distance restrictions
2. Erratic vehicle maneuvers such as: (a) stopping or backing at gore points; (b) wrong-way movements; (c) gore area encroachments; (d) shoulder encroachments; and (e) traffic violations.	2. Pavement and shoulder conditions
3. Vehicles experiencing difficulty in making turning movements.	3. Signal visibility
4. Vehicles experiencing difficulty in making merging or weaving movements.	4. Signs, including speed limits
5. Evidence of unreported accidents such as damaged guardrail or skid marks or tire tracks off of the pavement.	5. Curb radii
6. Pedestrians on freeway or crossing ramps.	6. Pavement markings
7. Pedestrian-vehicle conflicts.	7. Lighting
	8. Driveway and frontage road locations
	9. Fixed objects and roadside design

Many agencies have photologs available so that site conditions can be reviewed in the office. Review of the photolog is often valuable prior to or following a field visit to the interchange. Many geometric features are clearly visible in a photolog and unnecessary repeat visits to an interchange may be avoided. However, review of a photolog cannot replace on-site observation because photolog coverage of interchange areas is often incomplete and because traffic operational conditions cannot be reviewed in a photolog.

b. Traffic volume counts: Traffic volume counts at interchanges are needed for capacity and operational analyses, computation of accident rates in safety analyses, establishment of signal warrants and timing, and many other purposes. There are several different types of volume counts including average annual daily traffic, peak hour volumes for roadways and ramps, peak hour turning directional counts, lane use counts, vehicle classification counts, bicycle and pedestrian counts and traffic volume trends.

The average daily traffic volume (ADT) is necessary to calculate accident rates. It is recommended that for each problem interchange, ADT volumes be obtained for each interchange element including mainline, arterial crossroad and all ramps. These data were also used in Section IV for the identification of interchanges with safety problems and should be available. Other traffic volume counts should be made as needed.

Peak hour volumes should be obtained for analysis of delay and congestion problems, which are most prevalent during the morning and evening peak periods. Traffic counts should be made for 1 to 3 hours during each peak period. Peak hour turning movement counts at ramp terminals are needed to analyze signal operations and the need for exclusive turn lanes. Conventional turning movement counts measure through, left-turn and right-turn volumes on each approach to a ramp terminal. Special turning movement counts may be required at locations such as a ramp terminal located very close to a frontage road intersection, where complex turning movements may occur.

Directional counts are needed to determine the predominant traffic movement during different times of the day and lane use counts are valuable in analyzing merging or weaving problems.

Vehicle classification counts determine traffic volumes by vehicle type. Classification counts or estimates of the truck volumes are needed for determination of capacity and for the operational analyses presented in Section VIII.

Bicycle and pedestrian counts may be needed where bicycle or pedestrian safety problems are found at ramp terminals on urban arterials.

Vehicle occupancy counts to determine the average number of occupants per vehicle in the traffic stream are used only in specialized situations at interchanges when high-occupancy-vehicle (HOV) lanes are being considered to alleviate capacity problems on the mainline freeway.

Traffic volume studies often include consideration of historical trends of traffic volume growth used to predict future traffic volumes. Short-term projects may simply extrapolate the historical trends, but long-term estimates often consider changes in the development of the surrounding area, which can markedly affect turning movement volumes.

In addition to traffic volume data, data on trip purpose (work, shopping, recreation, etc.) and driver familiarity (local driver vs. out-of-town driver) may also be helpful in understanding the problems at a particular interchange.

3. Safety studies: The basic safety studies performed for interchanges should include accident tabulations and summaries, and collision diagrams.

a. Accident tabulations and summaries: Accident tabulations and summaries are used to document the magnitude of the safety problem at an interchange, the types of accidents that occur, the specific contributing factors and potential causes of the accidents. These accident data are also required to make estimates of the safety effectiveness of alternative improvements in Section VII.

It is recommended that 3 years of accident data be analyzed, whenever possible. The police accident report form (hard copy) is usually the most complete source of accident information available, although the computer systems of many State highway agencies provide adequate data for analysis. In most States, accident reports must be obtained from local jurisdictions to assure complete coverage of an interchange area, especially on the arterial crossroad.

The accident data should be tabulated by accident type, by location (also summarized graphically in a collision diagram), by time of day, day of week, month, by severity, by light condition, by pavement surface condition, by weather, by number of vehicles involved, and by vehicle type. Both accident frequencies and accident rates are of interest.

Accident types that are commonly used to classify intersection- or ramp-terminal-related accidents are right angle, rear-end, side swipe or merging, head-on, pedestrian, fixed object, overturning, right turn, and left turn. Accident types used to classify accidents that are not intersection- or ramp-terminal-related are ran-off-road, overturning, fixed object, pedestrian, rear-end, and side swipe or driveway-related.

The most common severity classifications for accidents are property-damage-only, nonfatal injury, and fatal. Nonfatal injury accidents are sometimes subdivided into incapacitating injury, nonincapacitating injury, and possible injury accidents.

Light conditions are most commonly described as daylight or darkness, although dusk and dawn classifications and indications of artificial lighting conditions are used by some agencies.

Surface conditions are usually classified as dry, wet, and snow, or ice. Weather conditions are usually classified as cloudy, clear, rain, or snow.

The number of vehicles involved is used to classify accidents as single vehicle or multiple vehicle. Vehicle types are classified as passenger vehicles, busses, and trucks, although more detailed classifications of each type of vehicle are possible.

The two primary measures of accident experience are accident frequency and accident rate. The accident frequency is the actual number of accidents at a location, while the accident rate is a ratio between the accident frequency and the exposure to traffic volume. Both the accident frequency and accident rate and their application to different portions of a freeway-arterial interchange have been defined in Section IV.

b. Collision diagrams: Collision diagrams present a graphical summary of the location and types of accidents at an interchange. The collision diagram can be used to examine the spatial distribution of accidents at the interchange and identify concentrations of accidents and patterns of similar accidents. A collision diagram can be based on the same accident data used to prepare the accident tabulations discussed above, if the location of each accident and the direction of travel of each vehicle are known.

The first step in preparing the collision diagram is to sketch the interchange. This sketch need not be to scale, but should be large enough to illustrate each accident distinctly. The interchange may be sketched on a single sheet (usually larger than 8½ x 11") or separate diagrams may be prepared for individual interchange elements such as ramp terminals. Collision diagrams that illustrate the accident experience of an entire interchange on a single sheet are often extremely valuable.

Each accident should be drawn on the collision diagram showing the intended movement of each involved vehicle. The basic characteristics of each accident that should be noted on the diagram are:

- Date, day of week, and time of occurance;
- Weather and pavement conditions;
- Light conditions; and
- Number of injuries or fatalities.

Special circumstances or driver comments such as traffic control device malfunctions should also be noted. Figure 6 presents an example of a collision diagram illustrating the symbols used to represent typical types of accidents at a crossroad ramp terminal.

The completed collision diagram should be examined to determine the concentrations and patterns of accidents that are present. Some of the most prevalent interchange accident problems that should be noted from the collision diagram are:

- Rear-end accidents on freeway approach roadway;
- Rear-end accidents in deceleration lane and off-ramp gore area;
- Rear-end accidents on off-ramps;
- Rear-end accidents on on-ramps;
- Crossroad accidents related to off-ramp backup;
- Merging accidents involving right- and left-turns at entrance to on-ramp or frontage road;
- Merging accidents in acceleration lane area of on-ramps;
- Rear-end, left-turn and right-turn accidents on arterial (at ramp terminals and adjacent intersections);
- Accidents within mainline freeway weaving areas;
- Rear-end and angle accidents in weaving area between free-flow ramp terminal and adjacent intersection (or major driveway);

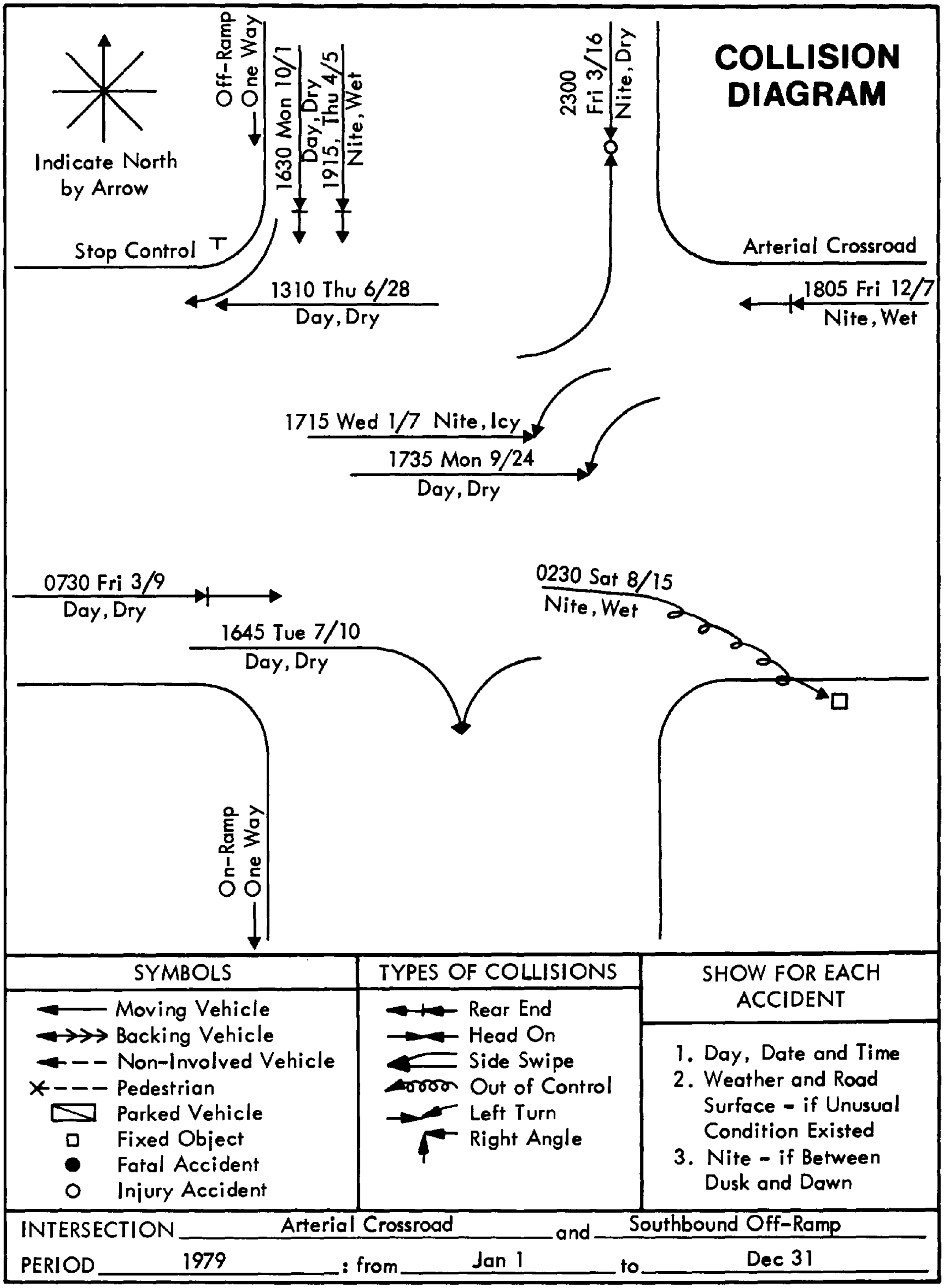


Figure 6 - Example of Collision Diagram for a Crossroad Ramp Terminal.

- Single-vehicle accidents on ramps;
- Head-on accidents on two-way ramps; and
- Wrong-way accidents on ramp or freeway.

Specific solutions to these safety problems are suggested in Section VI.

B. Supplementary Engineering Studies

Supplementary engineering studies should be employed to further define the nature of operational or safety problems, isolate the cause of the problem and help identify appropriate solutions. The most common kind of supplementary studies are capacity or travel time and delay analyses used to determine which portion(s) of the interchange are responsible for existing congestion. Additional supplementary studies are performed when a need is indicated by the basic engineering studies or to establish a warrant for a specific counter measure (e.g., installation of traffic signals). Table 3 lists the supplementary engineering studies used most frequently for interchange operational and safety problems and identifies the circumstances under which each might be performed. Supplementary studies could also include interviews with drivers, police officers and local maintenance personnel familiar with the interchange. The supplementary studies are discussed in more detail in Appendix B.

TABLE 3

SELECTION OF SUPPLEMENTARY ENGINEERING STUDIES

Supplementary Study	Purpose of Study	Symptom of Operational Study Problem That Indicates Study Needed
Capacity Studies	To determine operating conditions and pinpoint bottlenecks	Congestion and delays
Travel Time and Delay Studies	To determine location and extent of delay and average travel speeds	Intersection congestion Other congestion at interchanges Rear-end accidents during peak periods
Speed Studies	To determine actual vehicle speeds, actual speed profiles and adequacy of legal and advisory speed limits	Extremely high or low speeds observed during on-site visits Run-off-road accidents on ramps Rear-end accidents near intersections Rear-end accidents at on-ramp merging areas
Traffic Conflict and Erratic Maneuver Studies	To supplement traffic accident data and identify potential accident problems	Hazardous driver actions observed during on-site visits Public complaints of safety problems not evident in accident data.

TABLE 3 (Concluded)

Supplementary Study	Purpose of Study	Symptom of Operational Study Problem That Indicates Study Needed
Traffic Signal Studies	To determine need for and design of traffic signals, to identify improper phasing, timing or interconnect strategy and to identify unwarranted signals.	Right angle accidents at unsignalized intersections
		Excessive delay at STOP sign controlled intersections
		Excessive delay at existing signalized intersections
Sight Distance Studies	To determine adequacy of the length of highway visable to the driver	Rear-end accidents at horizontal curves, crest vertical curves, or decision points
		Right-angle accidents at uncontrolled intersections
		Turning accidents at intersections
Turning Radius Studies	To determine adequacy of existing curb radii	Sideswipe accidents involving vehicles traveling in opposite directions
		Rear-end accidents in right-turn lanes
		Evidence of large vehicles encroachment on curb or shoulder
Skid Resistance Studies	To determine the coefficient of tire-pavement friction	Run-off-road or skidding accidents under wet-pavement conditions

VI. IDENTIFY IMPROVEMENT ALTERNATIVES

Once a problem interchange has been identified and the nature of the problem has been thoroughly investigated through engineering studies, the next step is to identify alternative solutions to alleviate the problem. A procedure for identifying improvement alternatives is presented below.

A. Procedure to Identify Alternatives

The selection of a set of suitable alternative solutions for a particular interchange problem is highly dependent on the type of operational or safety problem(s), the interchange configuration, the geometrics of individual interchange elements, the traffic volumes present, the type of traffic control devices used, the right-of-way and other physical constraints and many other factors. Because so many factors are involved, it is often stated that the problems of each interchange are unique and require a unique solution; and, in the strictest sense, this is true. However, it is also true that certain general classes of problems recur frequently and that certain general classes of solutions are appropriate for these problems. Therefore, we have attempted to organize the state-of-the-art in a manner that will be useful to designers and traffic engineers who deal with freeway-arterial interchange problems.

Currently, engineers identify appropriate alternative solutions through engineering judgment based on previous experience with interchanges having operational and safety problems. Although a more formalized approach is recommended here, it is not our intention to supplant the engineer's judgment. Rather, we hope to assist the engineer by making the review of potential solutions as efficient as possible. At the same time, we want to ensure that the range of alternative solutions is not limited unnecessarily. An organized procedure to review potential solutions should reduce the possibility that the "best" solution will be missed.

The recommended approach uses a series of 9 charts (presented in Tables 4-12) that relate the common operational and safety problems of freeway-arterial interchanges to potential solutions. Each chart presents appropriate solutions for a specific problem or a set of related problems. Where several distinct problems are present at an interchange, the use of more than one chart may be required to identify all appropriate solutions. The potential solutions are classified by the type of ramp or interchange configuration to which they are appropriate, and by whether the overall interchange configuration or only a portion of the interchange (freeway, ramp, or arterial crossroad) is to be modified.

TABLE 4

DELAYS AND ACCIDENTS ON OFF-RAMPS

PROBLEMS
• Excessive delay on off-ramp • Vehicle queues on off-ramp and/or freeway • Rear-end accidents on off-ramp and/or freeway
FUNCTIONAL OBJECTIVES OF SOLUTIONS
• Increase capacity of crossroad ramp terminal and/or arterial • Increase space available for vehicle storage off of mainline freeway lanes • Reduce traffic volume on ramp

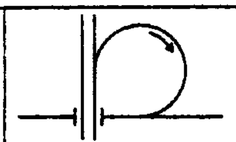
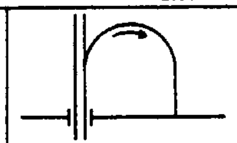
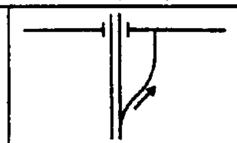
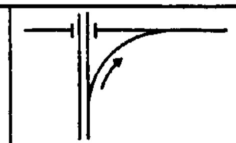
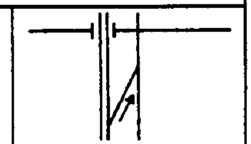
<table>
<tr><th colspan="6">LIST OF POTENTIAL SOLUTIONS</th></tr>
<tr><th></th><th></th><th></th><th></th><th></th><th></th></tr>
<tr><th></th><th>Free-Flow Loop</th><th>Controlled-Flow Loop</th><th>Diamond Ramp</th><th>Outer Connection Ramp</th><th>Frontage Road Slip Ramp</th></tr>
<tr><td rowspan="4">Interchange</td><td>Add C-D roadway</td><td colspan="2"></td><td>Add C-D roadway</td><td></td></tr>
<tr><td colspan="2"></td><td>Add loop off-ramp</td><td colspan="2"></td></tr>
<tr><td></td><td>Add outer connection off-ramp</td><td colspan="3"></td></tr>
<tr><td>Add directional off-ramp</td><td colspan="4"></td></tr>
<tr><td rowspan="3">Freeway</td><td colspan="5">Divert demand by improving accessibility on some other route</td></tr>
<tr><td colspan="5">Lengthen upstream weaving area or deceleration lane</td></tr>
<tr><td></td><td colspan="4">Add upstream auxiliary lane</td></tr>
<tr><td rowspan="5">Ramp</td><td colspan="5">Eliminate ramp</td></tr>
<tr><td></td><td colspan="4">Add lanes to ramp</td></tr>
<tr><td></td><td colspan="2">Channelize turning movements</td><td colspan="2"></td></tr>
<tr><td colspan="2"></td><td colspan="3">Lengthen ramp</td></tr>
<tr><td colspan="4"></td><td>Move ramp upstream</td></tr>
<tr><td rowspan="11">Arterial</td><td></td><td colspan="2">Optimize existing signal</td><td></td><td>Optimize existing signal</td></tr>
<tr><td></td><td colspan="2">Install signal</td><td></td><td>Install signal</td></tr>
<tr><td colspan="5">Lengthen acceleration lane on arterial</td></tr>
<tr><td colspan="5">Add auxiliary lane on arterial</td></tr>
<tr><td></td><td colspan="4">Close or move access point to major generator</td></tr>
<tr><td colspan="5">Make arterial one-way</td></tr>
<tr><td colspan="5">Other improvements to increase capacity of arterial (see table 6 for details)</td></tr>
<tr><td colspan="4"></td><td>Make frontage road one-way</td></tr>
<tr><td colspan="4"></td><td>Connect ramp directly to crossroad</td></tr>
<tr><td colspan="4"></td><td>Discontinue frontage road</td></tr>
<tr><td colspan="4"></td><td>Channelize frontage road to prevent weaving</td></tr>
</table>

TABLE 5

DELAYS AND ACCIDENTS ON ON-RAMPS

PROBLEMS
• Excessive delay on on-ramp • Congestion on arterial related to on-ramp backup • Rear end accident on on-ramp • Crossroad accidents related to off-ramp backup • Merging accidents (ramp to freeway)
FUNCTIONAL OBJECTIVES OF SOLUTIONS
• Increase capacity of ramp or freeway ramp terminal • Provide space available for vehicle storage off the arterial • Reduce traffic volume on on-ramp • Reduce conflicts between mainline freeway and on-ramp vehicles

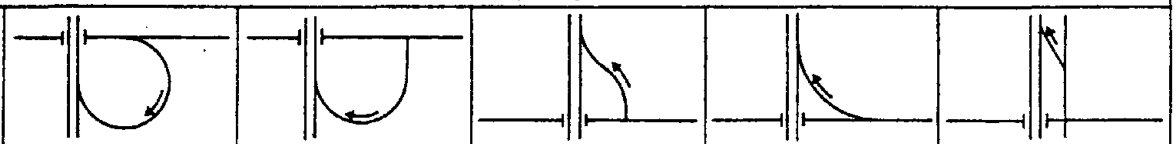
<table>
<tr><th colspan="6">LIST OF POTENTIAL SOLUTIONS</th></tr>
<tr><td></td><td colspan="5"></td></tr>
<tr><td></td><td>Free-Flow Loop</td><td>Controlled-Flow Loop</td><td>Diamond Ramp</td><td>Outer Connection Ramp</td><td>Frontage Road Slip Ramp</td></tr>
<tr><td rowspan="4">Interchange</td><td colspan="3">Add directional on-ramp</td><td colspan="2"></td></tr>
<tr><td></td><td>Add outer connection on-ramp</td><td colspan="3"></td></tr>
<tr><td colspan="2"></td><td>Add loop on-ramp</td><td colspan="2"></td></tr>
<tr><td>Add C-D roadway</td><td colspan="2"></td><td>Add C-D roadway</td><td></td></tr>
<tr><td rowspan="3">Freeway</td><td colspan="5">Lengthen acceleration lane or weaving area</td></tr>
<tr><td colspan="5">Divert demand by improving accessibility by some other route</td></tr>
<tr><td></td><td colspan="4">Add downstream auxiliary lane</td></tr>
<tr><td rowspan="4">Ramp</td><td colspan="5">Eliminate on-ramp</td></tr>
<tr><td colspan="2"></td><td colspan="3">Add lanes to on-ramp</td></tr>
<tr><td colspan="2"></td><td>Lengthen on-ramp</td><td></td><td>Lengthen on-ramp</td></tr>
<tr><td colspan="4"></td><td>Move ramp downstream</td></tr>
<tr><td rowspan="6">Arterial</td><td colspan="5">Add right turn deceleration lane</td></tr>
<tr><td colspan="5">Lengthen right-turn lane storage</td></tr>
<tr><td colspan="5">Add auxiliary lane</td></tr>
<tr><td></td><td colspan="2">Add left-turn lane</td><td colspan="2"></td></tr>
<tr><td></td><td colspan="2">Lengthen left-turn lane storage</td><td colspan="2"></td></tr>
<tr><td colspan="2"></td><td>Make arterial one-way</td><td></td><td>Make arterial one-way</td></tr>
</table>

TABLE 6

DELAYS AND ACCIDENTS ON ARTERIAL CROSSROAD

PROBLEMS

- Excessive delay for through traffic on arterial (at ramp terminals and adjacent intersections)
- Excessive delay for left and right turns from arterial (at ramp terminals and adjacent intersections)
- Rear-end, left-turn, and right-turn accidents on arterial (at ramp terminals and adjacent intersections)

FUNCTIONAL OBJECTIVES OF SOLUTIONS

- Increase intersection or ramp terminal capacity
- Reduce traffic volumes
- Define and separate conflict areas
- Reduce vehicle speed at conflict points
- Increase visibility

LIST OF POTENTIAL SOLUTIONS

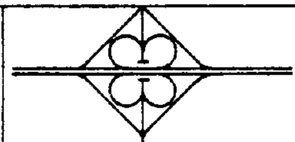
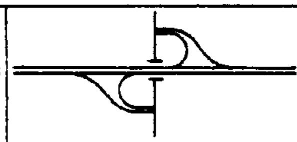
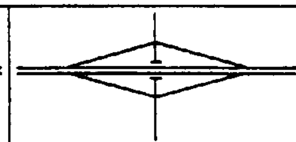
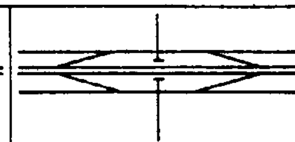
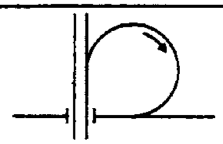
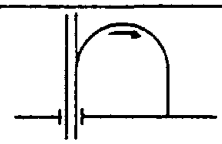
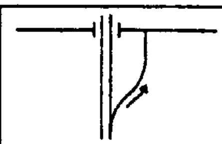
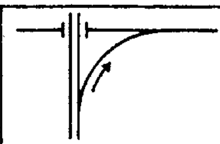
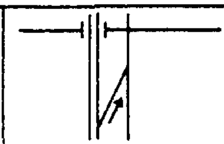
<table>
<tr><th></th><th>
Full Cloverleaf</th><th>
Partial Cloverleaf</th><th>
Diamond</th><th>
Slip Ramp Diamond</th></tr>
<tr><td rowspan="3">Interchange</td><td></td><td></td><td>Add loop on-ramp</td><td></td></tr>
<tr><td></td><td>Add outer connection on-ramp</td><td></td><td></td></tr>
<tr><td>Add directional on-ramp</td><td></td><td>Add directional on-ramp</td><td></td></tr>
<tr><td rowspan="3">Ramp</td><td></td><td colspan="3">Channelize turning movements</td></tr>
<tr><td colspan="4">Eliminate ramp</td></tr>
<tr><td colspan="4">Add lanes to on-ramps (except loops)</td></tr>
<tr><td rowspan="24">Arterial</td><td></td><td colspan="3">Optimize existing signal</td></tr>
<tr><td></td><td colspan="3">Add left turn phase</td></tr>
<tr><td></td><td colspan="3">Add all-red clearance phase</td></tr>
<tr><td></td><td colspan="3">Provide signal progression on arterial</td></tr>
<tr><td></td><td colspan="3">Install signal</td></tr>
<tr><td></td><td colspan="3">Install all-way stop</td></tr>
<tr><td></td><td colspan="3">Remove corner sight obstructions</td></tr>
<tr><td></td><td colspan="3">Increase curb radius for right-turns</td></tr>
<tr><td></td><td colspan="3">Add left-turn lane to arterial</td></tr>
<tr><td></td><td colspan="3">Add double left-turn lane to arterial</td></tr>
<tr><td></td><td colspan="3">Lengthen left-turn storage</td></tr>
<tr><td></td><td colspan="3">Increase distance between ramp terminals</td></tr>
<tr><td colspan="4">Reduce speed limit on arterial</td></tr>
<tr><td colspan="4">Add through lanes to arterial</td></tr>
<tr><td colspan="4">Add auxiliary lanes on arterial</td></tr>
<tr><td colspan="4">Increase distance between ramp terminals and nearby intersection</td></tr>
<tr><td>Lengthen weaving area</td><td></td><td></td><td></td></tr>
<tr><td colspan="3">Eliminate lane drop</td><td></td></tr>
<tr><td></td><td></td><td></td><td>Widen frontage road</td></tr>
<tr><td colspan="4">Close or move access point to major generator</td></tr>
<tr><td></td><td></td><td colspan="2">Make arterial one-way</td></tr>
<tr><td colspan="4">Install/improve lighting</td></tr>
<tr><td colspan="4">Use any of above to improve capacity of nearby intersection</td></tr>
</table>

TABLE 7

SINGLE-VEHICLE ACCIDENTS ON RAMPS

PROBLEMS
• Single vehicle accidents on ramps
FUNCTIONAL OBJECTIVES OF SOLUTIONS
• Reduce likelihood of leaving roadway • Reduce consequences of leaving roadway

LIST OF POTENTIAL SOLUTIONS					
					
	Free-Flow Loop	Controlled-Flow Loop	Diamond Ramp	Outer Connection Ramp	Frontage Road Slip Ramp
Freeway	• Lengthen deceleration lane or weaving area of off-ramp •				
	• Improve directional signing •				
Ramp	• Improve roadside design* •				
	• Improve horizontal and/or vertical alignment •				
	• Increase superelevation •				
	• Improve drainage at cross-slope transitions •				
	• Increase sight distance •				
	• Add shoulders •				
	• Post advisory speed limit •				
	• Increase skid resistance •				

*Remove curbing, unnecessary fixed objects, unnecessary guardrail, flatten slopes, and install warranted guardrail, crash cushions and breakaway devices.

TABLE 8

HEAD-ON ACCIDENTS ON TWO-WAY RAMPS

PROBLEMS
• Head-on accidents on two-way ramps
FUNCTIONAL OBJECTIVES OF SOLUTIONS
• Physically separate movements in opposing directions

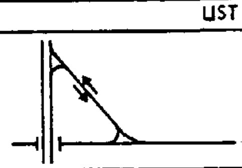
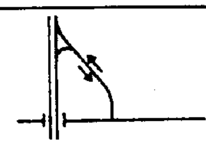
LIST OF POTENTIAL SOLUTIONS		
		
	Cloverleaf	Partial Cloverleaf
Ramp	• Install median and/or median barrier •	
	• Reconstruct ramps on separate alignments •	

TABLE 9

MERGING ACCIDENTS AT ENTRANCE TO ON-RAMPS

PROBLEMS
• Merging accidents involving right- and left-turns at entrance to on-ramp or frontage road
FUNCTIONAL OBJECTIVES OF SOLUTIONS
• Separate conflicting movements in space • Separate conflicting movements in time • Reduce traffic volumes in conflict

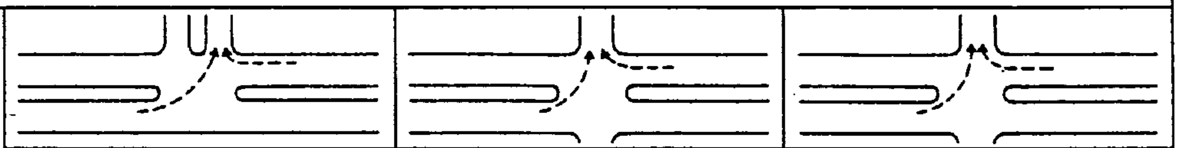
<table>
<tr><th colspan="4">LIST OF POTENTIAL SOLUTIONS</th></tr>
<tr><td></td><td colspan="3"></td></tr>
<tr><td></td><td>Partial Cloverleaf Ramp</td><td>Diamond Ramp</td><td>Frontage Road</td></tr>
<tr><td>Interchange</td><td colspan="2">Add loop or directional ramp</td><td></td></tr>
<tr><td rowspan="2">Ramp</td><td colspan="2">Improve channelization at ramp terminal</td><td></td></tr>
<tr><td colspan="2">Add lane to on-ramp</td><td></td></tr>
<tr><td rowspan="6">Arterial</td><td colspan="3">Make arterial one-way</td></tr>
<tr><td colspan="3">Add right-turn lane on arterial</td></tr>
<tr><td colspan="3">Signalize with protected left-turn phase</td></tr>
<tr><td colspan="3">Signalize with protected left-turn phase and prohibit RTOR</td></tr>
<tr><td colspan="2"></td><td>Improve channelization of frontage road intersection</td></tr>
<tr><td colspan="2"></td><td>Widen frontage road</td></tr>
</table>

TABLE 10

WRONG-WAY ACCIDENTS

PROBLEMS
• Wrong-way accidents on ramp or freeway
FUNCTIONAL OBJECTIVES OF SOLUTIONS
• Discourage wrong-way ramp entries

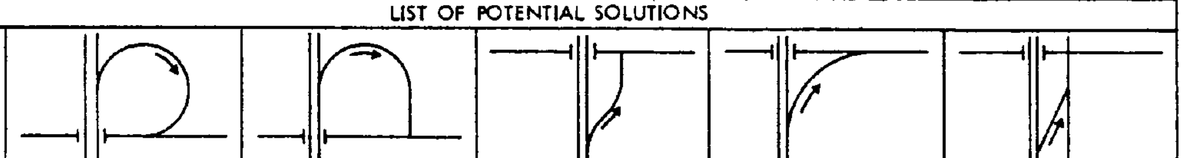
<table>
<tr><th colspan="6">LIST OF POTENTIAL SOLUTIONS</th></tr>
<tr><td></td><td colspan="5"></td></tr>
<tr><td></td><td>Free-flow Loop</td><td>Controlled-flow Loop</td><td>Diamond Ramp</td><td>Outer Connection Ramp</td><td>Frontage Road Slip Ramp</td></tr>
<tr><td>Interchange</td><td colspan="5">Provide all movements at partial interchanges</td></tr>
<tr><td rowspan="5">Ramp</td><td colspan="5">Realign off-ramp terminal channelization</td></tr>
<tr><td colspan="5">Improve signing</td></tr>
<tr><td colspan="5">Install active warning devices</td></tr>
<tr><td colspan="4"></td><td>Make frontage road one-way</td></tr>
<tr><td colspan="4"></td><td>Move ramp upstream</td></tr>
</table>

TABLE 11

DELAYS AND ACCIDENTS WITHIN MAINLINE FREEWAY WEAVING AREAS

<table>
<tr><th>PROBLEMS</th></tr>
<tr><td>• Congestion within mainline freeway weaving areas
• Accidents within mainline freeway weaving areas</td></tr>
<tr><th>FUNCTIONAL OBJECTIVES OF SOLUTIONS</th></tr>
<tr><td>• Eliminate weaving area
• Lengthen weaving area
• Separate weaving area from through lanes
• Reduce traffic volume</td></tr>
</table>

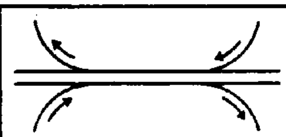
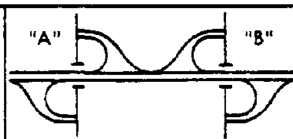
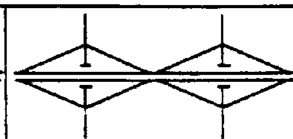
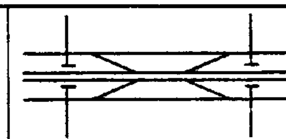
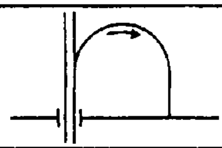
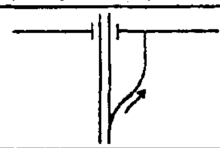
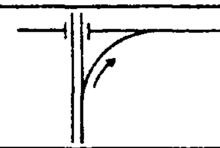
<table>
<tr><th colspan="5">LIST OF POTENTIAL SOLUTIONS</th></tr>
<tr><td></td><td></td><td></td><td></td><td></td></tr>
<tr><td></td><td>Within Full Cloverleaf</td><td>Between Adjacent Parclos</td><td>Between Adjacent Diamonds</td><td>Between Adjacent Slip Ramp Diamonds</td></tr>
<tr><td rowspan="7">Interchange</td><td colspan="3">Add C-D roadway</td><td></td></tr>
<tr><td></td><td></td><td>Add loop ramp to decrease existing on-ramp volume</td><td></td></tr>
<tr><td></td><td></td><td>Add frontage roads connecting adjacent crossroads</td><td></td></tr>
<tr><td></td><td colspan="2">Convert to Parclo AB design</td><td></td></tr>
<tr><td></td><td colspan="3">Create "braided" ramps using grade separation structures</td></tr>
<tr><td></td><td></td><td></td><td>Reverse order of on- and off-ramps</td></tr>
<tr><td colspan="4">Other conversions of interchange configurations</td></tr>
<tr><td rowspan="5">Freeway</td><td colspan="4">Divert demand by increasing accessibility on some other route</td></tr>
<tr><td colspan="4">Add auxiliary lane in weaving area</td></tr>
<tr><td colspan="4">Add recovery tapers</td></tr>
<tr><td>Add auxiliary lane prior to on-ramp gore</td><td></td><td></td><td></td></tr>
<tr><td>Add auxiliary lane beyond off-ramp gore</td><td></td><td></td><td></td></tr>
<tr><td rowspan="2">Ramp</td><td colspan="4">Eliminate ramps</td></tr>
<tr><td colspan="4">Realign ramps to create longer weaving areas</td></tr>
</table>

TABLE 12

DELAYS AND ACCIDENTS BETWEEN RAMP TERMINAL AND ADJACENT INTERSECTION

<table>
<tr><th colspan="4">PROBLEMS</th></tr>
<tr><td colspan="4">• Congestion on arterial between ramp terminal and adjacent intersection (or major driveway)
• Rear-end and angle accidents on arterial between ramp terminal and adjacent intersection (or major driveway)</td></tr>
<tr><th colspan="4">FUNCTIONAL OBJECTIVES OF SOLUTIONS</th></tr>
<tr><td colspan="4">• Increase separation between ramp terminal and intersection
• Increase capacity of ramp terminal and intersection
• Create gaps in arterial traffic
• Reduce traffic volume</td></tr>
<tr><th colspan="4">LIST OF POTENTIAL SOLUTIONS</th></tr>
<tr><td></td><td>
Controlled-Flow Loop</td><td>
Diamond Ramp</td><td>
Outer Connection</td></tr>
<tr><td>Freeway</td><td colspan="3">Divert demand by improving accessibility by some other route</td></tr>
<tr><td rowspan="9">Ramp</td><td colspan="3">Eliminate ramps</td></tr>
<tr><td colspan="3">Move ramp terminal further from intersection</td></tr>
<tr><td colspan="3">Realign ramp to intersect frontage road rather than arterial crossroad</td></tr>
<tr><td colspan="2"></td><td>Realign ramp to intersect arterial at 90°</td></tr>
<tr><td colspan="2"></td><td>Realign ramp and signalize ramp terminal</td></tr>
<tr><td colspan="2">Install signal at ramp terminal</td><td></td></tr>
<tr><td colspan="2">Retime signal at ramp terminal</td><td></td></tr>
<tr><td colspan="2">Widen ramp</td><td></td></tr>
<tr><td colspan="2">Provide turn lanes on ramp</td><td></td></tr>
<tr><td rowspan="9">Arterial</td><td colspan="3">Eliminate intersection</td></tr>
<tr><td colspan="3">Move intersection further from ramp terminal</td></tr>
<tr><td colspan="3">Retime signal at intersection</td></tr>
<tr><td colspan="3">Provide signal progression on arterial</td></tr>
<tr><td colspan="3">Provide auxilliary lane between ramp terminal and intersection</td></tr>
<tr><td colspan="3">Install left-turn lane at intersection</td></tr>
<tr><td colspan="3">Install double left-turn lane at intersection</td></tr>
<tr><td colspan="3">Widen arterial</td></tr>
<tr><td colspan="3">Install channelization to prohibit weaving</td></tr>
</table>

The charts can be used in the following manner. First, review all 9 charts to select the one(s) appropriate for the operational or safety problems at the interchange in question. Second, select the column appropriate for the type of ramp or interchange configuration at the problem location. Third, review all solutions in the selected column. Solutions which are not physically feasible at the particular location or are not appropriate for the site-specific problem can be eliminated at this stage.

Judgment must be exercised in interpreting these charts since, for example, installation of a signal is not an appropriate countermeasure at a location that is already signalized. Rather than introducing more classifications to the charts, such as signalized vs. unsignalized ramp terminals, it is assumed that the user is capable of eliminating such inappropriate solutions very quickly.

It should be recognized that no set of charts, such as those presented in Tables 4-12, can present every solution appropriate for a particular problem and location. The scope of the charts has been limited to improvements that preserve or are closely related to the existing interchange configuration. Although major rebuilding could alleviate a problem by a complete change in the interchange configuration, this is often infeasible in urban and suburban areas due to right-of-way restrictions. In addition, some interchanges will always present unusual geometric or traffic conditions that cannot be generalized. For these reasons, the charts also identify the functional objectives of the solutions presented. These functional objectives are the basic principles that underly the solutions and these basic principles are applicable even when the specific solutions in the charts are not. Additional solutions will often be apparent if the appropriate functional objectives are kept in mind. It is recommended that--in every case--the engineer should consider whether solutions not included in the charts are physically feasible and appropriate to the problem.

In the final step, the solutions that have been identified should be used--alone or in combination with one another--to define a set of formal alternatives for consideration in the design process. Whenever possible, preference should be given to alternatives that treat the basic cause of a problem rather than merely its symptoms. For example, where traffic backs onto the mainline freeway lanes from a diamond ramp, the basic problem is usually the capacity of the crossroad ramp terminal and solutions which increase its capacity are preferable to those that merely increase the vehicle storage available on the ramp.

There is a strong temptation to introduce budgetary constraints at this stage and eliminate the more costly alternatives. However, the retention of all reasonable alternatives and their analysis in more or less depth, as described in Sections VII and VIII, is recommended to ascertain the costs and relative merits of the alternatives. Even if funds are not immediately available to construct the "ultimate" improvement for the interchange, it may be possible to construct a smaller project which contributes to the "ultimate" solution through staged construction. For example, widening of a diamond off-ramp to increase vehicle storage may contribute to a later capacity improvement of the crossroad ramp terminal.

B. General Classes of Problems and Solutions

The 9 charts used to identify alternative interchange improvements each address one or more related operational or safety problems that frequently occur at freeway-arterial interchanges. The nature of these 9 sets of problems and their general solutions are briefly discussed in this section. The problems included in the charts illustrate the critical role of the arterial crossroad and crossroad ramp terminals as the source of problems at freeway-arterial interchanges. A review of 40 interchange rehabilitation projects, presented in Volume III, found that 85 percent of the projects involved improvements to the crossroad ramp terminals and that most improvements to the arterial crossroad and the ramps were related to the ramp terminals. Consequently, the charts indicate a heavy emphasis on solutions involving the crossroad and ramps.

1. Off-ramp delays and rear-end accidents: A common problem of freeway-arterial interchanges is the back up of traffic on an off-ramp resulting from insufficient capacity of the crossroad ramp terminal relative to the ramp volume. While most typical of diamond and partial cloverleaf interchanges, this problem could conceivably occur at any freeway-arterial interchange. The capacity deficiency at the crossroad ramp terminal can lead both to excessive delay for off-ramp motorists and to rear-end accidents at the end of the queue. The problem is generally considered most severe if the queue of vehicles extends into the through lanes of the mainline freeway. Table 4 presents a variety of improvements to the interchange configuration and to individual interchange elements each of which is intended to increase capacity, increase vehicle storage or reduce traffic volume. Judgment must be used in interpreting this chart because, for example, geometric improvements to the mainline freeway are appropriate only when the queue of vehicles extends beyond the off-ramp gore. As noted in the lower portion of the chart, capacity improvements to the arterial crossroad (presented in Table 6), can also contribute to the solution of these problems because such solutions can increase the capacity (green-to-cycle time ratio) for the off-ramp approach.

The countermeasure "optimize existing signal," which appears in Table 4 and several other tables, refers to a variety of signal improvements including adjusting timing, relocation of signal heads, installation of mast arms, installation of visors and/or back plates, etc.

2. On-ramp delays and accidents: An analogus problem can also occur when there is insufficent capacity for traffic entering the mainline freeway from an on-ramp. Such problems can arise from geometric constraints or from extremely heavy freeway volumes. (The solution of problems caused by heavy freeway volumes are not addressed directly here because they are not interchange-related, but some solutions presented here could be used to accomodate, rather than solve, freeway-related problems).

Merging accidents involving vehicles entering the mainline freeway are also included here because many of the same countermeasures are effective. Table 5 illustrates appropriate solutions for five on-ramp configurations. These countermeasures include those that increase capacity, increase vehicle storage, reduce traffic volume and reduce traffic conflicts. Geometric improvements to the arterial crossroad are appropriate only when the queue of off-ramp vehicles extends onto the crossroad.

3. Arterial crossroad delays and accidents: Table 6 addresses the solution of a variety of common interchange problems including delays for through, right-turn and left-turn vehicles on the arterial crossroad and several types of accidents at crossroad ramp terminals including rear-end, right-turn, and left-turn accidents. For these problems, it is most convenient to classify solutions based on the overall interchange configuration rather than the configuration of an individual ramp. There are a large number of conventional traffic engineering solutions applicable to interchange configurations with at-grade ramp terminals--partial cloverleafs, diamonds, and slip-ramp diamonds. However, Table 6 illustrates graphically that there are very few solutions appropriate for the full cloverleaf configuration. In many cases, the capacity of the crossroad in the interchange area may be constrained by an adjacent intersection (such as a frontage road) as well as by the ramp terminal. Most of the solutions in Table 6 are also applicable to frontage road intersections. A common problem involving a frontage road intersection located very close to a crossroad ramp terminal is treated as a special case in Table 12.

4. Single-vehicle accidents on ramps: One common interchange problem that does not directly involve the crossroad ramp terminals is the single vehicle run-off-road accident. This problem is often identified by observation of erratic maneuvers or roadside damage rather than by accident records. There has been considerable effort in recent years to decrease the frequency and, especially, the severity of roadside accidents based on the principles of the AASHTO Yellow Book [1] including gore area improvements, barrier installation, breakaway devices, etc. Table 7 presents ten general countermeasures to combat single vehicle run-off-road accidents. The functional objectives of these countermeasures are either to reduce the likelihood or consequences of leaving the roadway. The same set of countermeasures applies to all types of ramps.

5. Head-on accidents on two-way ramps: Some older cloverleaf and partial cloverleaf interchanges have two-way ramps where the two directions of travel are separated by either a painted centerline or a very narrow median. Such locations have the potential for head-on accidents, which can be alleviated by installation of a median and/or median barrier or by reconstruction of the ramps on separate alignments, as indicated in Table 8. (This design is also susceptable to wrong-way movements onto the freeway or arterial--see Table 10.)

6. Merging accidents at entrance to on-ramp or frontage road: Merging accidents can occur at an at-grade crossroad ramp terminal or frontage road intersection as right-turn and left-turn vehicles from the crossroad come into conflict. The functional objectives of solutions for this problem involve separating the conflicting vehicles in space or in time, or reducing the traffic volumes in conflict. The solutions, given in Table 9, include addition of another ramp to divert traffic; improved channelization of the ramp terminal or frontage road intersection; widening of the on-ramp or frontage road; signalization with a protected left-turn phase; and, converting the arterial crossroad to one-way operation.

7. Wrong-way accidents on ramp or freeway: Wrong-way accidents can occur on an off-ramp or mainline freeway when arterial crossroad vehicles enter an off-ramp in the wrong direction. Countermeasures for this type of accident, given in Table 10, are intended to discourage wrong-way entries by provision of all movements at partial interchanges; realignment of off-ramp terminal channelization (islands and medians); signing improvement; installation of active warning devices; making frontage roads one-way; and moving slip ramps upstream.

8. Congestion and accidents with mainline freeway weaving areas: A predominant interchange-related problem on mainline freeways is congestion and accidents in weaving areas within full cloverleaf interchanges or between adjacent interchanges. The functional objectives for solution of these problems include elimination of the weaving area; lengthening the weaving area; separating the weaving area from the through lanes; or reducing the traffic volumes. Table 11 presents a variety of freeway, ramp, and interchange-configuration-related solutions to the problems of traffic operations in weaving areas.

9. Congestion and accidents on arterial between ramp terminal and adjacent intersection: A common interchange-related problem occurs when a frontage road intersection (or major driveway) is located in close proximity to a ramp terminal. The operation of the ramp terminal and frontage road intersections may interfere with one another, and congestion and accidents may occur at both locations and between them on the arterial crossroad. Such problems occur in two distinct, but closely related, situations.

The first situation can occur between the free-flow outer connection off-ramp of a full cloverleaf and a signalized frontage road intersection. In the weaving area between the ramp terminal and the signal, crossroad vehicles change lanes to the right to turn right onto the frontage road and ramp vehicles change lanes to the left to turn left onto the frontage road or go straight on the arterial crossroad. Weaving maneuvers in this area are often constrained by queues of vehicles that form when the frontage road signal is red. This situation illustrates the basic incompatibility between the free-flow nature of the ramp and the interrupted-flow nature of the signalized frontage road intersection.

The second situation occurs between a controlled-flow off-ramp terminal, such as found in a diamond or partial cloverleaf interchange, and a frontage road intersection. The second situation differs from the first because both intersections may be signalized. However, weaving problems may still occur on the arterial crossroad between the intersections and, if the ramp-terminal has a free-flow right-turn lane, traffic operational problems identical to those described for the first situation may occur.

Table 12 illustrates several improvements to the freeway, ramp and arterial crossroad that can be employed to alleviate these problems. The functional objectives of these solutions are to increase the separation between ramp terminals and intersections; increase the intersection capacity; create gaps in arterial traffic for weaving vehicles; and reduce traffic volumes.

VII. QUANTIFY EFFECTS OF IMPROVEMENT ALTERNATIVES

Rational choices between alternative improvements must be based on a comparison of the anticipated effects of those alternatives. This section of the report presents procedures to quantify the effects of interchange rehabilitation alternatives in two distinct areas: traffic operations and traffic safety.

The emphasis is placed on quantifying traffic operational and safety effects because these are most directly related to the objectives of interchange rehabilitation projects. Noise and air pollution effects are discussed briefly because these effects may be important considerations in particular interchange rehabilitation projects. The scope of this section is limited to quantification of the traffic operational and safety effects for the existing condition and each rehabilitation alternative. The comparison of alternatives on the basis of these effects is addressed in the next section. Appendix E presents several examples of operational and safety analyses and comparison of alternatives.

A. Traffic Operational Effects

Many interchange rehabilitation projects provide significant benefits to motorists through traffic operational improvements. Operational benefits result from improvements that either increase the capacity or decrease the traffic demand for any portion of the interchange. These benefits include both reduced delay and reduced vehicle operating costs.

The recommended procedures to quantify these effects are based primarily on the AASHTO Manual on User Benefit Analysis of Highway and Bus-Transit Improvements-1977.[2] The procedures presented here illustrate the specific application of the AASHTO manual to interchange rehabilitation projects. Portions of the manual that are not applicable, such as correction factors for vehicle operating costs for gravel roads, have been omitted; and suggested procedures have been added for geometric situations not covered by the AASHTO manual, such as for traffic operations on freeway ramps. The analyses to determine travel time and vehicle operating costs have been combined into a single step-by-step procedure that utilizes several nomographs from the AASHTO manual, which are presented in Appendix A. For a justification of the development and accuracy of these nomographs and procedures, the reader should refer to the manual itself.

The user should first assemble all available operational data on the existing condition and each alternative to be analyzed. The required input data for operational analyses include the length, cross-section curvature, profile, traffic volume, traffic mix, and design speed (or average running speed) for each ramp and roadway section within the interchange. With these variables, the AASHTO procedure can realistically reflect the differences between alternatives. For example, widening of a roadway would be expected to increase average running speed, thus reducing travel time and either increasing or reducing vehicle operating costs depending on the speeds and delays involved. Addition of a loop ramp to a diamond interchange would increase vehicle speeds, but would also increase the distance traveled by some vehicles. In most cases, the analysis procedure should be applied both to the existing traffic volumes and future (design year) traffic volumes.

Because each ramp and roadway section is considered separately, manual application of the recommended procedure can be quite tedious. Ideally, the analysis of a major interchange reconstruction project might require consideration of four improvement alternatives (including the "do-nothing" alternative or existing condition); division of the interchange into 22 operationally-homogeneous analysis units; and separate consideration of three vehicle types during four periods of the day for two analysis years. Thus, a single interchange evaluation could require 4 x 22 x 3 x 4 x 2 = 2,112 separate calculations. Because of the repetitive nature of the calculations, the user applying manual evaluations must make as many simplifying assumptions as possible. Potential simplifications include:

- Reducing the number of periods of the day considered from four to two (peak/off-peak or morning peak/evening peak);

- Assuming similar operating conditions for similar portions of the interchange; e.g., analyzing one off-ramp and assuming the same travel times and operating cost for others; and

- Analyzing only those portions of the interchange directly affected by the improvement.

Good judgment must be exercised in making such assumptions to avoid eliminating important differences between alternatives. In the long run, practical application of the recommended procedures within a highway agency could be assisted by a computer program to perform the calculations automatically and display the results in a convenient format.

Delay effects are determined from the differences between alternatives in the total travel time for all or a portion of the interchange area. Procedures are provided to determine the travel time for roadway segments at the average running speed of traffic and to determine the added travel time due to stopping and idling at intersections.

Vehicle operating costs include the costs incurred by motorists for fuel, engine oil, tires, depreciation, and maintenance and repair. The AASHTO manual procedures allow the engineer to determine the vehicle operating costs for tangent sections and to estimate the added costs for speed changes and transitions, curves, and stopping and idling at intersections. The operating costs determined by the procedures are appropriate for January 1975 price levels and average 1975 passenger car fuel consumption. Additional procedures are provided to adjust these operating costs to current or projected future passenger car fuel consumption and prices.

One potential drawback of the AASHTO procedures is that they cannot account explicitly for every possible operational situation found at freeway-arterial interchanges. We have suggested that interchange ramps be treated using the procedures for urban arterials. However, no procedures are available to evaluate delays in weaving areas or merging areas under congested conditions.

Another potential drawback of the AASHTO procedures is that the fuel consumption for each alternative is never determined explicity in litres or gallons. Instead, the vehicle operating costs (in dollars) include the cost of fuel--gasoline for passenger cars and single-unit trucks, diesel fuel for combination trucks. Although energy consumption can be correctly accounted for in benefit-cost analyses through the vehicle operating costs, fuel consumption savings (expressed in litres or gallons) may become an important measure of effectiveness for interchange rehabilitation projects in the future, as energy conservation becomes an increasingly prominent national priority.

1. Traffic operational analysis procedure: The operational analysis procedure based on the AASHTO manual is presented below in step-by-step fashion. The nomographs referred to in the text are found in Appendix A.

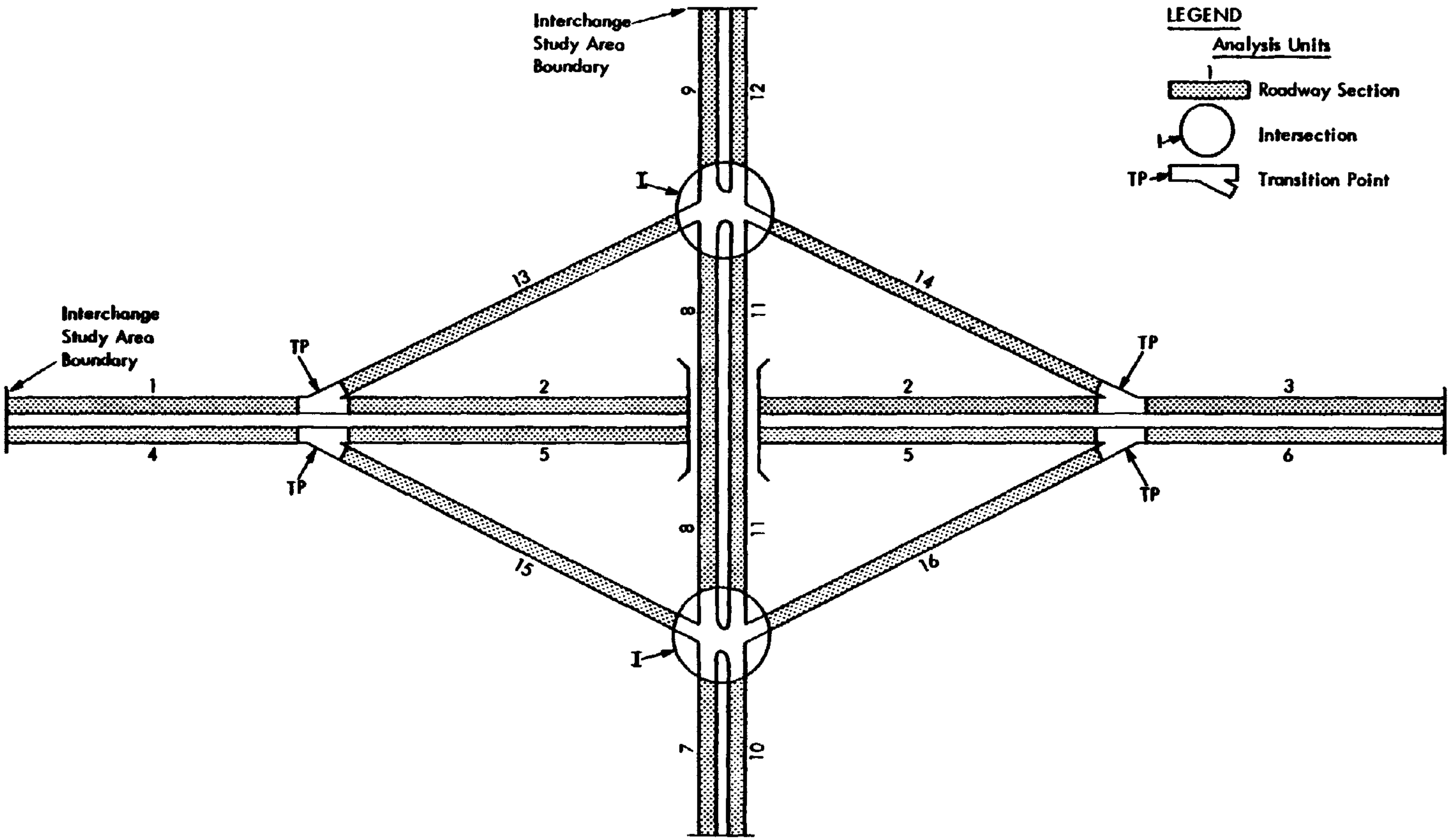


Figure 7 - Diamond Interchange Illustrating Operational Analyses Units.

Step 1 - Establish boundaries for the interchange study area on both the mainline freeway and the arterial crossroad. The same boundaries should be used for the analysis of the existing condition and all improvement alternatives. These boundaries should include all areas that will be directly affected by any of the alternatives. For a minor rehabilitation project, such as ramp widening, the interchange study area may be limited to a particular ramp and adjacent ramp terminals. For major reconstruction of the interchange, the study area should include the entire interchange, as illustrated in Figure 7.

Step 2 - Divide the interchange study area into analysis units. The three kinds of analysis units to be considered are:

- Roadway sections;
- Intersections; and
- Transition points.

A roadway section is a continuous segment of roadway in one direction of travel with reasonably uniform operational and geometric characteristics. Thus, roadways must be analyzed separately in each direction. (This restriction applies to both divided and undivided roadways.) Mainline freeway segments, arterial crossroad segments, and ramps should all be treated as roadway sections. For computational efficiency, roadway sections should be as long as possible, consistent with the need to minimize variation in traffic volume, traffic mix, design speed, average running speed, capacity, horizontal curvature, and grade within each section.

Intersections are locations where traffic streams cross at grade. For example, a crossroad ramp terminal controlled by STOP signs or a traffic signal would be treated as an intersection. If a crossroad-frontage road intersection were within the interchange study boundaries, it would also be considered. For analysis purposes, each intersection approach is considered separately.

Transition points are used to represent locations where changes in the speed of freely-flowing traffic occur. A ramp merging with or diverging from a freeway is an example of a transition point. A crossroad ramp terminal with free-flow or YIELD-sign-controlled merging (e.g., a cloverleaf loop or outer connection ramp) would also be treated as a transition point as would a boundary between two analysis units with different running speeds. However, any location where many or most vehicles come to a complete stop, rather than merely slowing down or speeding up, cannot be treated as a transition point.

Figure 5 illustrates the minimum set of 22 analysis units needed to evaluate travel time and vehicle operating costs for a conventional diamond interchange. These include 6 mainline freeway sections, 6 arterial crossroad sections, 4 ramp sections, 4 mainline ramp terminals (transition points) and 2 crossroad ramp terminals (intersections). Additional analysis units could be required if operational or geometric conditions vary markedly within a given unit.

Step 3 - Establish the time periods of the day to be evaluated and the length of each period. Most interchange situations can be represented adequately by a maximum of four time periods:

- Morning peak period;
- Evening peak period;
- Off-peak period (daytime); and
- Off-peak period (nighttime).

In general, only weekday conditions are evaluated although weekend conditions could be included for interchanges impacted by recreational traffic, regional shopping centers, or special events. In many cases, further simplifications may be possible, such as eliminating the off-peak periods if it has been established that the major operational differences between alternatives occur in the peak periods.

ROADWAY SECTIONS

Step 4 - Assemble the following data for each roadway section in each time period:

- Facility type: Procedures have been established for four facility types: freeways, multilane highways, two-lane highways, and urban arterials. No procedures have been established by AASHTO for interchange ramps, but it is recommended that they be treated as urban arterials.

- Length of section (L).

- Design speed (or average running speed).

- Traffic volume in vehicles/hour (V): For analysis of the base year, the traffic volumes can be the existing volumes or a short-term projection based on uniform growth of the existing volumes. For analysis of the design year, a 20-year forecast of traffic volume is usually needed and this forecast should consider potential shifts in the pattern of turning movements due to development of the surrounding area.

- Traffic mix: The percentage of the traffic stream in three different vehicle classifications--passenger cars (P_1), single-unit trucks (P_2) and combination trucks (P_3)--is needed. These percentages should total 100 percent.

- Roadway capacity in vehicles/hour (C).

- Grade (percent).

- Curvature (degrees).

Step 5 - Determine the travel time for the first roadway segment in the first time period. The data needed for this determination are (1) design speed and volume-to-capacity (V/C) ratio or (2) average running speed. The travel time should be determined from one of the following nomographs, depending on the facility type:

Facility Type	Nomograph Used to Determine Travel Time
Freeway	Figure 10
Multilane Highway	Figure 11
Two-Lane Highway	Figure 12
Urban Arterial (or Ramp)	Figure 13

The travel time (TT) obtained from the nomograph is expressed in units of hours per 1,000 vehicle-kilometres. This should be converted to the total time (T) for all vehicles in the appropriate roadway section and time period, in the following manner:

$$T = \frac{(TT)(V)(L)(N)}{1,000}$$

where T = Total time for the appropriate roadway section and time period (hours),
TT = Travel time from nomograph for appropriate roadway section and time period (hours per 1,000 vehicle-kilometres),
V = Traffic volume for appropriate roadway section and time period (vehicles/hour),
L = Length of roadway section (kilometres), and
N = Duration of time period (hours).

Step 6 - Estimate the basic running costs for the appropriate section and time period separately for each vehicle type. The following table indicates the appropriate nomograph for each combination of facility type and vehicle type:

Facility Type	Vehicle Type: Passenger Car	Single Unit Truck	Combination Truck
Freeway	Figure 10	Figure 14	Figure 18
Multilane Highway	Figure 11	Figure 15	Figure 19
Two-Lane Highway	Figure 12	Figure 16	Figure 20
Urban Arterial (or Ramp)	Figure 13	Figure 17	Figure 21

Figure 8 illustrates a typical nomograph, and contains an example illustrating its use. The data needed to use the nomographs are: (1) design speed, volume-to-capacity (V/C) ratio, level of service, percent grade and degree of curvature or (2) average running speed, level of service, percent grade and degree of curvature.

The basic running cost for each vehicle type is the sum of three components: (1) the tangent running cost on grades; (2) the added running cost of curves (CRC); and (3) the added speed change cost (SCC). All three costs are determined from the nomographs in units of dollars per 1,000 vehicle-miles and combined in the following manner:

$$RC = \sum_{i=1}^{3} (TRC_i + CRC_i + SCC_i) \frac{(V)(L)(N)}{1,000} \left(\frac{Pi}{100}\right)$$

where RC = Running cost for appropriate roadway section and time period (dollars),

TRC_i = Tangent running cost on grades for i^{th} vehicle type (dollars per 1,000 vehicle-kilometres),

CRC_i = Added running cost on curves for i^{th} vehicle type (dollars per 1,000 vehicle-kilometres),

SCC_i = Added speed charge cost for i^{th} vehicle type (dollars per 1,000 vehicle-kilometres),

P_i = Percentage of traffic stream for i^{th} vehicle type,

i = 1 for passenger cars,

i = 2 for single unit trucks, and

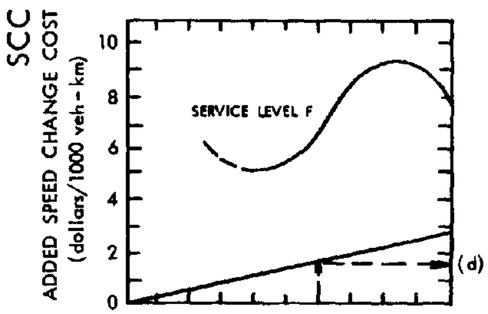
i = 3 for combination trucks.

Table 7 of the AASHTO Manual[2] provides an approximate method to account for the effects of trucks on vehicle running costs without evaluating the above expression for each vehicle type.

EXAMPLE

GIVEN:	SOLUTION:	
Vehicle Type: Passenger Car	Average Running Speed = 80 km/h	
Facility: 4-Lane Freeway	(a) Time: 13 hrs x $3.00*	$ 39
Design Speed: 112 km/h	(b) Tangent Running Cost	52
Service Level F ? No	(c) Added Running Cost Due to Curves	12
v/c Ratio: 0.6	(d) Added Running Cost Due to Speed Changes	1.40
Grade: +2%	Total Basic Section Costs per 1,000 veh-km	$104.40
Curvature: 4°		

* Assumed hourly value of time per vehicle



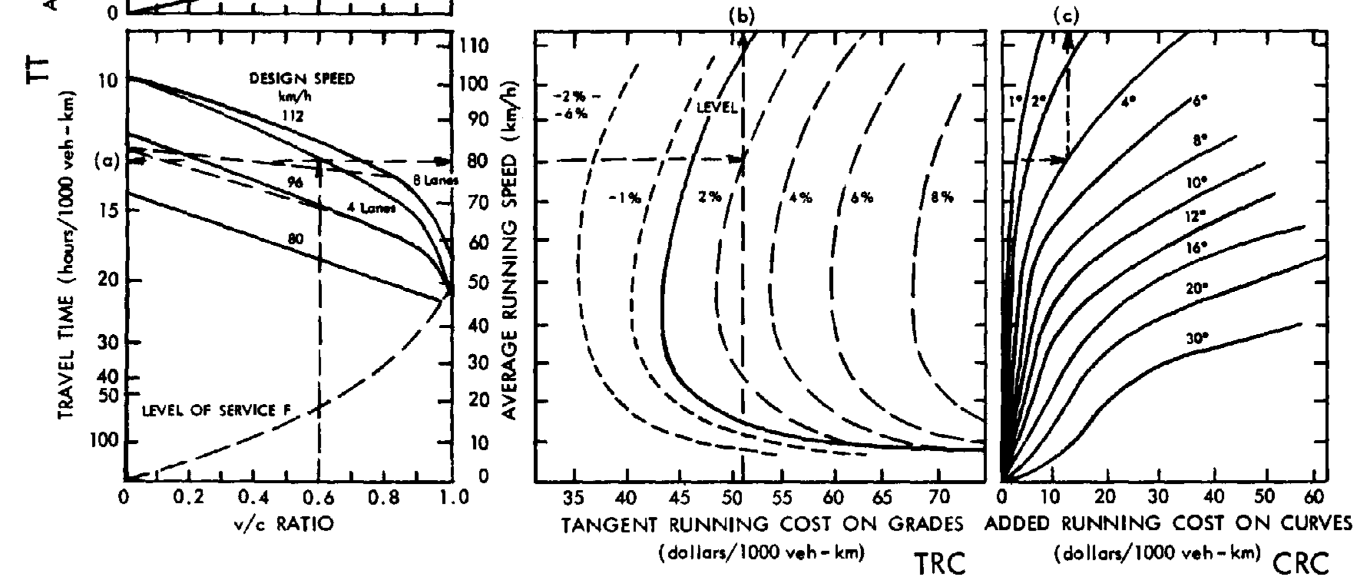


Figure 8 - Typical Nomograph Used in Operational Analyses[2]

Step 7 - Repeat Steps 5 and 6 for each roadway section in each time period.

INTERSECTIONS

Step 8 - Assemble the following data for each intersection approach in each time period:

. Traffic volume in vehicles/hour (V).

. Traffic mix (same definition as for roadway sections).

. Cycle length for traffic signals (sec).

. Green-to-cycle time ratio: The ratio of effective green time for a signal approach to the cycle length of the signal, both expressed in the same unit of time (usually seconds). For an approach controlled by a STOP sign or a flashing red beacon, use $\lambda = 0.0$.

. Saturation flow (S): The approach volume in vehicles per hour of green that is computed by the Highway Capacity Manual procedures (Chapter 6) when the load factor is 1.0 and the appropriate adjustment factors are applied (roughly 1,700-1,800 vphg times the number of lanes on the approach).

. Capacity (C): The service volume of the approach at a load factor of 1.0. Capacity is computed as the saturation flow times the green-to-cycle time ratio ($C = \lambda S$).

. Degree of Saturation (X): Also known as volume-to-capacity ratio. Computed as the ratio of the traffic volume on the approach to the capacity of the approach ($X = v/\lambda S = V/C$).

. Approach speed: Also termed the "midblock speed." This should be the average running speed used in Step 5 for analysis of the roadway section containing the intersection approach.

Step 9 - Determine, in the following manner, the added travel time (ITT) for the first intersection approach in the first time period:

a. Enter the nomograph of Figure 20 with the degree of saturation (X), the green-to-cycle time ratio (λ) and the approach speed. Determine the added stopping delay (ASD) in hours per 1,000 vehicles. The average number of stops per vehicle (SPV) should also be noted for later use in the air pollution analyses described in Section VII-C.

b. Determine the appropriate time cost adjustment factor (TCF) for trucks in the traffic stream from the table in Figure 20 based on approach speed and traffic mix.

c. For a traffic signal, enter the nomograph in Figure 21 with the capacity (C), the degree of saturation (X), the green-to-cycle time ratio (λ) and the cycle length. Using the procedures illustrated in the nomograph, determine the idling time (IT) in hours per 1,000 vehicles.*

For a STOP sign, analytical estimates or field studies are required to determine the average delay per vehicle. The nomograph in Figure 21 can be used (entering from the left) to convert average delay per vehicle in seconds to idling time (IT) in hours per 1,000 vehicles.

d. Determine the appropriate idling time adjustment factor (ITF) from the table in Figure 21. If the idling time was measured in the field for a STOP sign or the influence of trucks on the traffic stream was included in the analytical estimate, then let ITF = 1.0.

e. Compute the added travel time (ITT) due to the intersection operation as:

$$\text{ITT} = \Big((\text{ASD})(\text{TCF}) + (\text{IT})(\text{ITF}) \Big) \frac{(\text{V})(\text{N})}{1{,}000}$$

The quantity ITT, has units of hours of added travel time for the appropriate intersection approach and time period.

Step 10 - Determine the added running cost (IRC) for the intersection approach:

a. Determine the added stopping cost (ASC) in dollars per 1,000 vehicles, running cost adjustment factor (RCF), idling cost (IC), and, idling cost adjustment factor (ICF) from Figures 20 and 21 in a manner entirely analogous to Step 9, a through d.

b. Compute the intersection running cost (IRC) as:

$$\text{IRC} = \Big((\text{ASC})(\text{RCF}) + (\text{IC})(\text{ICF}) \Big) \frac{(\text{V})(\text{N})}{1{,}000}$$

* The procedure in Figure 21 is not appropriate for level of service F ($X = 1.0$) where the average delay per vehicle exceeds the signal cycle. A discussion of such cases is found in Section VII.A.2.

The quantity, IRC, has units of dollars of running cost for the appropriate intersection approach and time period.

Step 11 - Repeat Steps 9 and 10 for each intersection approach in each time period.

TRANSITION POINTS

Step 12 - For each transition point, assemble data needed to compute the transition cost including:

- Average running speed on slower section.
- Average running speed on faster section.
- Traffic volume making the speed transition in vehicles/hour (v).
- Traffic mix (same definition as for roadway sections).

At a merge or diverge point, the traffic volume making the speed transition is the entering or exiting volume. However, if the average running speed of the through traffic also changes at the merge or diverge points, a transition cost for the through volume should also be computed.

Step 13 - For the first transition point:

a. Enter Figure 22 with the speeds of the slower and faster sections to determine the transition cost (TC) in dollars per 1,000 vehicles.

b. Determine the transition cost adjustment factor (TCAF) for truck traffic from the table in Figure 22 based on the traffic mix.

c. Compute the total transition cost (TTC) in dollars as:

$$TTC = (TC)(TCAF)\,\frac{(V)(N)}{1{,}000}$$

Step 14 - Repeat Step 13 for each transition point.

ACCUMULATE TRAVEL TIMES AND RUNNING COSTS

Step 15 - Accumulate the travel time in a typical day for each roadway section and intersection in the following manner:

$$\text{Total Travel Time} = \sum_{\substack{\text{all}\\\text{time}\\\text{periods}}} \left(\sum_{\substack{\text{all}\\\text{roadway}\\\text{sections}}} T + \sum_{\substack{\text{all}\\\text{inter-}\\\text{sections}}} ITT \right)$$

Step 16 - Accumulate the running costs in a typical day for each roadway section, intersection, and transition point in the following manner:

$$\text{Total Running Cost} = \sum_{\substack{\text{all}\\\text{time}\\\text{periods}}} \left(\sum_{\substack{\text{all}\\\text{roadway}\\\text{sections}}} RC + \sum_{\substack{\text{all}\\\text{inter-}\\\text{sections}}} IRC + \sum_{\substack{\text{all}\\\text{transition}\\\text{points}}} TTC \right)$$

Step 17 - Adjust the total running cost computed in Step 16 to the appropriate analysis year. Running costs determined from the nomographs are appropriate for the January 1975 cost data used in the AASHTO manual. Several alternative procedures are available to update the running costs.

The simplest method is to multiply the total running cost determined in Step 16 by the factor:

$$\frac{CPI_{AY}}{CPI_{1975}}$$

where CPI_{AY} = Estimated private transportation component of the Consumer Price Index for the analysis year, and

CPI_{1975} = Private transportation component of the Consumer Price Index for January 1975 = 142.2.

This method is applicable if the cost increases for the vehicle running cost components from January 1975 to the analysis year are roughly proportional.

The recent dramatic increases in the cost of fuel demonstrate that the assumption of proportional increases in the components of vehicle operating costs may be unreasonable. However, this trend will be partially offset by the increase in fuel economy as newer, more fuel-efficient vehicles enter the vehicle population and older vehicles are scrapped. The following update factor for which operating costs consider both of these trends:

$$M = 0.00029(FC)\ CPI_F + 0.0001\ CPI_O + 0.0004\ CPI_T + 0.0016\ CPI_M + 0.0032\ CPI_D$$

where M = Updating factor for running costs,

FC = Average passenger car fuel economy for analysis year (kilometres/litre),

CPI_F = Consumer Price Index (private transportation--gasoline regular and premium) in analysis year,

CPI_O = Consumer Price Index (private transportation--motor oil premium) in analysis year,

CPI_T = Consumer Price Index (private transportation--tires, new tubeless) in analysis year,

CPI_M = Consumer Price Index (private transportation--auto repairs and maintenance) in analysis year, and

CPI_D = Consumer Price Index (private transportation--automobiles, new) in analysis year.

The update factor, M, is based on the running cost updating factors presented in Appendix B of the AASHTO Manual[2] and fuel consumption estimates in Table 13, obtained from Mannering and Sinha[24] and Shonka, Loebl, and Patterson[35] and is appropriate for passenger cars operating on level tangents. For sites with large truck volumes, steep grades or sharp curves, additional updating formulas are presented in Appendix B of the AASHTO Manual. Separate updating factors are provided for three vehicle types (passenger cars, single unit trucks and combination trucks) and six running cost components (level tangents, positive grades, negative grades, excess curve costs, speed change, and stopping costs and idling costs). Because of the detailed nature of these updating factors, they must be applied to individual cost elements determined from nomographs in Steps 6, 10, and 13 rather than the total running cost resulting from the analysis.

Historical and current values for the Consumer Price Index and its components can be found in the monthly Department of Labor publication, Bureau of Labor Statistics News.[41] Similar data on the Producer Price Index (formerly the Wholesale Price Index) to be used for truck cost updating factors is found in another monthly Bureau of Labor Statistics publication Producer Prices and Price Indexes.[42]

TABLE 13

AVERAGE PASSENGER CAR FUEL CONSUMPTION (FC) FOR USE IN VEHICLE OPERATING COST UPDATING FACTOR (M)

Year	Average Passenger Car Fuel Consumption[a] kilometres/litre	(miles/gallon)
(Base)		
1975	5.8	(13.7)
(Projections)		
1979	6.2	(14.7)
1980	6.5	(15.2)
1981	6.6	(15.6)
1982	6.8	(16.0)
1983	7.0	(16.5)
1984	7.2	(17.0)
1985	7.4	(17.5)
1986	7.7	(18.0)
1987	7.8	(18.4)
1988	8.0	(18.9)
1989	8.2	(19.4)
1990[b]	8.5	(19.9)

Source: References 24 and 35.

a Based on projections of the passenger car fleet actually on the road in each given year.

b No reliable projections are available beyond 1990; it is recommended that the 1990 data be used for subsequent years.

Step 18 - Repeat Steps 4 through 17 for each interchange rehabilitation alternative and analysis year of interest.

2. Estimating level of service F conditions: Special problems in applying the recommended operational analysis procedure arise for both roadway sections and intersections under level of service F conditions. These occur when the traffic volume for a roadway section or intersection approach exceeds its capacity.

The AASHTO procedures for a roadway section are applicable when the traffic volume only slightly exceeds the capacity and no substantial queues of vehicles form. The nomographs in Figures 10 through 21 include procedures to determine the impact of level of service F conditions on delay and speed change costs for uninterrupted flow conditions. An example of these procedures is provided in Figure 11. If substantial queues of vehicles form, the queueing analysis procedures for uninterrupted flow presented in NCHRP Report 133[13] (pages 19-23) are applicable.

Because the AASHTO procedures for intersection delay and idling costs are based on Webster's model,[43] they are not applicable to Level of Service F conditions where the average delay per vehicle exceeds the signal cycle. However, the nomographs in Figures 22 and 23 can be used if the average delay per vehicle can be estimated by some other means (such as the procedure on pages 33-36 of NCHRP Report 133).

Queueing analyses based on NCHRP Report 133 can be very tedious. Therefore, the AASHTO Manual recommends that users should not perform a queueing analysis until both demand estimates and design parameters are carefully reviewed to determine whether queueing is actually a problem. In other words, the analyst should make sure that the estimated demand is realistic and design capacity cannot and should not be increased. For level of service F in the existing conditions, field studies presented in Appendix B (such as travel time delay and intersection delay studies) should be considered as an alternative.

B. Safety Effects

The prediction of the safety effectiveness of interchange rehabilitation projects is a difficult task because only a few completed interchange rehabilitation projects have been formally evaluated and because questions of interpretation frequently arise in studies of the incremental effects of geometric elements. All safety estimates require careful exercise of engineering judgment and the specific values used, such as the estimates of percent accident reduction, must be regarded as approximate. The purpose of this section is to illustrate some simple techniques for developing effectiveness estimates using both existing research results and data from traffic accident records systems.

1. General approach: Five general approaches to estimating safety effectiveness are recommended. There is no fixed order of preference for these approaches, although safety effectiveness estimates based on valid research or evaluation of similar projects are generally preferable to estimates based solely on judgment. The engineer should select the safety evaluation approach that appears most reliable given the nature of the improvement being considered and the available safety effectiveness estimates.

a. Estimate safety effectiveness based on the agency's own experience in similar projects: One useful source of safety estimates available to a highway agency are evaluations of similar projects constructed by the same agency, in the same geographic area and under similar operational conditions. Safety estimates based on local experience are generally preferred to estimates based on the experience of other agencies or research results. However, consistent data on the effectiveness of past projects are not readily available in many agencies, because the evaluation of completed projects is not done on a routine basis. Therefore, other approaches are often necessary.

b. Estimate safety effectiveness based on reported experience in similar projects: Another source of safety effectiveness estimates are project evaluations and research studies conducted by other agencies. Most commonly such results are reported as the percent accident reduction effectiveness.

Forty interchange rehabilitation projects are evaluated in Volume III of this report. The combined results of these evaluations can be applied to the estimation of safety effectiveness. On the average, major geometric modifications--large projects that rebuilt all or a major portion of an interchange--resulted in a 24 percent reduction in accident rate for the entire interchange. Minor ramp and/or crossroad modifications resulted in a 16 percent reduction in accident rate for the portion of the interchange directly affected by the improvement. Thus, major rehabilitation projects are not only more effective on a percentage basis than minor projects, but usually also influence a greater portion of the interchange.

Previous research has estimated percent accident reduction for a variety of other accident countermeasures. Most of this research did not specifically address the application of these countermeasures at interchange locations, but such results are potentially useful, especially at crossroad ramp terminals which are often similar to other urban intersections. Some of the estimates that may be appropriate to interchanges are presented in Table 14. These estimates are based primarily on studies by Roy Jorgenson and Associates[20,21] and the California Department of Transportation.[36] The estimates are presented in the form utilized in the original sources; some estimates apply to all accidents, some to specific severity levels and some to specific accident types. Many of these esti-

TABLE 14

SAFETY EFFECTIVENESS ESTIMATES

Improvement	Percent Accident Reduction Effectiveness By Type of Accident[b]											
	ALL	FI	PDO	HO	RE	RA	SS	LT	RT	FO	PD	NT
Install/improve directional/warning signs												
Two-lane crossroad	29	51										
Multilane crossroad	41	47										
Install all-way STOP sign [a]	68	67										
Improve signals												
Two-lane crossroad	31	35										
Multilane crossroad	-2	10										
Add left-turn lane without signal												
Two-lane crossroad	19	80										
Multilane crossroad	6	54	18									
Add left-turn lane and signal	27											
Add left-turn signal without turn lane	39	57										
Add left-turn lane, signal and illumination	46	76										
Install new traffic signals	29	50										
Improve pavement markings						10	20	10	10		10	10
Install actuated signals					10	10	20	10	20			
Improve sight distance				20		20		20	20			
Relocate fixed objects										60		
Install new safety lighting												75
Upgrade safety lighting												50
Install protective guardrail at embankment curves	50											
Improve superelevation	50											
Relocate driveways				20	20	10	10	10	10			

a Minor street (or ramp) must be 35 percent or more of total intersection volume which must be less than 8,000 ADT.

b Codes for accident types:

ALL	All accidents	SS	Sideswipe accidents
FI	Fatal and injury accidents	LT	Left-turn accidents
PDO	Property-damage-only accidents	RT	Right-turn accidents
HO	Head-on accidents	FO	Fixed-object accidents
RE	Rear-end accidents	PD	Pedestrian accidents
RA	Right-angle accidents	NT	Night accidents

mates are known to be quite imprecise; the Jorgenson study, for example, estimated the range of variation in some effectiveness estimates at 150 percent.

c. Estimate safety effectiveness based on engineering judgment: In some situations where effectiveness measures developed from previous research are not available, estimates can be developed through engineering judgment. For example, modification of ramp terminal geometrics and addition of signing to reduce wrong-way entries could reasonably be assumed to eliminate 70 to 100 percent of wrong-way accidents; capacity improvements to eliminate a queue of vehicle extending into the mainline freeway lanes from an off-ramp would be expected to eliminate 100 percent of the rear-end accidents on the freeway, although some rear-end accidents might still occur on the ramp. In each case, the assumed accident reduction applies only to the specific type(s) of accidents closely associated with the need for improvement. If the improvement is found to be unjustified economically, even with a 100 percent accident reduction, then the improvement can be rejected. If the improvement is found to be economically acceptable, but doubt remains about the effectiveness estimate, a sensitivity analysis to identify the economic acceptability over the full range of accident reduction effectiveness estimates (0 to 100 percent) is suggested.

d. Assume that the improvement will reduce the accident rate to the statewide average determined by the agency: Another possible estimate of effectiveness is to assume that the accident rate is reduced from its existing (presumably high) level to the statewide average for the interchange as a whole or some specific portion of the interchange. This would usually be a conservative assumption because a design built to current geometric standards would often be expected to operate at a below-average accident rate. Statewide average accident rates for specific interchange configurations and interchange elements can be derived from the agency's accident records system and are suggested for use in accident surveillance systems in Section IV.

e. Assume that the improvement will reduce the accident rate to an average rate determined from the literature: The final technique is similar to the previous approach, but the average accident rates for different interchanges and interchange elements are based on the literature rather than the agency's own experience. Although analysis of accident records is not required to derive these average rates from the literature such values are obviously less representative of local conditions. Appendix E presents some typical accident rates for interchange analyses selected from the literature. The primary sources of these data are the Interstate System Accident Research (ISAR) study[11], a recent reanalysis of the ISAR data[28] and a study by Lundy[23] in California. The data for all of these studies were collected at least 15 years ago and may not be representative of current conditions.

2. Measures of effectiveness: The objective of these procedures is to quantify the safety effectiveness of alternative improvements in a form useful in decisionmaking. The percent accident reduction effectiveness of a countermeasure can be used in combination with the existing accident rate at the interchange to determine the desired effectiveness measures: Number of fatal and injury accidents reduced per year, and number of property-damage-only accidents reduced per year. This determination should be made for traffic volume levels both at the completion date of the contract and in the design year (e.g., 20 years hence). These two estimates should be combined to determine the average accident reduction over the design life of each alternative improvement.

C. Air Pollution

The reduction of air pollution levels may be an important factor to be considered in the evaluation of interchange rehabilitation alternatives. It is important to recognize that there is no conflict between our national goal to improve the quality of the environment and the objectives of most interchange rehabilitation projects. Most traffic operational and safety improvements are intended to increase vehicle speeds, reduce volume-to-capacity ratios and reduce vehicle delay--all of which also tend to reduce pollutant levels. At worst, a safety improvement with minor operational impact would have no effect on air pollution; at best, operational improvements should reduce pollutant levels. These impacts, which may be an important advantage of many interchange rehabilitation projects, should be quantified to receive due consideration in the decision making process.

D. Noise

The prediction of traffic noise levels has become an increasingly important part of the design of highway projects. Noise levels should be considered in interchange rehabilitation projects because traffic speeds--and, consequently, noise levels--may be increased and because ramps and roadways may be relocated closer to existing development. On the other hand, noise levels can also be reduced through interchange rehabilitation by relocation of ramps and roadways further from existing development, by reduction of accelerations and decelerations of large trucks, or by incorporation of a noise barrier in the interchange design. There are no noise prediction models intended specifically for use at interchanges. One model that can be used is the Highway Traffic Noise Prediction Model recently developed by the Federal Highway Administration.[8]

VIII. EVALUATE ALTERNATIVES AND SELECT THE BEST

The greatest challenge in the interchange rehabilitation process is the selection of one alternative for implementation from among those evaluated. The designer needs to consider all factors that potentially affect the decision, assess the effects of each factor, consider the trade-offs between alternatives, and then choose one design. Three kinds of factors are considered in design decisions: those that can be quantified in monetary terms; those that can be quantified, but not in monetary terms; and those that cannot be quantified. Section VII focused on four effects that can be quantified: traffic operations, safety, noise, and air quality. A fifth factor that can, of course, be quantified is construction cost.

Decisions concerning minor interchange rehabilitation projects can often be based solely on the factors that directly concern highway engineers: traffic operations, safety, and cost. However, the larger a project becomes, the more important the role that social, economic, and environmental factors play in design decisions. Table 15 suggests the broad range of factors that could be considered in design decisions, not necessarily arranged in order of importance. While many of these factors are more typical of new construction projects than rehabilitation of existing facilities, each could potentially influence an interchange rehabilitation decision.

A. Tradeoff Evaluation

Alternative improvements for a particular interchange problem should be compared on the basis of their anticipated effects. The trade-offs between alternatives can be considered by a variety of formal and informal techniques including economic analyses, decision theory, and engineering judgment.

Most design decisions in highway agencies are currently based on the exercise of engineering judgment within the bounds set by accepted design policies. Practicing engineers regard this reliance on engineering judgment as inevitable because of the multiplicity of factors involved and the qualitative nature of many factors. However, engineering judgment should not be exercised in lieu of a formal decision making process without extensive documentation supporting the engineering judgment decision.

Bayesian decision theory provides one possible approach to formalizing the design decision making process. Rather than replacing the engineer's judgment, decision theory seeks to refine these judgments and express them as a set of probabilities that can be used to compare alternatives. The most

TABLE 15

FACTORS IN DESIGN DECISIONS

- Fast, safe and efficient transportation;
- National defense;
- Economic activity;
- Energy conservation;
- Employment
- Recreation and parks;
- Fire protection;
- Aesthetics;
- Public utilities;
- Public health and safety;
- Residential and neighborhood character and location;
- Religious institutions and practices;
- Conduct and financing of government;
- Conservation (including erosion, sedimentation, wildlife, and general ecology of the area);
- Natural and historic landmarks;
- Noise, air, and water pollution;
- Property values;
- Multiple use of space;
- Replacement housing;
- Education (including disruption of school district operations);
- Displacement of families and businesses;
- Engineering, right-of-way and construction costs of the project and related facilities;
- Maintenance and operating costs of the project and related facilities; and
- Operation and use of existing highway facilities and other transportation facilities during construction and after completion.

concise explanation of the application of Bayesian decision theory to highway design problems is in the Major Interchange Design study performed for FHWA by Penn State University. Decision theory, although a valid decision making technique, has not achieved widespread use or acceptance in the highway community and its use is not recommended.

Another approach for formalizing the design decisionmaking process is the use of economic analyses, where factors are quantified in monetary terms. With costs and measures of effectiveness expressed in dollars, the tradeoffs between the traffic operations, safety, and construction cost of alternative designs can be considered explicitly. Economic analyses have been used by highway agencies to justify improvements by demonstrating that the benefits of a proposed improvement exceed its cost. However, an economic analysis can never be the sole criterion for a design decision, because nonmonetary and nonquantifiable factors are not considered.

It is our assessment that the evaluation of tradeoffs between design alternatives must remain an essentially subjective process, relying in large measure on the judgment of experienced designers. However, it is important that the decision process be <u>as objective as possible</u> in considering those factors that can be quantified. Thus, the use of economic analyses as a major factor--though not the sole criterion--in decisions is recommended. Procedures for an economic analysis are presented in the next section.

Because of the important role of engineering judgment in making tradeoffs, it is vital that the anticipated effects be documented by the engineer in a formal report or memorandum as the basis on which judgments are made. This could take the form of a list of advantages and disadvantages of each alternative and the designer's assessment of the merits of each alternative relative to its construction cost and the budget constraints on the project. All of the evaluation factors in Table 15 that are affected by any of the improvements should be listed in the assessment of advantages and disadvantages. A formal report of these assessments can be used to document the reasons for the designer's decision to highway agency management, to FHWA and to the public.

B. Economic Analysis

This section presents a simple method for economic analysis of the operational and safety effects of alternative interchange designs relative to their construction costs. The analysis considers only factors that are quantifiable in monetary terms; nonquantifiable and nonmonetary factors must be weighed on the basis of engineering judgment. Several economic analysis examples are presented in Appendix E.

The economic analysis is based on the operational and safety effects that were quantified in Section VII. Each effect derived there was expressed in its natural units: travel time in hours; vehicle operating costs in dollars; and safety in number of accidents. These effects must all be expressed in dollars for the economic analysis.

The recommended criterion for ranking alternative improvements at a given interchange is the net return, defined for each alternative as:

$$\begin{aligned}\text{Net Return} = {} & (TC_B - TC_A) + (RC_B - RC_A) \\ & + (AC_B - AC_A) + (OC_B - OC_A) \\ & - (CC)\ (CRF,\ i\%,\ n)\end{aligned}$$

where TC = Cost of travel time (delay);
RC = Vehicle operating cost;
AC = Accident cost;
OC = Other cost;
CC = Construction cost; and
(CRF, i%, n) = Capital recovery factor for n years at i percent interest;
subscript B refers to the existing condition ("do-nothing" alternative); and, subscript A refers to the improved condition for the alternative under consideration.

All cost elements in the equation for net return are expressed in dollars and each element is discussed later in this section.

The alternative designs can be ranked in order of preference on the basis of their net returns. Any alternative with net return greater than zero is preferable to the existing condition. When arranged in order of descending net return, the alternative designs are ranked in priority order for implementation. The alternative with the highest net return is the most preferable, subject to the constraint that its construction cost does not exceed the available budget. If no alternative has a net return greater than zero, then no improvement to the existing condition is justified on the basis of operational and safety effects alone.

An economic analysis using the net return as a decision criterion has been recommended in preference to the use of a benefit-cost ratio. This ratio is similar to the net return except that the construction cost, rather than being subtracted from the other costs, is placed in the denominator of the ratio. When benefit-cost ratios are used as the basis for an economic analysis, the alternatives must be compared on the basis of an incremental analysis, which requires that the additional construction cost for each progressively more expensive alternative must be justified by the additional savings in operational and accident costs for that particular alternative. The net return method is recommended because it is computationally simpler than the benefit-cost-method, while the ranking of alternatives produced by the two methods are identical.

1. Travel time (delay) cost: Procedures to quantify the total travel time during a typical day in the interchange study area for each alternative and for the existing condition were presented in Section VII-A. The difference in travel time between the existing condition and a given alternative is the daily delay reduction (in vehicle-hours), for that alternative. This value should be multiplied by 365 to obtain the delay reduction on an annual basis. (For interchanges where peak hour congestion predominates, it may be more appropriate to multiply by 250--the approximate number of working days per year.)

For use in an economic analysis, the delay reduction must be expressed as a monetary amount. A study of the value of travel time to motorists was conducted by Thomas and Thompson,[39] who found the travel time value to be sensitive to trip purpose, traveler's income levels and the amount of time savings per trip. In general, travel time values were found to be very low for time savings of less than five minutes on any trip. Although only a few interchange rehabilitation projects could conceivably result in an average delay reduction of more than five minutes per vehicle, many projects are a part of a long-term systematic program of improvements that cumulatively save much time. On the basis of the Thomas and Thompson results, the authors of the AASHTO user benefit analysis manual judged the value of travel time to be \$3.00 per vehicle-hour, which was derived from a value of \$2.40 per person-hour and an average occupancy of 1.25 persons per vehicle. The value of \$3.00 per vehicle-hour, like all other cost data derived from the AASHTO manual, are appropriate for January 1975. An appropriate updating factor, based on the Consumer Price Index, as discussed in Section VII-A, should be used to project this value to the actual analysis year.

The term (TC_B - TC_A) in the net return expression is the product of the delay reduction in vehicle-hours and the value of travel time, appropriately updated.

2. Vehicle operating costs: Daily vehicle operating costs (or running costs) for the existing condition and each alternative were derived explicitly in Section VII-A. These costs should also be converted to annual costs through multiplication by 365 (or 250). For a particular alternative, the term (RC_B - RC_A) in the net return expression is the difference between the operating costs for the existing condition and for the alternative.

3. Accident costs: In Section VII-B, the accident reduction effectiveness of each alternative were derived in terms of:

- Number of fatal and injury accidents reduced per year; and
- Number of property-damage-only accidents reduced per year.

These accident reductions can be converted to monetary amounts based on the cost of accidents to the involved individuals and to society.

The costs of motor-vehicle accident involvements have been estimated by the National Highway Traffic Safety Administration as:[29]

Fatality	$287,175
Injury	8,085*
Property-Damage-Only Involvement	520

These costs cannot be used directly in the economic analysis because they represent the cost of accident involvements rather than accidents. For example, there is an average of 1.71 vehicles involved in a property-damage only accident, an average of 1.50 injuries per injury accident and 1.17 fatalities and 2.03 injuries per fatal accident. The averages can be used to compute the cost of accidents from the cost of involvements.[2] The cost of property-damage-only accidents should also be increased by 90% to account for unreported accidents. The resulting accident costs are:

Fatal Accident	$352,400
Injury Accident	12,100
Property-Damage-Only Accident	1,700

These accident costs based on the NHTSA study reflect 1975 cost levels and should be updated to present levels based on the Consumer Price Index (or its automobile insurance component), as discussed in Section VII-A.

The term $(AC_B - AC_A)$ in the net return expression is the sum of the values for fatal, injury and property-damage-only accidents reduced.

4. Other costs: The term $(OC_B - OC_A)$ in the net return expression gives the analyst an opportunity to include in the analysis any other cost items that affect a particular decision and can be quantified. Other costs that either increase or decrease the attractiveness of a par-

* based on Abbreviated Injury Scale (AIS) Severity Level 3 (severe, not-life-threatening injury)

ticular alternative can be considered. For example, if an improvement alternative is anticipated to increase or decrease the annual maintenance cost for the interchange, this additional expense or savings should be incorporated in the analysis.

5. Construction cost: The final term in the net return expression represents the construction cost of the interchange rehabilitation. The construction cost (in dollars) for each improvement alternative should be estimated by the designer.

The travel time costs, vehicle operating costs, accident costs, and other costs in the analysis are all expressed in dollars on an annual basis throughout the service life of the improvement. By contrast, construction costs are incurred on a one-time basis at the beginning of the improvement service life. The Capital Recovery Factor (CRF (i,n)) is used to convert the one-time construction cost to a series of equivalent uniform annual costs. This conversion requires the analyst to estimate the service life of the improvement (n) and the minimum attractive rate of return (i).

Service lives of the major capital items in interchange rehabilitation projects are estimated as:

Right-of-way	50-100 years
Pavement, Earthwork, and Structures	20-40 years
Lighting	15-20 years
Signals	10-15 years
Guardrail	10-15 years
Signing (major)	8-10 years
Signing (minor)	3-5 years
Pavement Markings	1-5 years

A range of service lives is suggested for each capital item to allow for variations in highway agency policies and experience. The engineer should select the service life for the project based on the predominant capital items being used. Major reconstruction projects should have a 20-year life, while a minor project involving essentially signal improvements would have a 10 or 15 year life, etc. Right-of-way has a much longer useful life than other capital items and may deserve special treatment in projects that involve major right-of-way acquisition. For example, it may be desirable to assign a salvage value to the right-of-way at the end of the project life.

It is generally accepted that public investments in highway improvements should be justified on the basis of their benefits to the public. The minimum attractive rate of return used in the Capital Recovery Factor assures that improvements must have at least a specified percentage return to be considered economically acceptable. The recommended range of values for the minimum attractive rate of return is 4 to 10 percent to be selected on the basis of highway agency policies. The appropriate Capital Recovery Factors at 5, 8 and 10 percent interest for each service life are given in Table 16. Capital Recovery Factors for any other interest rate will be found tabulated in economic analysis texts, such as Winfrey.[45]

TABLE 16

CAPITAL RECOVERY FACTOR AT 5, 8 AND 10 PERCENT INTEREST

Service Life of Improvement (years)	Capital Recovery Factor at 5%	at 8%	at 10%
1	1.050	1.080	1.100
2	0.538	0.561	0.576
3	0.367	0.388	0.402
4	0.282	0.302	0.315
5	0.231	0.250	0.264
6	0.197	0.216	0.230
7	0.173	0.192	0.205
8	0.155	0.174	0.187
9	0.141	0.160	0.174
10	0.129	0.149	0.163
11	0.120	0.140	0.154
12	0.113	0.133	0.147
13	0.106	0.127	0.141
14	0.101	0.121	0.136
15	0.096	0.117	0.131
16	0.092	0.113	0.128
17	0.089	0.110	0.125
18	0.086	0.107	0.122
19	0.083	0.104	0.120
20	0.080	0.102	0.117
30	0.065	0.089	0.106
40	0.058	0.084	0.102
50	0.054	0.082	0.101
100	0.050	0.080	0.100

IX. IMPLEMENT AND EVALUATE IMPROVEMENT PROJECT

The remaining stages in the interchange rehabilitation process are to implement the selected improvement project(s) and, subsequently, to evaluate the improvement effectiveness.

Implementation of the project includes detailed design of the selected alternative; preparation of construction plans; approval by highway agency management and (where appropriate) by FHWA and local authorities; and construction of the improvement. These activities are a vital part of the interchange rehabilitation process, but are outside the scope of this report.

The evaluation of each project after its implementation provides formal measures of the effectiveness of each project and the entire interchange rehabilitation program. Feedback from the evaluation of projects is needed to increase the ability to estimate the effectiveness of future projects, particularly in the area of safety effectiveness. The remainder of this section discusses effectiveness evaluations.

A. General Approach to Effectiveness Evaluations

An effectiveness evaluation should be conducted for each interchange rehabilitation project. Effectiveness evaluations could address any of the impacts investigated in Section VII, but traffic operations and safety are generally emphasized because these are the motivating factors behind most interchange rehabilitation projects. However, when other factors play an important role in the decision between alternatives, the project's impact on those factors can be evaluated as well.

The objective of an effectiveness evaluation is to compare the actual and predicted effects to establish whether (and to what extent) the project has achieved what was intended. It is important that both "successes" and "failures" be documented, so that future "failures" can be avoided and the available funds can be optimally invested in "successful" projects.

A two-stage approach to effectiveness evaluation is recommended. The first evaluation should be made within several months after the improvement is opened to traffic. The primary purpose of this evaluation is to detect any operational or safety problems that remain or have developed since completion of the project. A complete operational evaluation can be performed at this time, but a longer period is usually required to draw conclusions about the project's safety effectiveness with a high level of statistical confidence. Consequently, a second evaluation of each project is recommended when three years of accident data are available for the period after the project. This final evaluation should establish a measure of safety effectiveness (e.g., percent accident reduction) for the project.

B. Operational Evaluations

Operational evaluations should make use of a combination of three techniques:

- Field observation of traffic flow;
- Level of Service evaluation; and
- Delay measurement.

Field observation of traffic flow should involve the same procedures for on-site observation discussed in Section V. During a field visit to the interchange site the engineer should attempt to determine whether the problems that prompted the improvement project have been alleviated and whether any new operational problems are apparent. Queues of vehicles and other apparent problems should be noted.

A capacity analysis for roadway sections and/or intersection approaches should be performed for comparisons with the condition before the improvement project. Since a capacity analysis is usually performed during the design of an interchange improvement, this evaluation may require simply the collection of traffic volume data (ADT and peak hour volumes) and comparison of actual and projected volumes.

Delay studies can be used to quantify the reduction in delay from the before to the after period. The route delay and intersection delay methods discussed in Appendix B are appropriate for this purpose. As an alternative, the input data required for the AASHTO operational analysis procedure (presented in Section VII) should be collected and the analysis performed for the actual field conditions.

Any other engineering studies that were used to diagnose a problem in the before condition can be repeated, as appropriate, to establish an effect of the project.

C. Safety Evaluations

The recommended safety evaluation procedure is a Chi-Square comparison of the accident experience before and after the project. This comparison, based on the percent reduction in accident rate, should utilize accident data for the portion of the interchange directly affected by the project. It is desirable to use at least three years of accident data in both the before and after periods, although the test can be applied with less accuracy to shorter time periods.

The safety effectiveness evaluation is usually applied to the total accident experience at the location being studied. However, depending on the countermeasure being evaluated, the user may choose some other measure of effectiveness, such as the fatality and injury rate or the rate for a specific type of accident.

The accident rate for the period before the project is calculated as:

$$AR_b = \frac{(N_b)(10^6)}{(L)(ADT)(D)}$$

for a roadway section, and

$$AR_b = \frac{(N_b)(10^6)}{(ADT)(D)}$$

for a point location (including a ramp or an intersection), where

AR_b = Accident rate for before period (accidents per million vehicle-kilometres or vehicle-miles);
N_b = Number of accidents in before period;
L = Length of roadway section (kilometres or miles);
ADT = Average Daily Traffic for roadway section, ramp, or intersection (vehicles/day); and
D = Length of study period (days) (e.g., 3 years = 3 (365) = 1,095 days).

The ADT of an intersection is the sum of the daily entering volumes, while the ADT for a ramp or roadway section is the traffic volume traversing the section. The accident rate for the after period, (AR_a), is computed in a similar manner.

The percent reduction in accident rate is computed as:

$$\text{Percent Reduction} = \frac{AR_b - AR_a}{AR_b} \times 100$$

The Chi-Square test is performed by locating the point in Figure 9 defined by the actual Percent Accident Reduction observed for the project and the number of accidents before the project (N_b). If this point lies above the line in Figure 9, then the reduction in accident rate is statistically significant and it is presumed that the interchange improvement project is responsible for the reduction. If this point lies below the line, then the reduction in accident rate is not statistically significant. However, this does not necessarily mean that the project was not effective--just that it cannot be proven to be effective at some specified level of confidence (in this case, 95 percent). Although the safety evaluation procedure has been described as applicable to the percent reduction in accident rate, it is equally appropriate to determine whether a percent increase in accident rate is statistically significant.

As an example of the use of Figure 9, consider a site where 10 accidents occurred in the year before a project was constructed that reduced 60% of these accidents. The point in Figure 9 defined by 10 accidents before the project and a 60% accident reduction is located in the shaded area below the line, indicating that the observed accident reduction was not statistically significant and could have resulted from chance alone. On the other hand, if a project at another site with 30 accidents experienced a 60% reduction, the corresponding point in Figure 9 lies above the line; this accident reduction is said to be statistically significant and is presumed to result from the improvement project.

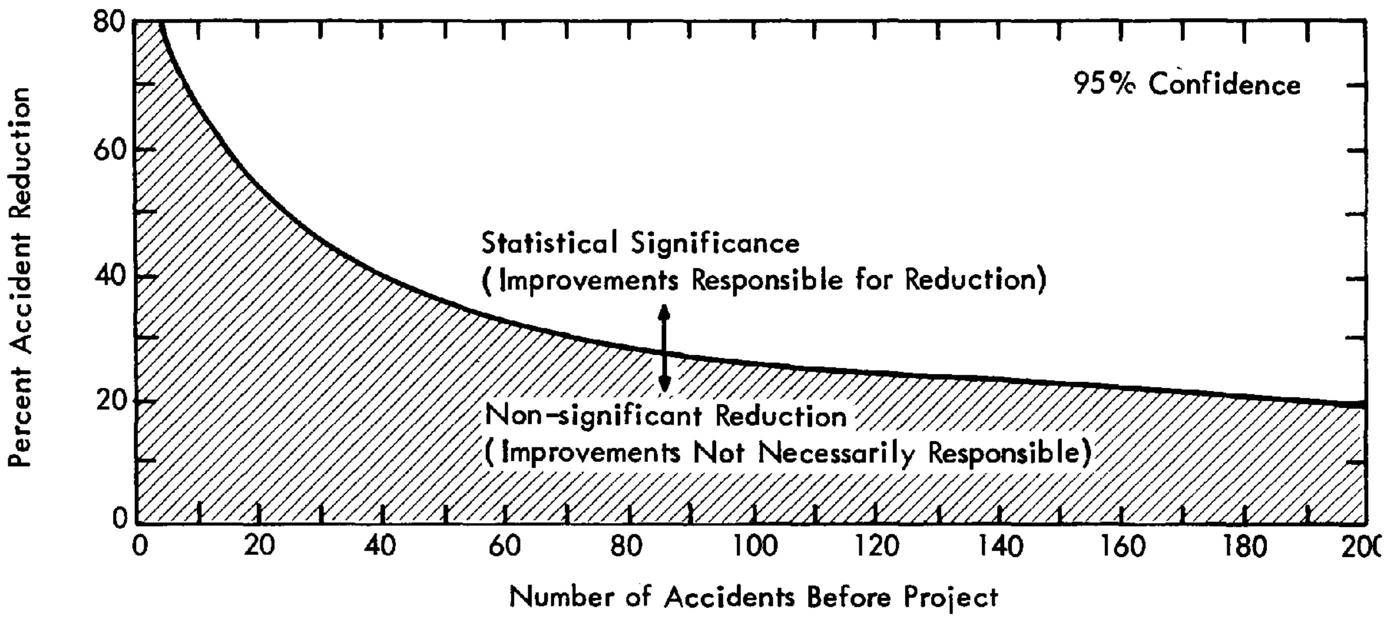


Figure 9 - Chi-Square Relationship to Test the Accident Reduction Effectiveness of Safety Improvement Projects.[27]

The statistical test in Figure 9 is based on the assumption that the accident data for the before and after periods have a Poisson distribution. A more general statistical procedure, that is less dependent on the assumption of a Poisson distribution, is presented in Appendix C. This procedure--the two sample t-test--has been placed in an Appendix because it is computationally more complex than the Chi-Square test. However, with the step-by-step approach used in Appendix C, no specialized statistical background is required to apply the test.

D. Evaluation Report

It is recommended that a short evaluation report or memorandum be prepared for each interchange rehabilitation project. The report should briefly describe the objectives of the project along with the anticipated operational benefits (reduction in delay or an increase in Level of Service for particular portions of the interchange). The safety benefits should be presented as the percent accident reduction for all accidents or for some specific accident type and the report should identify whether or not the accident reduction is statistically significant. It may be convenient to compare the operational and safety benefits relative to the project construction cost by means of a net return or benefit-cost ratio (see Section VIII) although, when this is done, nonquantitative benefits should be described as well.

REFERENCES

1. American Association of State Highway Officials, "Highway Design and Operational Practices Related to Highway Safety," Washington, D.C., 1967.

2. American Association of State Highway and Transportation Officials, "A Manual on User Benefit Analysis of Highway and Bus-Transit Improvements-1977," Washington, D.C., 1978.

3. American Association of State Highway Officials, A Policy on Geometric Design of Rural Highways-1965, Washington, D.C., 1966.

4. American Association of State Highway and Transportation Officials, A Policy on Design of Urban Highways and Arterial Streets-1973, Washington, D.C., 1973.

5. American Society for Testing and Materials, Recommended Practice E-274-77, Part 15, ASTM Annual Book of Standards, 1978.

6. Anscombe, F., Biometrika 35:246-54, 1948, cited in Freyer, H.G., Concepts and Methods of Experimental Statistics, Allyn and Bacon, Boston (1966).

7. Baker, Michael, Jr., Inc., Contract No. DOT-FH-11-9183, Federal Highway Administration.

8. Barry, T. M. and J. A. Reagan, "FHWA Highway Traffic Noise Prediction Model," Report No. FHWA-RD-77-108, Federal Highway Administration, December, 1978.

9. California Department of Transportation, Manual of Traffic Accident Surveillance and Analysis System, "Section 100 -- System and Data Base Description," 1973.

10. California Department of Transportation, Manual of Traffic Accident Surveillance and Analysis System, "TASAS Accident Location Coding, " 1978.

11. Cirillo, J. A., S. K. Dietz and R. C. Beatty, "Analysis and Modeling of Relationships Between Accidents and the Geometric and Traffic Characteristics of the Interstate System," Federal Highway Administration, August 1969.

12. Courage, Kenneth G. and Paraskevi Papapanou, "Estimation of Delay at Traffic-Actuated Signals," Transportation Research Record 630, Transportation Research Board, 1977.

13. Curry, David A. and Dudley G. Anderson, "Procedures for Estimating Highway User Costs, Air Pollution and Noise Effects," NCHRP Report 133, Highway Research Board, 1972.

14. Freyer, H.G., Concepts and Methods of Experimental Statistics, Allyn and Bacon, Boston, 1966.

15. Garrett, John W. and Kenneth J. Tharp, "Development of Improved Methods for Reduction of Traffic Accidents," NCHRP Report 79, Transportation Research Board, 1969.

16. Glauz, W. D. and D. J. Migletz, "Application of Traffic Conflict Analysis at Intersections," NCHRP Report 219, Transportation Research Board, February 1980.

17. Highway Research Board, Highway Capacity Manual, Special Report 87, 1965.

18. Institute of Traffic Engineers, Manual of Traffic Engineering Studies, Fourth Ed., Arlington, Virginia, 1976.

19. Institute of Traffic Engineers, Transportation and Traffic Engineering Handbook, John E. Baerwald, Ed., Prentice-Hall, Inc., Englewood Cliffs, N.J., 1976.

20. Jorgenson, Roy and Associates and Westat Research Analysis, Inc., "Evaluation of Criteria for Safety Improvements on the Highway," U.S. Department of Commerce, Bureau of Public Roads, 1966.

21. Jorgenson, Roy and Associates, "Methods for Evaluating Highway Safety Improvements," NCHRP Report 162, Transportation Research Board, 1975.

22. Leisch, J. E., "Capacity Analysis Techniques for Design and Operation of Freeway Facilities," Report No. FHWA-RD-74-24, Federal Highway Administration, February 1974.

23. Lundy, Richard A., "The Effect of Ramp Type and Geometry on Accidents," California Division of Highways, May 1965.

24. Mannering, Fred L. and Kumares C. Sinha, "Forecasting Automobile Fuel Efficiency," Traffic Quarterly, Vol. 33, No. 4, October 1979.

25. McGee, H. W., et al., "Decision Sight Distance for Highway Design and Traffic Control Requirements," Report No. FHWA-RD-78-78, Federal Highway Administration, February 1978.

26. McInerney, H. E. and S. G. Peterson, "Intersection Capacity Measurement through Critical Movement Summations: A Planning Tool," Traffic Engineering, January 1971.

27. Michaels, Richard M., "Two Simple Techniques for Determining the Significance of Accident-Reducing Measures," Public Roads, Vol. 34, No. 5, February 1964.

28. Morganstein, D. R. and J. Edmonds, "Analysis of Interstate System Accident Research (ISAR) Data," Volume 1, Federal Highway Administration, November 1978.

29. National Highway Traffic Safety Administration, "1975 Societal Costs of Motor Vehicle Accidents," December 1976.

30. Opland, William H., "Interchange Priority Study -- Phase I," Michigan Department of State Highways and Transportation, May 1977.

31. Opland, William H., "Interchange Priority Study -- Phase II," Michigan Department of State Highways and Transportation, December 1977.

32. Reilly, W. R., C. C. Gardner and J. H. Kell, "A Technique for Measurement of Delay at Intersections," Volume 1, Technical Report, Report No. FHWA-RD-76-135, Federal Highway Administration, September 1976.

33. Reilly, W. R., C. C. Gardner and J. H. Kell, "A Technique for Measurement of Delay at Intersections," Volume 3, User's Manual, Report No. FHWA-RD-76-137, Federal Highway Administration, September 1976.

34. St. John, A. D., R. R. Blackburn and D. W. Harwood, "Effectiveness of Alternative Skid Reduction Measures, Volume II: Benefit-Cost Model," Federal Highway Administration, Report No. FHWA-RD-79-23, November 1978.

35. Shonka, D. B., A. S. Loebl and P. D. Patterson, "Transportation Energy Conservation Data Book: Edition 2," Oak Ridge National Laboratory, Report No. ORNL-5320, October 1977.

36. Tamburri, Thomas N. and Richard N. Smith, "The Safety Index: A Method of Evaluating and Rating Safety Benefits," Highway Research Record 332, Highway Research Board, 1970.

37. Taylor, J. I. and H. W. McGee, "Improving Traffic Operations and Safety at Exit Gore Areas," NCHRP Report 145, Highway Research Board, 1973.

38. Taylor, J. I., et al., "Major Interchange Design, Operation and Control," Reports No. FHWA-RD-73-80, -81 and -82, Federal Highway Administration, July 1973.

39. Thomas, Thomas C. and Gordon I. Thompson, "The Value of Time Saved by Trip Purpose," Contract No. DOT-FH-11-6881, Federal Highway Administration, 1970.

40. Transportation Research Board, "Interim Materials on Highway Capacity," Transportation Research Circular, Number 212, January 1980.

41. U. S. Department of Labor, Bureau of Labor Statistics, News, published monthly.

42. U. S. Department of Labor, Bureau of Labor Statistics, Producer Prices and Price Index, published monthly.

43. U. S. Department of Transportation, Federal Highway Administration, Manual on Uniform Traffic Control Devices for Streets and Highways, 1978.

44. Webster, F. V., "Traffic Signal Settings," Technical Paper 39, Transport and Road Research Laboratory, Crowthorne, England, 1958.

45. Winfrey, Robley, Economic Analysis for Highways, International Textbook Company, Scranton, Pennsylvania, 1969.

APPENDIX A

NOMOGRAPHS FOR OPERATIONAL ANALYSIS

Source: A Manual on User Benefit Analysis for Highway and Bus-Transit Improvements - 1977, Copyright, 1978, by the American Association of State Highway and Transportation Officials.

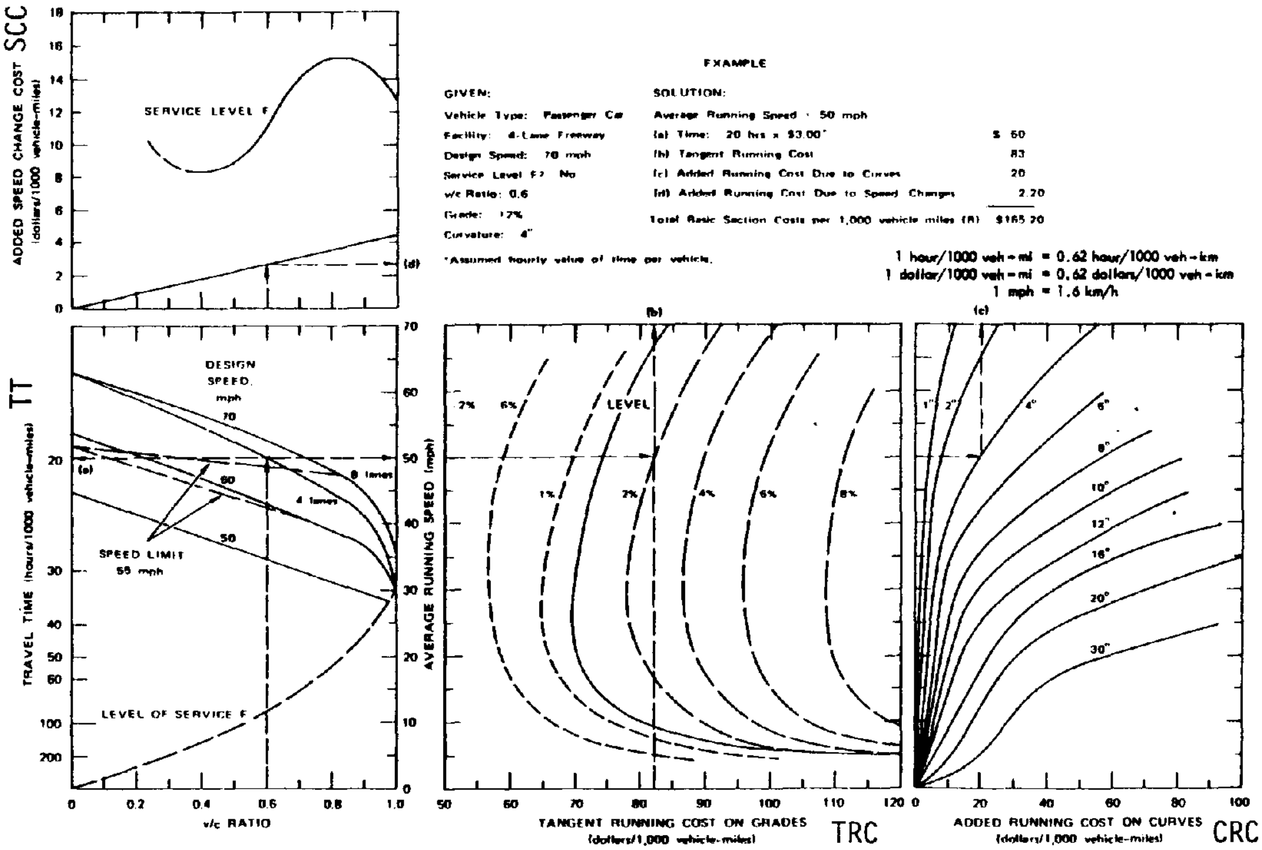


Figure 10 - Basic Section Costs for Passenger Cars on Freeways.

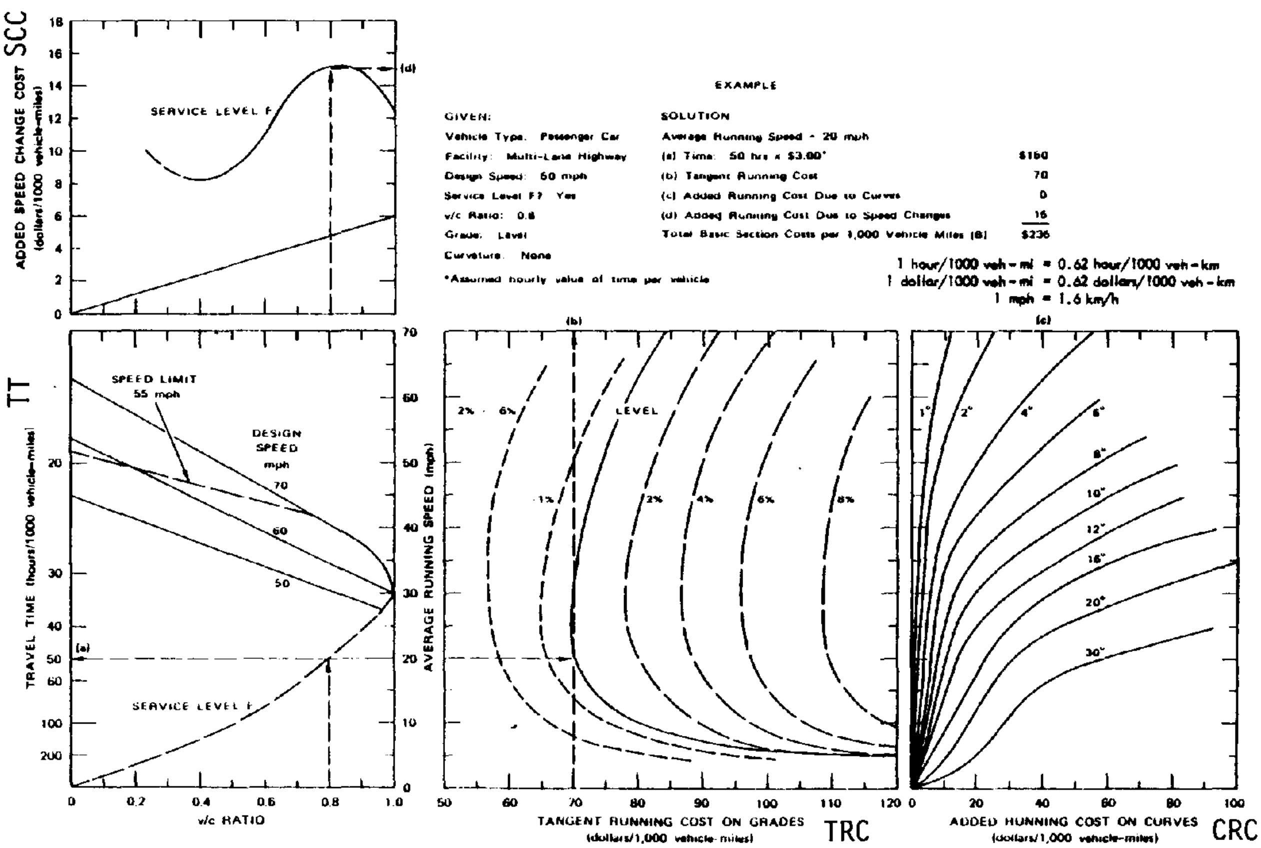


Figure 11 - Basic Section Costs for Passenger Cars on Multi-Lane Highways.

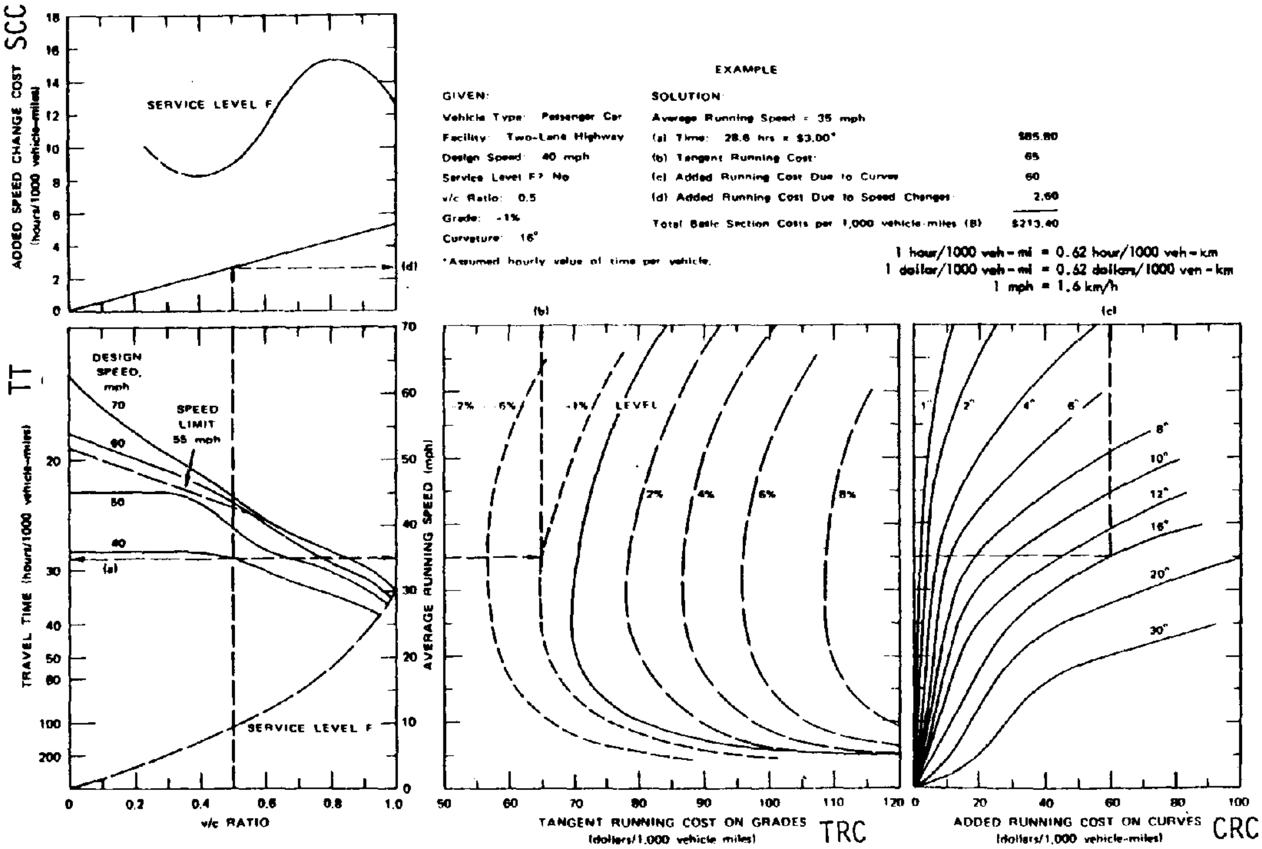


Figure 12 - Basic Section Costs for Passenger Cars on Two-Lane Highways.

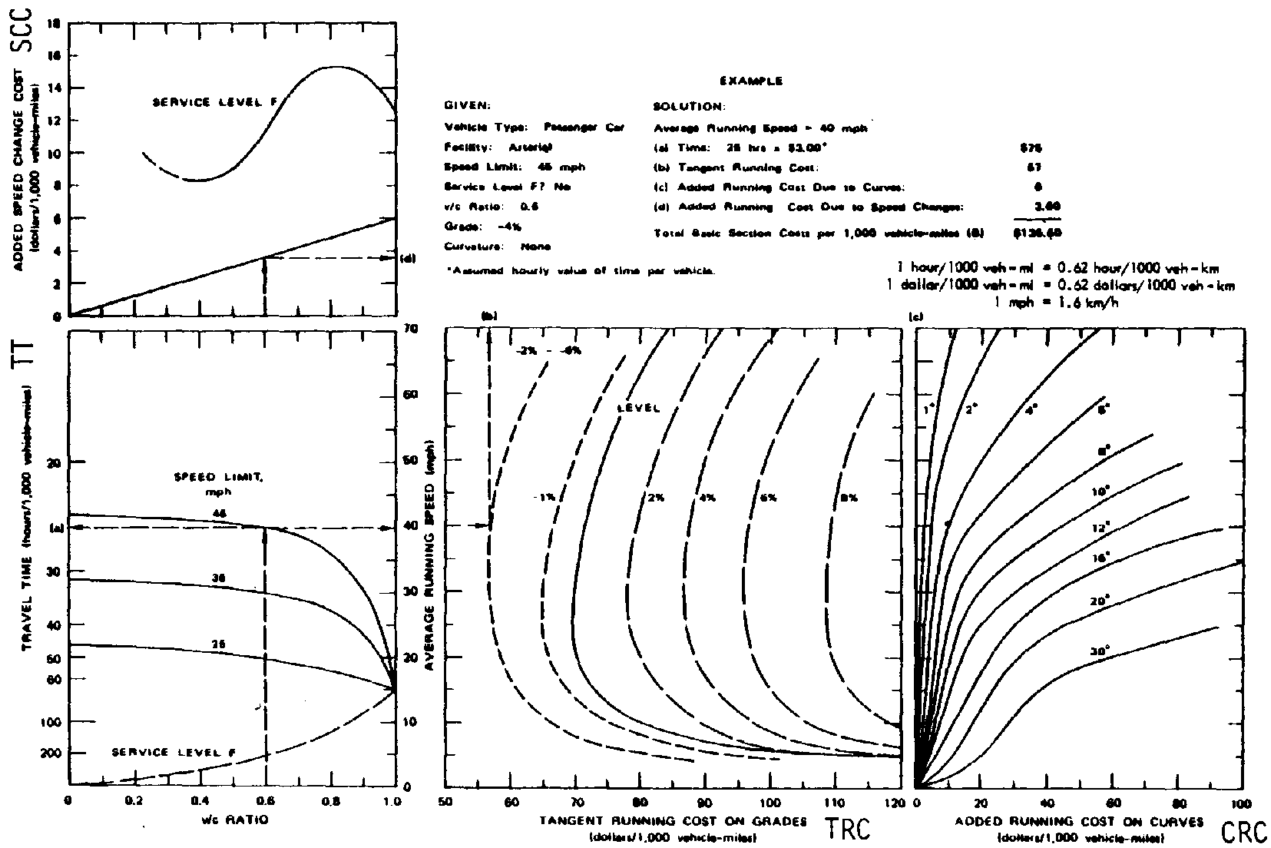


Figure 13 - Basic Section Costs for Passenger Cars on Arterials.

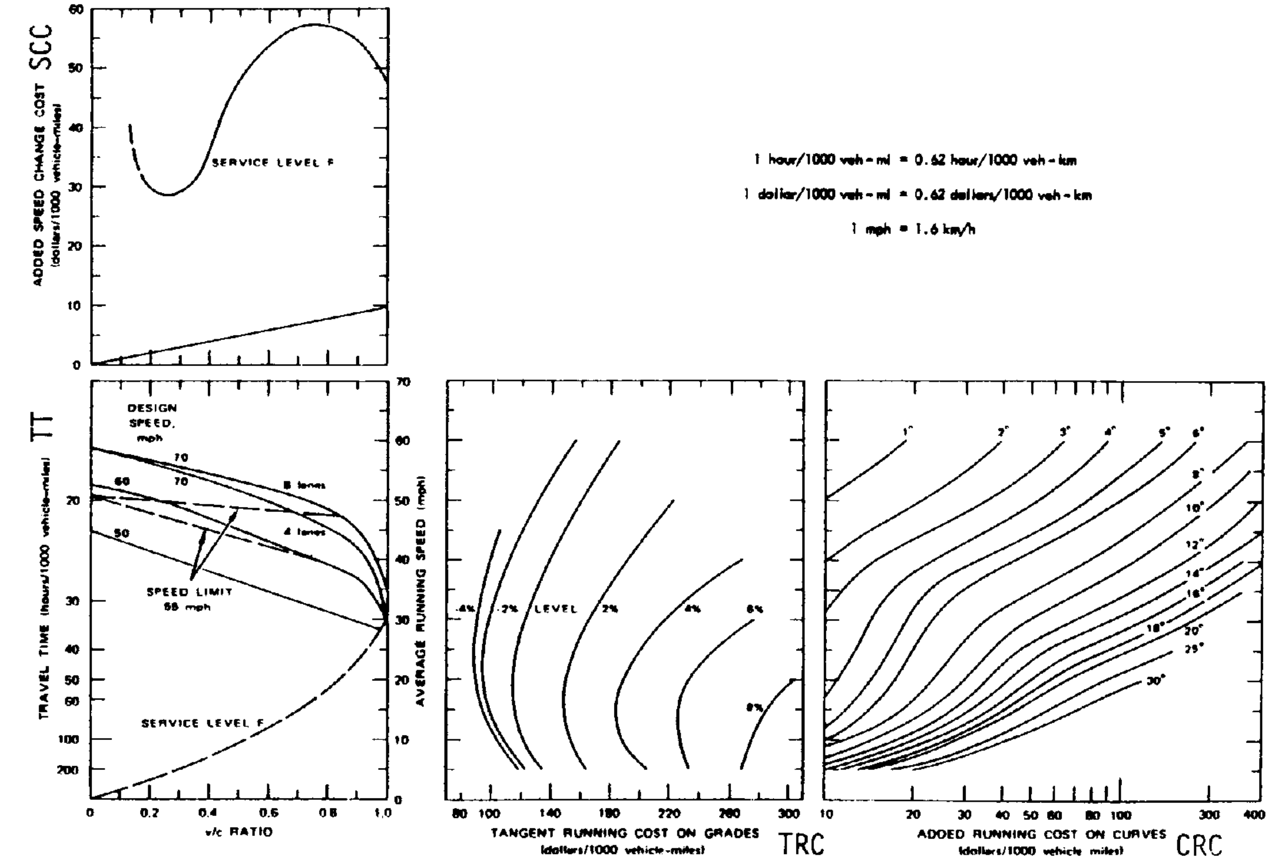


Figure 14 - Basic Section Costs for Single Unit Trucks on Freeways

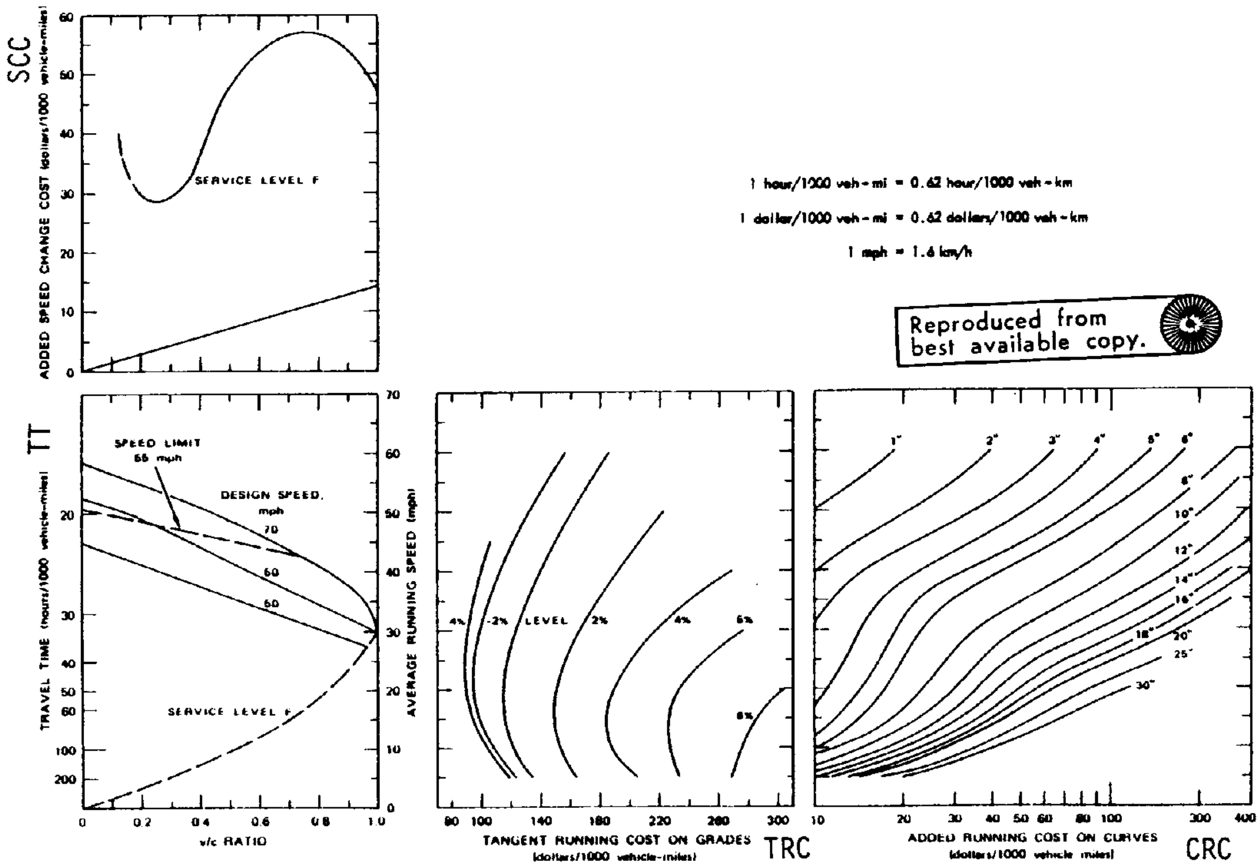


Figure 15 - Basic Section Costs for Single Unit Trucks on Multi-Lane Highways.

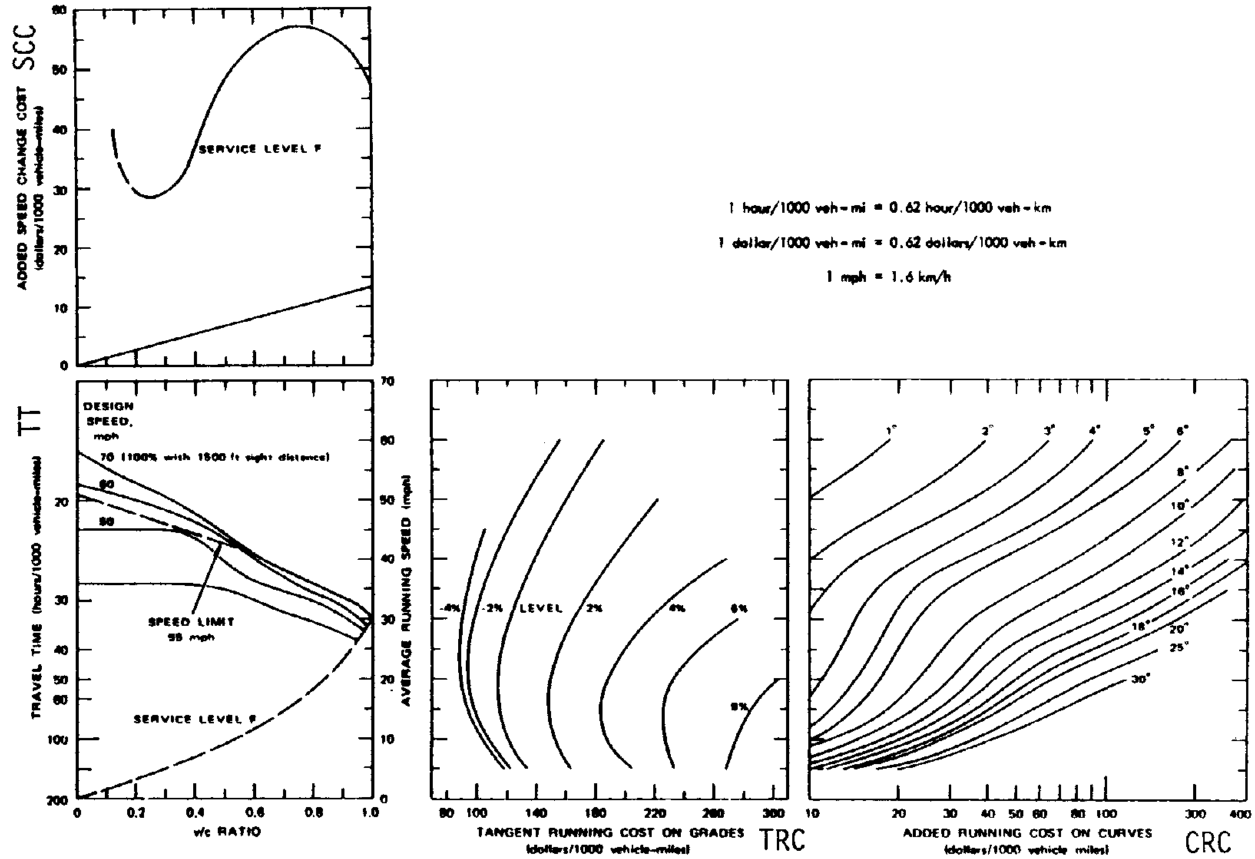


Figure 16 - Basic Section Costs for Single Unit Trucks on Two-Lane Highways.

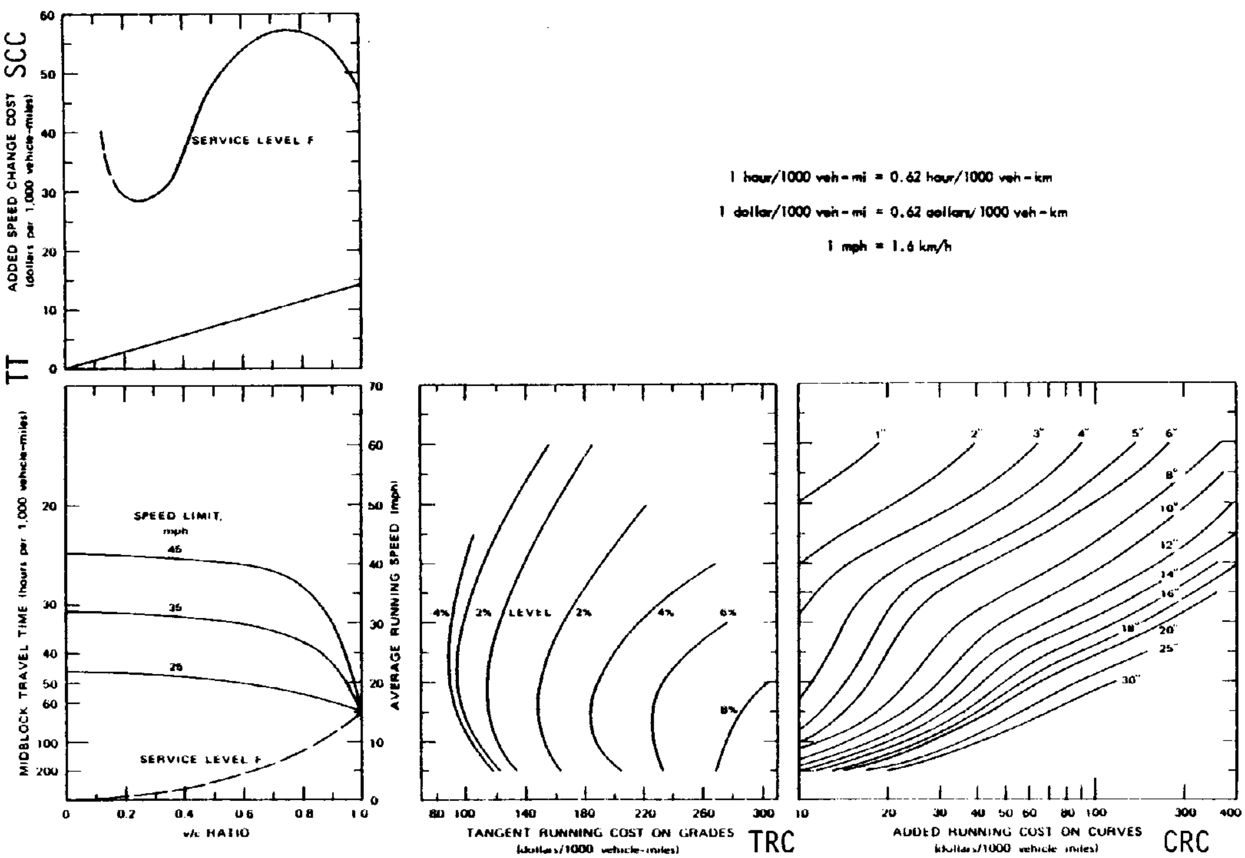


Figure 17 - Basic Section Costs for Single Unit Trucks on Arterials.

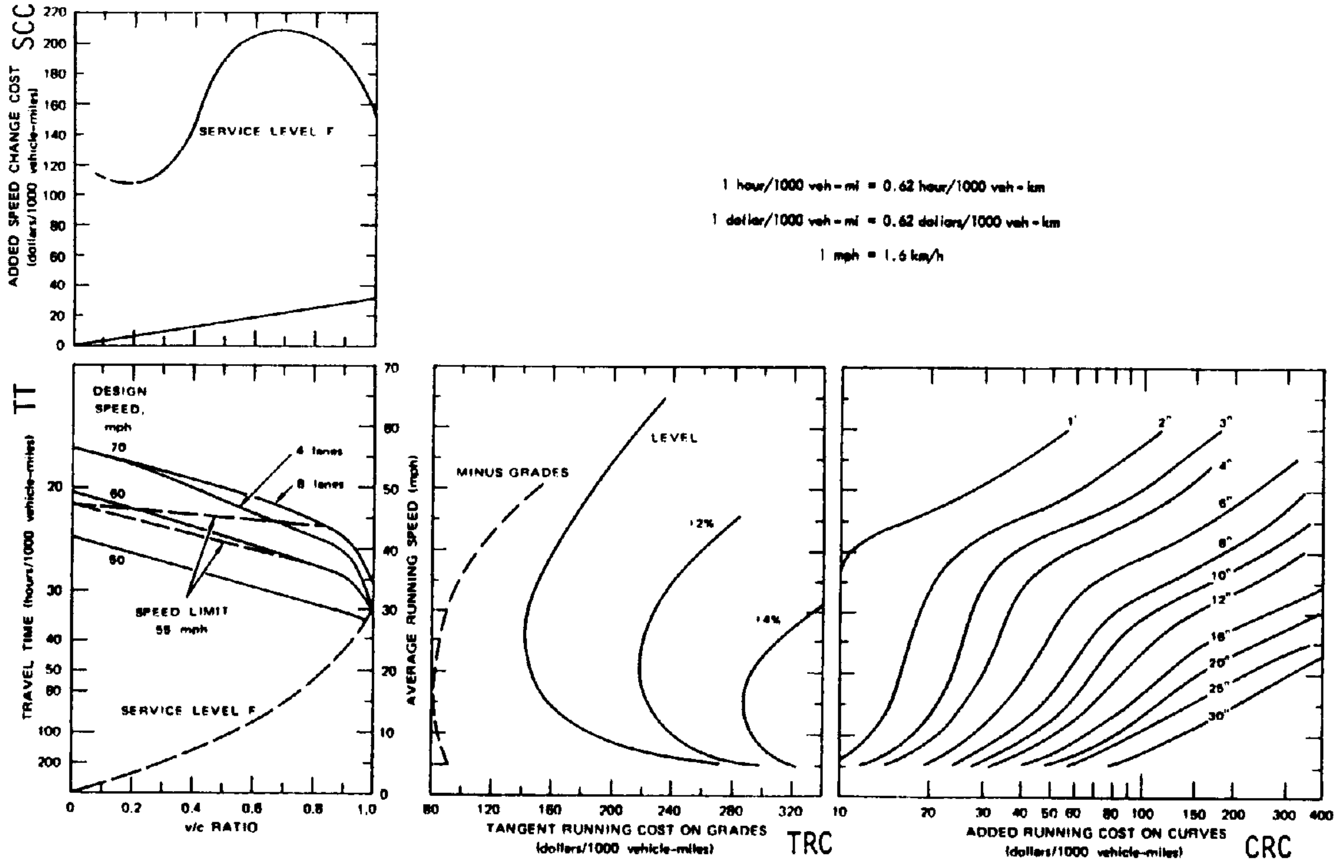


Figure 18 - Basic Section Costs for Combination Trucks on Freeways.

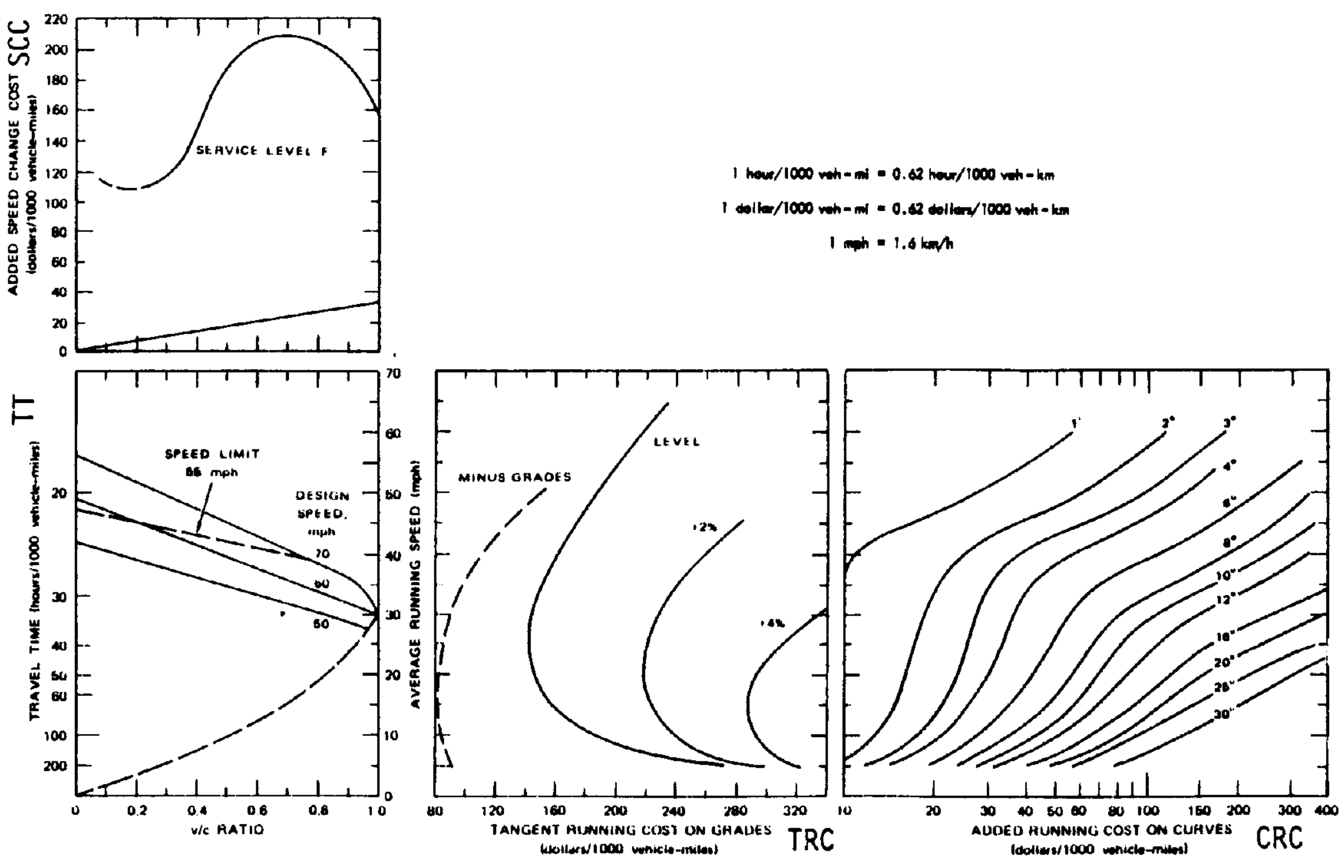


Figure 19 - Basic Section Costs for Combination Trucks on Multi-Lane Highways.

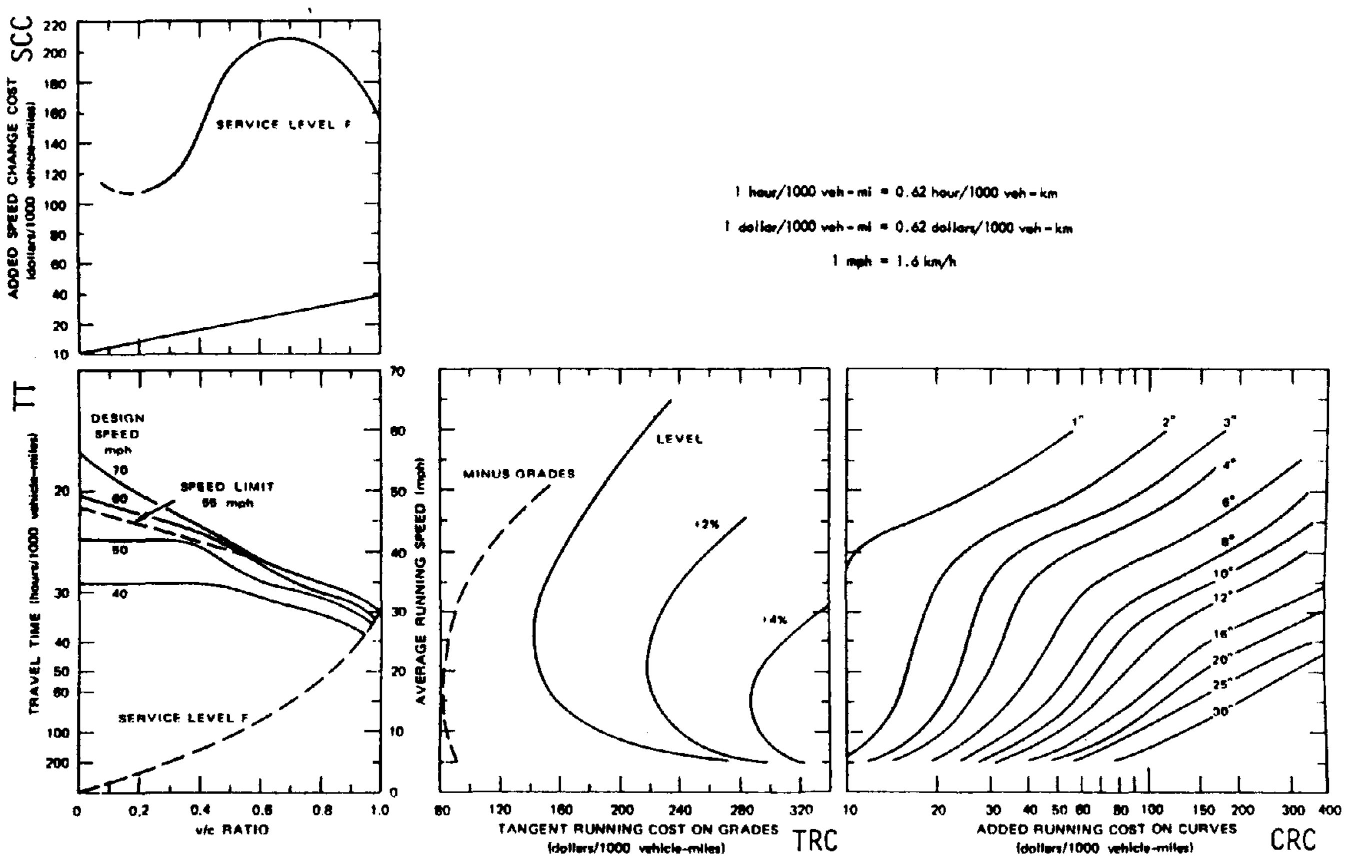


Figure 20 - Basic Section Costs for Combination Trucks on Two-Lane Highways

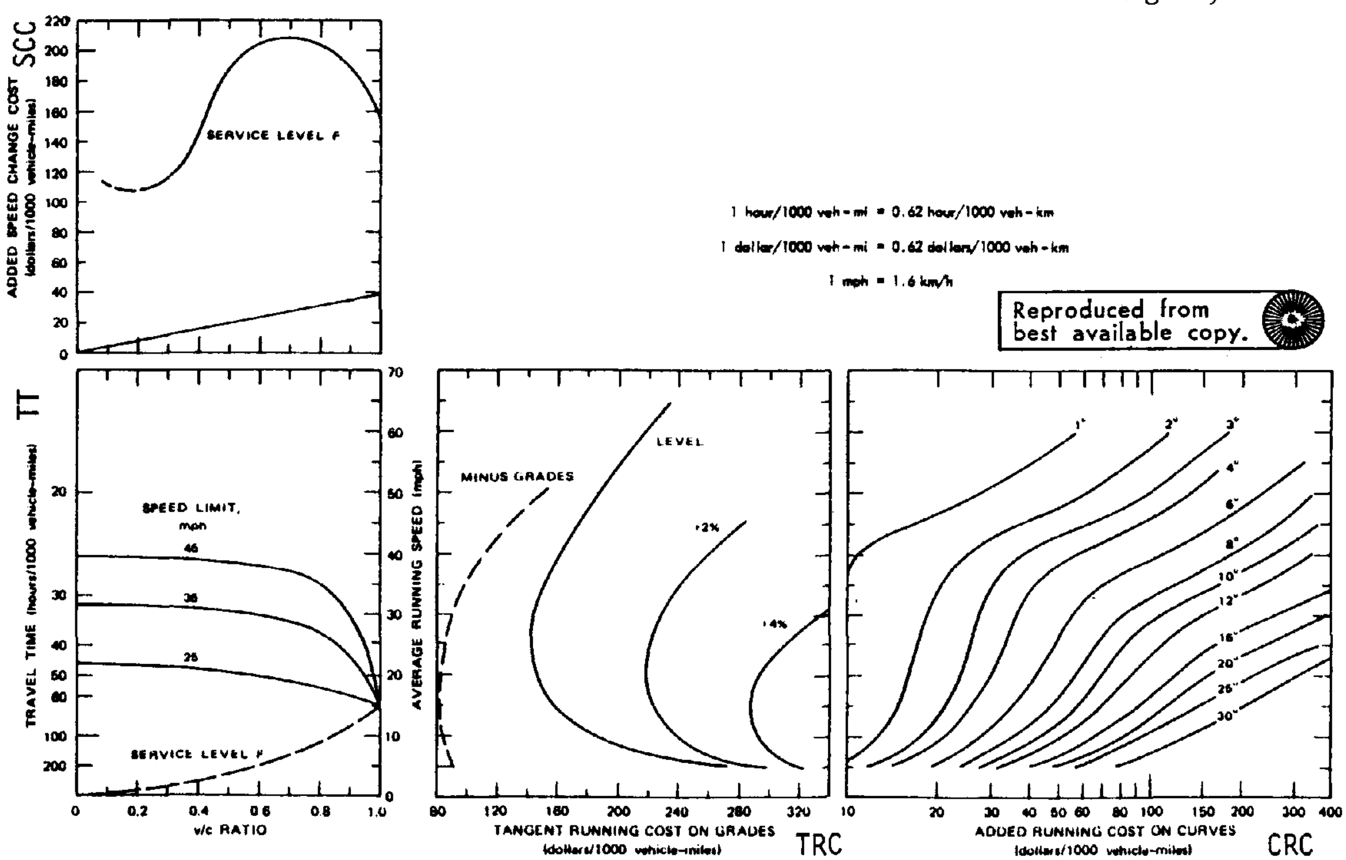


Figure 21 - Basic Section Costs for Combination Trucks on Arterials

EXAMPLE

GIVEN:
Volume: 480 vehicles/hr
Saturation Flow: 1,600 vehicles/hr
Signal Cycle Time: 60 sec
Effective Green Time: 30 sec
Intersection Approach Speed: 30 mph
5% Single Unit Trucks
5% 3-S2 Combination Trucks

SOLUTION:
$\lambda = 30/60 = 0.5$
Capacity of Approach = 0.5 × 1600 = 800
$x = 480/800 = 0.6$
(a) Average Stops per Vehicle (per Signal): 0.71
(b) Stopping Delay per Signal: 2.5 hrs
(c) Cost of Stopping: $10.30

Time Cost: 2.5 × $3.00* × 1.35†	$10.13
Running Cost: $10.30 × 1.42†	14.63
Total Cost Due to Stopping per 1,000 vehicles per Signal (excludes idling):	$ 24.76

*Assumed hourly value of time per passenger car.
†Adjustment factors for trucks in traffic stream.

ADJUSTMENT FACTORS FOR PERCENT TRUCKS IN TRAFFIC STREAM

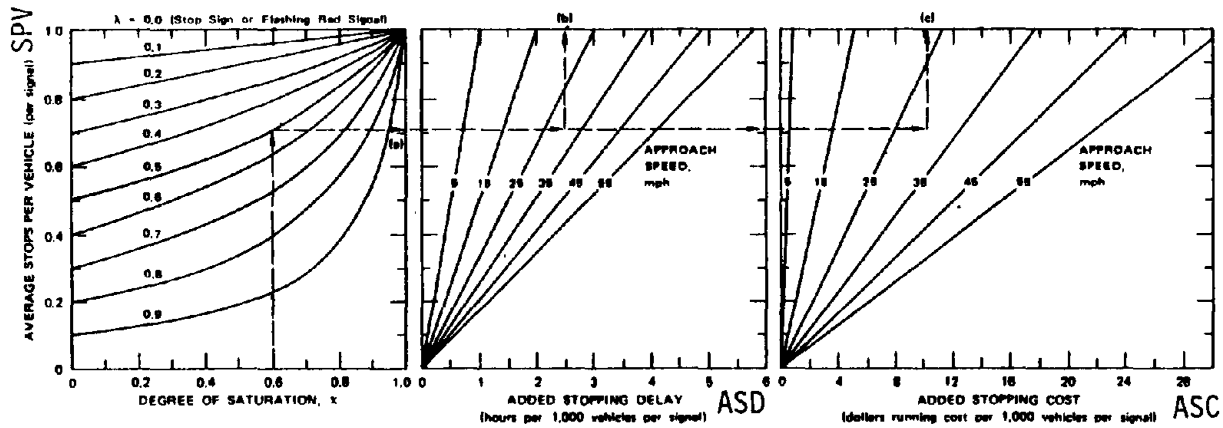
TIME COST TCF

APPROACH SPEED (mph)	SINGLE UNIT TRUCKS (percent)	3 S-2 COMBINATION DIESEL TRUCKS (percent in traffic stream)				
		0	5	10	20	100
5-20	0	1.00	1.15	1.30	1.81	4.03
	5	1.03	1.22	1.37	1.87	-
	10	1.13	1.28	1.43	1.74	-
	20	1.26	1.41	1.57	1.87	
	100	2.31	-	-	-	-
21-40	0	1.00	1.29	1.51	2.01	6.05
	5	1.10	1.35	1.60	2.11	-
	10	1.20	1.45	1.70	2.21	-
	20	1.40	1.85	1.90	2.41	-
	100	2.99	-	-		-
41-60	0	1.00	1.41	1.82	2.63	9.17
	5	1.11	1.56	1.93	2.74	
	10	1.22	1.61	2.04	2.85	
	20	1.44	1.85	2.26	3.07	
	100	3.20	-	-	-	-

RUNNING COST RCF

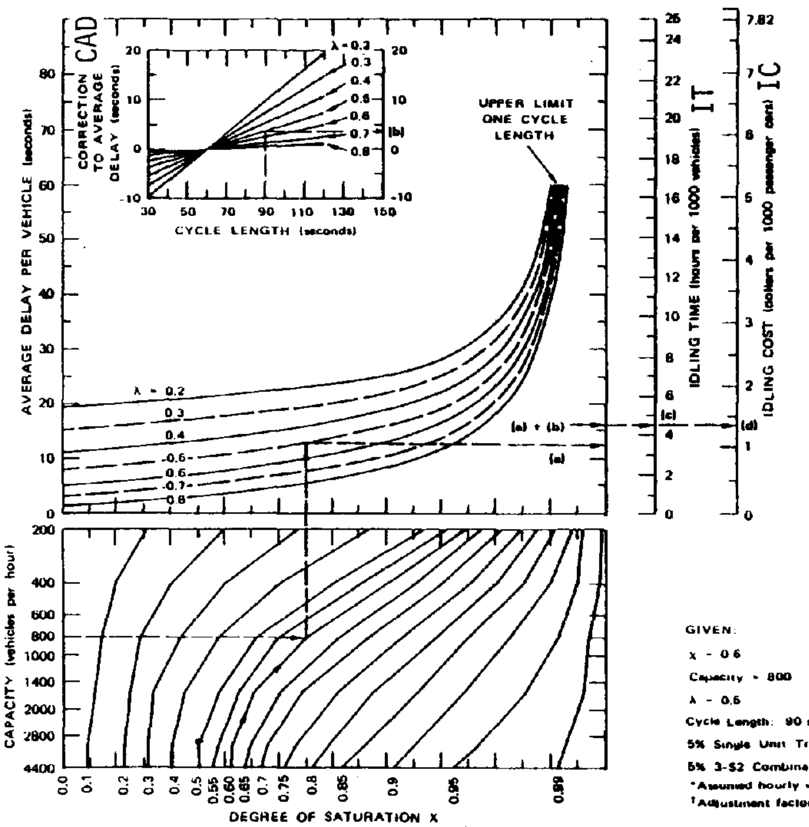
APPROACH SPEED (mph)	SINGLE UNIT TRUCKS (percent)	3 S-2 COMBINATION DIESEL TRUCKS (percent in traffic stream)				
		0	5	10	20	100
5-20	0	1.00	1.35	1.70	2.40	6.02
	5	1.08	1.43	1.78	2.48	-
	10	1.16	1.51	1.86	2.57	-
	20	1.32	[illegible]	2.03	2.73	-
	100	2.62	-	-	-	-
21-40	0	1.00	1.35	1.71	2.41	8.07
	5	1.07	1.42	1.78	2.48	-
	10	1.14	1.49	1.84	2.55	-
	20	1.27	1.63	1.99	2.69	-
	100	2.37	-	-	-	-
41-60	0	1.00	1.35	1.70	2.38	7.98
	5	1.06	1.41	1.76	2.45	-
	10	1.12	1.47	1.82	2.51	-
	20	1.24	1.59	1.94	2.63	-
	100	2.21	-	-	-	-

1 mph = 1.6 km/h



NOTE: Where X = v/λs = v/capacity s = saturation flow v = volume λ = green to cycle time ratio

Figure 22 - Costs Due to Stopping at Intersections (Excludes Idling).



NOTE: Where: x = v/λs = v/capacity s = saturation flow v = demand volume λ = green to cycle time ratio

ADJUSTMENT FACTORS FOR PERCENT TRUCKS IN TRAFFIC STREAM

IDLING TIME FACTOR ITF

		3-S2 COMBINATION TRUCKS (percent)				
		0	5	10	20	100
SINGLE UNIT TRUCKS (percent)	0	1.00	1.08	1.17	1.33	2.67
	5	1.07	1.15	1.23	1.40	–
	10	1.13	1.22	1.30	1.47	–
	20	1.27	1.35	1.43	1.60	–
	100	2.33	–	–	–	–

IDLING COST FACTOR ICF

		3-S2 COMBINATION TRUCKS (percent)				
		0	5	10	20	100
SINGLE UNIT TRUCKS (percent)	0	1.00	0.98	0.96	0.92	0.62
	5	0.99	0.98	0.96	0.92	–
	10	0.99	0.97	0.95	0.91	–
	20	0.98	0.96	0.94	0.90	–
	100	0.89	–	–	–	–

EXAMPLE

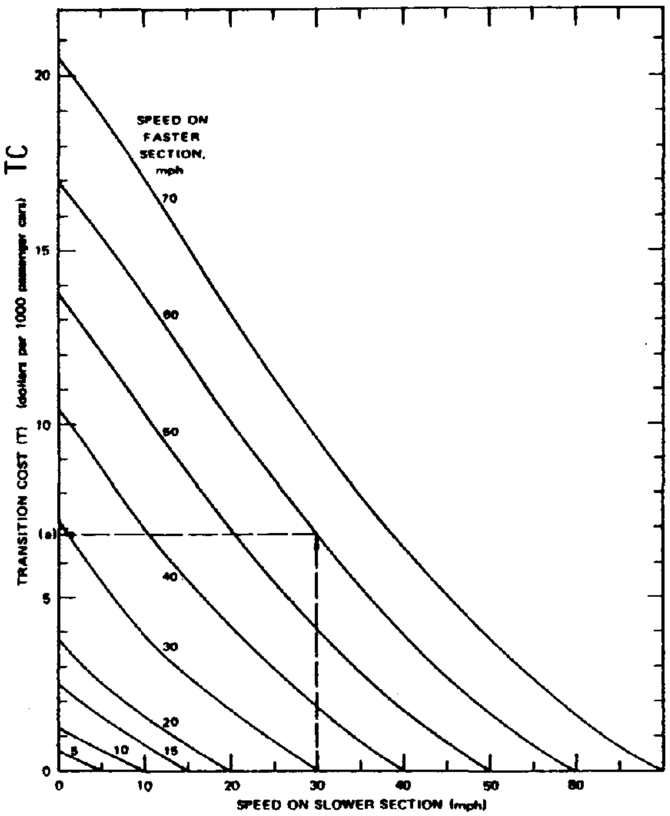
GIVEN:
$x = 0.6$
Capacity = 800
$\lambda = 0.5$
Cycle Length: 90 seconds
5% Single Unit Trucks
5% 3-S2 Combination Trucks

SOLUTION: Average Delay per Vehicle: (a) + (b) = 16.2 sec
(c) Idling Hours: 4.5 hrs
(d) Idling Cost: $1.40

Total Delay: 4.5 hrs × $3.00* × 1.161	$16.53
Total Idling Cost: $1.40 × 0.98†	1.38
Total Cost Due to Idling per 1000 Vehicles (per signal)	$16.91

*Assumed hourly value of time per passenger car.
†Adjustment factors for percent trucks in traffic stream.

Figure 23 - Costs Due to Idling at Intersections.



ADJUSTMENT FACTORS FOR TRUCK TRAFFIC TCAF

SINGLE UNIT TRUCKS (percent)	3-S2 COMBINATION TRUCKS (percent)											
	0	1	2	3	4	5	6	8	10	20	50	100
0	1.00	1.10	1.19	1.29	1.38	1.48	1.57	1.77	1.96	2.92	5.79	10.58
1	1.02	1.12	1.21	1.31	1.40	1.50	1.59	1.79	1.98	2.94	5.81	-
2	1.04	1.14	1.23	1.33	1.42	1.52	1.61	1.81	2.00	2.96	5.83	
3	1.06	1.16	1.25	1.35	1.44	1.54	1.63	1.83	2.02	2.98	5.85	-
4	1.08	1.18	1.27	1.37	1.46	1.56	1.65	1.85	2.04	3.00	5.87	-
5	1.10	1.20	1.29	1.39	1.48	1.58	1.67	1.87	$\boxed{2.06}$	3.02	5.89	-
6	1.12	1.22	1.31	1.41	1.50	1.60	1.69	1.89	2.08	3.04	5.91	-
8	1.16	1.26	1.35	1.45	1.54	1.64	1.73	1.93	2.12	3.08	5.93	-
10	1.20	1.30	1.40	1.49	1.58	1.68	1.77	1.97	2.14	3.12	5.97	-
20	1.41	1.50	1.60	1.70	1.78	1.88	1.94	2.17	2.34	3.32	6.17	-
50	2.02	2.12	2.21	2.31	2.39	2.49	2.58	2.78	2.96	3.93	6.78	-
100	3.04	-	-	-	-	-	-	-	-	-	-	-

EXAMPLE

GIVEN:

A stream of traffic consists of 5% single unit trucks and 10% combination trucks. The traffic is travelling at 60 mph and enters a slower section on which the speed is 30 mph.

SOLUTION:

(a) Passenger Car Cost = $6.75

T = $6.75 × 2.06

= $13.91 per 1000 Vehicles

1 mph = 1.6 km/h

Figure 24 - Transition Costs.

APPENDIX B

SUPPLEMENTARY ENGINEERING STUDIES

This Appendix describes supplementary engineering studies that are often employed to investigate operational and safety problems at freeway-arterial interchanges. They include: capacity analysis, travel time and delay speed, traffic conflict and erratic maneuver, traffic signal, sight distance, turning radius, and skid resistance studies. Additional information about a problem location can often be obtained from interviews with drivers, police officers and local maintenance personnel familiar with the interchange.

Capacity Analyses

The capacity of an interchange element is defined as the maximum number of vehicles that can pass through the element in a given period of time (usually one hour) under prevailing conditions. The purpose of a capacity analysis is to enable the description of traffic operations of an interchange element in relation to its capacity.

The Highway Capacity Manual[17] (HCM) provides analytical procedures to determine the capacity and Level of Service of most interchange elements. The procedures pertaining to freeways and interchanges have been organized and expanded by Leisch.[22] The following procedures for operational analyses of interchange elements are suggested:

Interchange Element	Source
Crossroad Ramp Terminals with STOP-sign or signal control (diamond or parclo ramps)	HCM Chapter 6 - At-Grade Intersections
Crossroad Ramp Terminals with free-flow conditions (cloverleaf outer-connection and loop ramps)	HCM Chapter 7 - Weaving Areas HCM Chapter 8 - Ramps
Freeway Ramp Terminals (merging and diverging areas)	HCM Chapter 8 - Ramps
Ramp Proper	See Leisch[22], pp. 25-26, or AASHTO policy[3,4]
Mainline Freeway	HCM Chapter 9 - Freeways
Arterial Crossroad	HCM Chapter 10 - Streets and Highways without Access Control

TABLE 17

QUALITATIVE DEFINITIONS OF LEVELS OF SERVICE

Level of Service	Flow Characteristics
A	Free flow
B	Stable flow (upper speed range)
C	Stable flow
D	Approaching unstable flow
E	Unstable flow
F	Forced flow

The measure used by the Highway Capacity Manual for the quality of traffic operations is the Level of Service. Six Levels of Service, from A through F, are defined qualitatively in Table 17. The highest level of Service, A, represents completely free-flow conditions. Levels B through D are increasing states of congestion, while Level of Service E represents the capacity condition. At Level of Service F, where traffic volumes exceed the capacity, queues of vehicles tend to form and excessive delays occur.

It is important to recognize that the quantitative methods of defining the Level of Service vary greatly among the types of interchange elements. For a signalized intersection or ramp terminal approach, where interrupted flow conditions prevail, the Level of Service is based on the "load factor."* For uninterrupted flow conditions on freeways and arterial streets, Levels of Service are based on both operating speed and volume-to-capacity ratio (V/C). Finally, the Level of Service for a weaving area is based on the weaving volumes and the length of the weaving area.

A major effort is currently underway to develop a revised and expanded HCM. The completion of this effort is still several years away, but tentative procedures for analysis of at-grade intersections, freeways and weaving areas have been presented in Transportation Research Circular 212, "Interim Materials on Highway Capacity," published in January 1980. The Level of Service concept has been retained as the basis for traffic operational analysis, but major changes in the procedures to determine the Level of Service have been proposed For example, a critical movement analysis technique is suggested to evaluate the Level of Service for an entire at-grade intersection rather than treating each approach separately, as in the 1965 HCM. When the revised HCM is completed and published, its use is recommended. In the meantime, the interim procedures may be used on a trial basis.

To perform traffic service and capacity analyses of all portions of an interchange, a user would need:

. Peak hour traffic volume turning movements and traffic composition for each interchange elements;

. Geometrics of intersections approaches, ramps, weaving areas, freeway lanes and arterial lanes from construction plans or field measurements;

. Design speed for free-flow sections;

. Lateral clearance to roadside obstacles;

. Phase lengths and cycle time for all traffic signals;

---

* The proportion of signal phases continuously and completely utilized by traffic.

- Presence or absence of parking on intersection approaches;
- Metropolitan area population; and
- Location within metropolitan area.

The Level of Service and capacity of each interchange element should be evaluated as interchanges with operational problems. The analysis will identify those interchange elements with deficient traffic service and will, thus, help to pinpoint the cause of the problem.

Travel Time and Delay Studies

Travel time and delay studies are performed to determine the location and extent of delay to motorists and to determine average travel speeds. There is no single, generally applicable definition of delay, but the concept describes travel time in excess of an "ideal" condition and occurs when a vehicle's progress is impeded by other vehicles. Travel time and delay studies can be used both to identify the locations and causes of delay and to quantify delay for operational evaluations, such as those discussed in Section VII.

Two techniques for measuring route delay are discussed below: the test vehicle method and the license plate method. Both methods could be applied to determine travel times for any portion of an interchange. A delay measurement technique applicable to at-grade intersections, such as crossroad ramp terminals, is also discussed.

In the test vehicle method, a car is driven over a selected path through the interchange in a series of runs to obtain representative travel times. Usually, the test driver attempts to approximate the average speed of vehicles in the traffic stream. Two observers with two stop-watches are generally required for this method. The first stop-watch is started at the beginning of each run and the time, cause, and location of all stops or slowing are recorded. The second stop-watch is used to determine the length of stopped-time delays. These two types of data combined can be used to describe both freely-flowing traffic on a freeway or arterial and intersection queueing situations. It may be possible for the vehicle driver to record all information if a dash mounted stop-watch and a tape recorder are used.

In the license plate method, observers are stationed at the beginning and end of a section. In complex situations, as might occur at an interchange, more than two observer locations are needed. Each observer uses a synchronized stop-watch and records the time and license plate number (usually the last three digits) of each vehicle passing his station. The license plate numbers are matched after the study and the travel time is determined as the difference between the two recorded times. This method has only limited application because of the large effort required to reduce the data.

In both route delay techniques, travel time information may be presented as either the average travel time or the average travel speed maintained on the section. If stopped delays were recorded, then the average stopped delay for each run and the causes of the delay can be determined. It is also possible to determine the vehicle delay rate by computing the observed travel time in minutes per kilometer (or per mile) and comparing this value with the level of service recommended for the study section. The difference between the two values (minutes/kilometer) is the delay rate. The delay rate multiplied by the volume gives a vehicle delay rate in vehicle-minutes per mile.

Intersection delay is a useful measure of the operational performance of an intersection because, unlike the "load factor" used by the Highway Capacity Manual, it has a direct relationship to the experience of motorists using the intersection. Intersection delay can be defined in many different ways and, consequently, several different delay measures have been used. The most inclusive delay measure is approach delay which is the difference between the time required to pass through the intersection approach and the time required by an unimpeded vehicle moving at the free-flow speed of traffic to travel the same distance. Stopped delay, as the name implies, is the time a vehicle is stopped on an intersection approach. The time-in-queue delay is the time from the first stop until the vehicle crosses the stop line. The percent of vehicles stopping is a general indication of delay and is defined as the number of vehicles that incur stopped delay divided by the total volume of vehicles that cross the stop line.

Approach delay is generally accepted as the best indicator of intersection performance, but it is quite difficult to obtain directly in the field. The objective of a recent FHWA study conducted by JHK and Associates [32] was to select the type of delay most appropriate for use at signalized intersections and to develop a field method for collecting data to estimate the most appropriate type of delay. The JHK study concluded that approach delay was "the delay type...most representative of efficiency of operation of an intersection." Four field methods were tested for accuracy and precision in comparison to delay measurements made from time lapse film. From these studies it was concluded that the point sample stopped delay method and percent of vehicles stopping method were best suited for easy and accurate field use.

JHK also developed a manual to explain the collection and analysis of these delay measures to potential users.[33] The recommended procedures include the simultaneous conduct of two types of studies, which requires two or four observers per approach. For a typical location with moderate traffic volumes and queue lengths, one observer would conduct the stopped time study and the second observer would conduct the percent stopping study. Each approach is studied for a minimum of 13 minutes. The manual also recommends that studies be conducted during both peak and off-peak periods.

The stopped time study is done by sampling (i.e., recording) the number of vehicles stopped on the approach at 13 or 15 second intervals. In the percent stopping study, each vehicle is categorized as "stopping" or "not stopping." The study data, together with regression equations developed in the research, can be used to derive:

- Stopped delay, in vehicle-seconds;
- Approach delay, in vehicle-seconds;
- Stopped delay per vehicle, in vehicle-seconds per vehicle;
- Approach delay per vehicle, in vehicle-seconds per vehicle; and
- Percent of vehicles stopping.

Although the JHK study was limited to signalized intersections, the point sample stopped delay technique described in the user's manual is also applicable to STOP-sign controlled intersections. However, no regression relationships to convert the raw data to stopped delay or approach delay have been developed for STOP-signs.

Speed Studies

Speed studies are used to measure actual vehicle speeds for a freeway, crossroad or ramp location and to investigate the need for changes in legal or advisory speed limits.

Spot speed studies are intended to determine the actual distribution of traffic speeds on an interchange element. The speeds of a sample of vehicles are measured at a point or over a short distance to estimate the speed distribution of the entire traffic stream. Spot speed studies are useful when there is evidence that vehicle speeds are too high for roadway conditions. Typical symptoms that would indicate need for speed studies are run-off-road accidents on ramps or curves, rear-end accidents near intersections, rear-end accidents in on-ramp merging areas and speed limits or advisory speeds that do not seem to match existing vehicle speeds.

The most appropriate location for a speed study should be determined on the basis of problem symptoms identified at the interchange. Generally, the site of a speed study should be removed from the influence of STOP-signs and signals. Radar is often the most convenient method for measuring speeds although stop-watch methods can also be employed.

Spot speed studies are usually made during off-peak hours. One recommended method is to sample for one hour at three times during the day: Once between 9:00 a.m. and 12:00 p.m., once between 3:00 p.m. and 6:00 p.m., and once between 8:00 p.m. and 10:00 p.m. Normally the speeds of at least 50 vehicles, and preferably 100, should be measured. Only the speeds of unimpeded vehicles are usually measured, so that the results represent the "desired" speeds of drivers.

The speed measurements can be recorded by tallying the number of vehicles (usually divided into cars and trucks) in a one to two mph range.

The speed measurements are analyzed to determine the characteristics of the speed distribution at the study site. Some of the most frequently used speed distribution characteristics are the mean speed, the 85th percentile speed, the standard deviation, and the pace.

The mean speed is the average speed of all observed vehicles. It can be found by multiplying the mean speed of each group by the number of observations in that group, summing the products, and dividing by the total number of observations.

The 85th percentile speed is the speed below which 85 percent of the observed vehicles travel. This speed is sometimes referred to as the critical speed and used to set speed limits.

The standard deviation is a measure of the dispersion of the observed speeds and can be estimated by the formula:

$$\sigma = \sqrt{\frac{\Sigma f_N \ (U_N)^2}{(\Sigma f_N) - 1} - \left(\frac{\Sigma f_N U_N}{\Sigma f_N} \right)^2}$$

where σ = Standard deviation,
U_N = Midpoint of Nth speed range; and,
f_N = Number of vehicle observed in Nth speed range.

The pace is another means of measuring the central tendency of the speed measurements. It is the 16 km/h (10 mph) range in which the highest number of observations were recorded. The percentage of vehicles in the pace is another measure of the dispersion of the vehicle speeds. Safety studies have shown that accident rates increase as the dispersion of speeds increase.

The measurement of speed profile is a useful technique to determine the longitudinal variation of vehicle speeds. Speed measurements can be made from a test vehicle, from time-lapse film or from a series of spot speed studies. The latter technique is currently being used by Michael Baker, Jr., Inc. in a study of speed profiles on ramps. [7]

Speed limits should be established on the basis of the 85th percentile speed determined from a spot speed study. Advisory speed limits for horizontal curves should be established on the basis of (1) the standard curve formula presented in AASHTO Policies [3,4] or (2) trial speed runs using a ball-bank indicator.

Traffic Conflict and Erratic Maneuver Studies

Traffic conflicts and erratic maneuver studies have become increasingly popular as methods for investigating potential safety problems without the need for long accident histories to develop. Both traffic conflicts and erratic maneuvers are traffic operational surrogates for accident data.

A traffic conflict is a traffic event involving two or more road users (usually vehicles) in which one user performs some atypical or unusual action, such as a change in direction or speed that places another user in jeopardy of a collision unless an evasive maneuver is undertaken. Note that a traffic conflict by definition must involve two or more vehicles. Traffic conflict studies involve the measurement of conflict frequencies at locations with a potential for multiple vehicle accidents, such as at-grade intersections.

Midwest Research Institute has recently completed a study entitled "Application of Traffic Conflict Analysis at Intersections." [16] The final report for this project contains a procedural manual for traffic conflicts observers and an instructors' and engineers' guide that describes the use the conflicts technique and how to interpret the data. The MRI report is recommended as the basic source for the engineer interested in conducting a traffic conflicts study. Several important conclusions of the study are:

1. The traffic conflicts technique is an excellent tool for diagnosing safety/operational problems at identified problem intersections.

2. Traffic conflicts data should be viewed as supplements to, not replacements of, accident data.

3. Traffic volume data should be counted with traffic conflicts.

4. Raw conflicts counts are not as useful as certain sums or rates.

Further research is needed in other areas of application including midblock locations, freeway entrances and exits, weaving areas, construction zones, and pedestrian crossings.

Erratic maneuvers are atypical or unusual actions of a single vehicle. Erratic maneuvers can be observed at intersections, gore areas, or midblock sections, and the types of erratic maneuvers are virtually limitless. Erratic maneuvers can be studied alone or in coordination with a traffic conflicts study. Types of erratic maneuvers that can be observed at interchanges include wrong-way movements on ramps or freeway, gore area encroachments, shoulder encroachments, stopping or backing near gore areas and various traffic control device violations.

NCHRP Report 145 "Improving Traffic Operations and Safety at Exit Zone Areas," presents the results of erratic maneuvers observations at nine exit sites.[37] Eight types of gore area erratic maneuvers were classified. This report concluded that erratic maneuver rates greater than 0.2 percent (two erratic maneuvers per 1,000 observed vehicles) at gore areas are an indication that corrective treatments should be considered.

Traffic Signal Studies

This section presents two kinds of traffic signal studies: signal warrant studies and signal design reviews. Other types of studies appropriate for signalized intersections, such as capacity analyses, intersection delay studies and traffic conflict studies have been described previously.

Traffic signals should not be installed or maintained in operation unless the location meets at least one of the eight traffic signal warrants established in the Manual on Uniform Traffic Control Devices for Streets and Highways[43] (MUTCD). The MUTCD warrants are applicable both to conventional intersections and to at-grade ramp terminals. These warrants are:

Warrant 1 - Minimum vehicular volume
Warrant 2 - Interruption of continuous traffic
Warrant 3 - Minimum pedestrian volume
Warrant 4 - School crossing
Warrant 5 - Progressive movement
Warrant 6 - Accident experience
Warrant 7 - Systems
Warrant 8 - Combination of warrants

The MUTCD states that six different types of data are desirable for signal warrants studies. These are:

1. Hourly traffic counts for 16 hour of a representative day;

2. Turning movement counts for each 15 minute period during the 2 hour a.m. and p.m. peaks;

3. Pedestrian counts during the same hours as turning movement counts and during the peak pedestrian time of day. If young or old pedestrians are a special problem the pedestrians should be classified by age, as: under 13 years, 13-60 years, or over 60;

4. The 85 percentile speed of all vehicles on the uncontrolled approaches to the location;

5. A condition diagram which shows the physical layout of the intersection; and

6. A collision diagram which shows at least one year of accident experience at the intersection.

It is also desirable for a more precise understanding of the intersection operation to measure: vehicle-seconds delay for each approach during the peak hours; the distribution of gaps available in major street traffic; the 85 percentile speeds on controlled approaches to the location; and, pedestrian delay time for two 30 minute peak pedestrian delay periods.

Several design factors are critical in the installation of new signals or in the review of problems as existing signalized locations. A complete list of these design factors is given in the MUTCD, but some of the most critical factors are:

1. Two signal faces should be available for through traffic on each approach. The minimum visibility distance is dependent on the 85 percentile speed on each approach and varies from 30m (100 ft) at 32 km/h (20 mph) to 213m (700 ft) at 97 km/h (60 mph).

2. Where possible at least one and preferably both signals faces should be not less than 12m (40 ft) or more than 37m (120 ft) beyond the stop line.

3. Where possible at least one and preferably both signal faces should be located between two lines intersecting the center of the approach lanes at the stop line, one line making an angle of 20° right of the approach center line extended and the other making an angle of 20° left of the approach center line extended.

4. Signal supports and controller cabinets should be placed as far as practicable from the traveled way without affecting signal visibility.

5. Signalized locations within 0.8 km (0.5 mile) of one another along a major route or in a network of major intersecting routes should be operated in coordination, preferably with interconnected controllers.

Sight Distance Studies

Sight distance is the length of highway visible to the driver. Three types of sight distance requirements that are applicable to freeway-arterial interchanges are stopping sight distance, decision sight distance and intersection sight distance.

AASHTO design policies include requirements for minimum stopping sight distance to assure that the driver always has adequate time to see an object, react and brake to a halt.[3,4] Minimum and desirable sight distances, based on AASHTO policies, are given in Table 19. The stopping sight distance is measured from the driver's eye height of 1.14 m (3.75 ft)* to an object 150mm (0.5 ft) high. Stopping sight distance must be considered in the design of both horizontal and vertical curves.

It has been recognized that additional sight distance, over and above that required to stop, is required at locations, such as the approaches to exit ramps, where drivers must make decisions. This longer sight distance requirement, known as decision sight distance, was recently studied by McGee, et al.[25] Their recommended decision sight distance criteria are also presented in Table 18.

The final type of sight distance is applicable to intersections such as crossroad ramp terminals. To enter or cross an intersecting roadway, a driver needs adequate sight distance along the intersecting approach to perceive oncoming traffic. The sight distance requirements for intersections depend on the approach speed of oncoming traffic. Intersection sight distance is measured from the driver eye height to an object 1.4m (4.5 ft) high. Corner sight obstructions, such as underpass or overpass structures, often restrict the intersection sight distance at crossroad ramp terminals.

Accident patterns including angle accidents at intersections and rear-end accidents at horizontal curves, vertical crests and decision points may indicate that the sight distance available to the driver is inadequate. In such situations, it is recommended that the sight distance be measured in the field or from plans and compared with applicable sight distance requirements.

---

* Based on recent field surveys, a lower eye height, 1.07m (3.50 ft), has been recommended.

TABLE 18

COMPARISON OF STOPPING SIGHT DISTANCE AND DECISION SIGHT DISTANCE REQUIREMENTS[25]

Design Speed (km/h)[a]	Stopping Sight Distance Requirement (m)		Decision Sight Distance Requirement (m)
	Minimum	Desirable	
40	40	40	115-160
60	75	90	175-235
80	115	145	230-315
100	160	210	310-400
120	205	290	360-470
140	250	325	420-550

Design Speed (mph)	Stopping Sight Distance Requirement (ft)		Decision Sight Distance Requirement (ft)
	Minimum	Desirable	
30	200	200	450-625
40	275	325	600-825
50	375	475	750-1025
60	525	650	1000-1275
65	550	725	-
70	625	850	1100-1450
75	675	950	-
80	750	1100	1250-1650

a Sight distance requirements in metres for design speed in km/h were derived from interpolation and extrapolation of the sight distance requirements in feet presented in Reference 25. The computed sight distance requirement, rather than the value rounded for design, was used as the basis for conversion.

Turning Radius Studies

The need for turning radius studies may be indicated by a pattern of side swipe accidents by vehicles traveling in opposite directions, by rear-end accidents at right-turn lanes or by truck encroachments on shoulders or curbs. The adequacy of curb radii can be judged by comparison with truck turning radius diagrams in the AASHTO policies or through field studies of truck speeds and erratic maneuvers. If trucks are unable to complete turning movements without encroaching on opposing lanes, shoulders or curbs, redesign of the curb radii is indicated.

Skid Resistance Studies

Concentrations of wet-pavement, skidding or ran-off road accidents in the interchange area may indicate inadequate tire-pavement skid resistance. The most common measure of skid resistance is the skid number (SN) at 65 km/hr (40 mph), which is determined with a locked-wheel skid tester according to the requirements of ASTM E-274-77.[5] Skid testing is recommended at sites where wet-pavement accident problems are identified. Where skidding problems are suspected at a horizontal curve, the skid resistance study should be conducted in coordination with a study of superelevation, drainage and vehicle speeds.

APPENDIX C

SAFETY EFFECTIVENESS EVALUATION USING THE t-TEST

This appendix presents a step-by-step procedure for application of the two-sample t-test to safety effectiveness evaluations. This procedure is more generally applicable than the Chi-Square procedure presented in Section IX, but is computationally more complex. Its use is recommended if a general program of performing safety effectiveness evaluations is initiated. The following procedure should be followed:

Step 1 - Select the area(s) of the interchange to be evaluated, including all portions of the interchange whose safety experience could be affected by the improvement.

Step 2 - Select two study periods--one before and one after the improvement. Both periods should preferably be at least three years in length.

Step 3 - Select the safety measure(s) of effectiveness to be used. The total accident rate is the usual measure of effectiveness, but other measures, such as accident rates by severity level (fatal and injury/property-damage-only) by accident type (rear-end/right-angle/head-on, etc.), by pavement surface condition (wet/dry), and by light condition (day/night), may be used where appropriate to a particular countermeasure.

Step 4 - For each year of the before and after study period obtain the accident frequency and traffic volume for the area to be evaluated. For a roadway section, calculate an accident rate for each year as:

$$AR = \frac{N(10^6)}{(D)(ADT)(L)}$$

where AR = Accident rate (accidents per million vehicle-kilometres or per million vehicle-miles);
N = Accident frequency:
ADT = Average daily traffic (vehicles/day);
D = Number of days in period (e.g., 1 year = 365 days); and
L = Length of section (kilometres or miles).

For a ramp, an intersection or an entire interchange, calculate the accident rate for each year as:

$$AR = \frac{N(10^6)}{(D)(ADT)}$$

where AR = Accident rate (per 10^6 vehicles).

For analysis of an entire interchange, the variable ADT should be the sum of the average daily traffic volume entering the interchange on each mainline freeway and crossroad approach (or the sum of the two-way traffic volumes divided by 2). For each intersection or ramp, the variable ADT represents the average daily traffic volume on the ramp or entering the intersection. If less than two years of accident data are available in either the before or after period, the accident frequency (N) should be determined for each quarter year (three-month period) and the number of days (D) should be adjusted accordingly. Quarter years should not be used except when absolutely necessary because uncontrolled seasonal variations may be present.

Step 5 - Compute an average accident rate ($\overline{AR_b}$ and $\overline{AR_a}$) for both the before and after periods. Determine the percent accident rate reduction as:

$$\text{Percent Reduction} = \frac{AR_b - AR_a}{AR_a} \times 100$$

Step 6 - For each year (or quarter year) of the before and after study periods, calculate a transformed accident rate as:

$$X = \sqrt{AR + 0.375}$$

This transformation, developed by Anscombe,[6] converts the Poisson-distributed accident data to a normal distribution. The square-root transformation is generally applicable, not just to the Poisson distribution, but to any data where the variance is proportional to the mean.

Step 7 - Compute the mean and variance of the transformed accident rate for the before periods. These are:

$$\bar{X}_b = \frac{\Sigma X_b}{N_b}$$

and,

$$S_b^2 = \frac{N_b \, \Sigma X_b^2 - (\Sigma X_b)^2}{N_b \, (N_b - 1)}$$

where, X_b = Accident rate for a particular before year (or quarter year);
N_b = Number of years (or quarter years) in before period;
$\bar{X}_b$ = Mean accident rate for before period (accidents per 10^6 vehicles); and
S_b^2 = Variance of accident rate for before period.

The mean and variance of accident rate for the after period, $\bar{X}_a$ and S_a^2, should be calculated in a manner analogus to the before period.

Step 8 - Determine whether the variances from the before and after periods are equal.

a. Compute the following F-statistic:

$$F = \frac{S_1^2}{S_2^2}$$

where S_1^2 = Larger value of S_b^2 and S_a^2; and
S_2^2 = Smaller value of S_b^2 and S_a^2.

This F-statistic has (N_1-1, N_2-1) degrees of freedom; where:

N_1 = Number of periods (years or quarter years of accident data) for S_1^2, and
N_2 = Number of periods (years or quarter years of accident data) for S_2^2.

b. Obtain a critical value of the F-distribution (F_c) for (N_1-1, N_2-1) degrees of freedom from Table 19.

c. If $F < F_c$, then the variances are not significantly different and are presumed to be equal.

If $F \geq F_c$, then the variances are significantly different and are presumed to be unequal.

Step 9 - Determine whether the before and after accident rates are significantly different.

a. Compute the following t-statistic:

$$t = \frac{\bar{X}_b - \bar{X}_a}{\sqrt{(N_a-1)\, S_a^2 + (N_b-1)\, S_b^2}} \qquad \sqrt{\frac{N_a N_b\, (N_b + N_a - 2)}{N_a + N_b}}$$

TABLE 19

CRITICAL VALUES OF THE F-DISTRIBUTION
(95 percent confidence)

Enter table with V_1 (degrees of freedom for numerator) and V_2 (degrees of freedom for denominator to determine $F(V_1, V_2)$.

V_1

V_2	1	2	3	4	5	6	7	8	9	10
1	161.4	199.5	215.7	224.6	230.2	234.0	236.8	238.9	240.5	241.9
2	18.51	19.00	19.16	19.25	19.30	19.33	19.35	19.37	19.38	19.40
3	10.13	9.55	9.28	9.12	9.01	8.94	8.89	8.85	8.81	8.79
4	7.71	6.94	6.59	6.39	6.26	6.16	6.09	6.04	6.00	5.96
5	6.61	5.79	5.41	5.19	5.05	4.95	4.88	4.82	4.77	4.74
6	5.99	5.14	4.76	4.53	4.39	4.28	4.21	4.15	4.10	4.06
7	5.59	4.74	4.35	4.12	3.97	3.87	3.79	3.73	3.68	3.64
8	5.32	4.46	4.07	3.84	3.69	3.58	3.50	3.44	3.39	3.35
9	5.12	4.26	3.86	3.63	3.48	3.37	3.29	3.23	3.18	3.14
10	4.96	4.10	3.71	3.48	3.33	3.22	3.14	3.07	3.02	2.98

TABLE 20

CRITICAL VALUES OF t-DISTRIBUTION
(95 percent confidence)

Degrees of Freedom	t_c	Degrees of Freedom	t_c
1	6.314	16	1.746
2	2.920	17	1.740
3	2.353	18	1.734
4	2.132	19	1.729
5	2.015	20	1.725
6	1.943	21	1.721
7	1.895	22	1.717
8	1.860	23	1.714
9	1.833	24	1.711
10	1.812	25	1.708
11	1.796	26	1.706
12	1.782	27	1.703
13	1.771	28	1.701
14	1.761	29	1.699
15	1.753	inf	1.645

b. Determine a critical value of the t-distribution (t_c) from Table 20. If the variances were found to be equal in Step 7, then the critical value from Table 20 should have ($N_a + N_b - 2$) degrees of freedom. If the variances were found to be unequal in Step 7, the degrees of freedom for the critical value should be reduced to:

$$\frac{\left(\frac{S_b^{\,2}}{N_b} + \frac{S_a^{\,2}}{N_a}\right)^2}{\frac{(S_b^{\,2}/N)^2}{N_b - 1} + \frac{(S_a^{\,2}/N_a)^2}{N_a - 1}}$$

c. If $t \geq t_c$, then the mean accident rates for the before and after periods are significantly different and the improvement project is presumed to have been effective.

If $t \leq t_c$, then the mean accident rates for the before and after periods are not significantly different and the improvement project cannot be proven statistically to have been effective.

<u>Step 10</u> - Report the percent accident rate reduction computed in Step 5 and its statistical significance determined in Step 8.

APPENDIX D

TYPICAL ACCIDENT RATES FOR INTERCHANGE ELEMENTS

TABLE 21

TYPICAL ACCIDENT RATES BY RAMP TYPE [23]

Ramp Type	Accident Rates (Accidents per Million Vehicles) On-Ramp	Off-Ramp
1. Diamond Ramps	0.40	0.67
2. Trumpet Ramps	0.84	0.85
3. Cloverleaf Ramps Without C-D Roads	0.72	0.95
4. Cloverleaf Ramps With C-D Roads	0.45	0.62
5. Loops Without C-D Roads	0.78	0.88
6. Cloverleaf Loops With C-D Roads	0.38	0.40
7. Left Side Ramps	0.93	2.19
8. Direct Connections	0.50	0.91
9. Button Hook Ramps	0.64	0.96
10. Scissors Ramps	0.88	1.48
AVERAGE	0.59	0.95

TABLE 22

ACCIDENT RATES FOR ACCELERATION LANES, BY TYPE OF CONNECTED RAMP[28]

Associated Ramp Type	Ramp of a diamond	Outer Connection	Loop	Direct/semi-direct connection
Number of lanes examined	837	953	665	114
Mean accident rate (1)	2.09	2.68	2.84	4.14
95% Confidence Interval	1.960 - 2.220	2.583 - 2.777	2.684 - 2.996	3.384 - 4.896

(1) Per million vehicles through the lane.

TABLE 23

ACCIDENT RATES FOR DECELERATION LANES, BY TYPE OF CONNECTED RAMP[28]

Associated Ramp Type	Ramp of a diamond	Outer Connection	Loop	Direct/semi-direct connection
Number of lanes examined	841	712	894	119
Mean accident rate (1)	1.91	2.80	2.82	4.85
95% Confidence Interval	1.809 - 2.011	2.645 - 2.955	2.705 - 2.935	4.201 - 5.499

(1) Per million vehicles through the lane.

TABLE 24

ACCIDENT, INJURY, AND FATALITY RATES AT INTERCHANGES WITH AND WITHOUT C-D ROADWAYS [28]

Type of Interchange	Accident Rate (1)	Injury Rate (1)	Fatality Rate (1)
With C-D Roadways	8.16	2.03	0.17
Without C-D Roadways	9.48	3.59	0.16
Total	9.37	3.47	0.16

(1) Per million vehicles through the unit.

TABLE 25

ACCIDENT, INJURY, AND FATALITY RATES FOR CLOVERLEAF INTERCHANGES [28]

	Ramp Type					
Maximum	Outer Connection			Loop		
Degree of Curvature (2)	Accident Rate (1)	Injury Rate (1)	Fatality Rate (1)	Accident Rate (1)	Injury Rate (1)	Fatality Rate (1)
5	6.16	2.44	0.00	1.01	0.84	0.00
5 - 8:59	6.12	1.71	0.31	18.26	18.26	0.00
9 - 14:59	5.85	2.47	0.14	1.88	1.88	0.00
15 - 23:59	9.59	3.48	0.18	11.53	4.81	0.09
24 - 35:59	11.47	4.11	0.12	6.19	2.05	0.00
36+	11.92	4.03	0.00	9.23	3.03	0.05
Total	8.44	3.03	0.13	8.23	2.81	0.04

(1) Per million vehicles through the unit.
(2) Notation is degrees:minutes.

TABLE 26

ACCIDENT, INJURY, AND FATALITY RATES FOR RAMPS OF DIAMOND INTERCHANGES[28]

Length of Ramp	Accident Rate (1)	Injury Rate (1)	Fatality Rate (1)
30m(100 ft)	2.35	0.00	0.00
60m(200 ft)	9.19	0.33	0.00
90m(300 ft)	5.49	1.61	0.04
120m(400 ft)	5.93	1.74	0.00
150m(500 ft)	6.15	2.45	0.00
180m(600 ft)	5.31	1.21	0.00
210m(700 ft)	4.14	1.76	0.01
240m(800 ft)	7.77	3.02	0.00
270m(900 ft)	4.86	1.58	0.06
305+m(1,000+ ft)	8.25	1.25	0.00
Total	6.35	2.12	0.01

(1) Per million vehicles through the unit.

TABLE 27

ACCIDENT, INJURY, AND FATALITY RATES AT RAMP TERMINAL AREAS[28]

Type of Unit	Accident Rate (1)	Injury Rate (1)	Fatality Rate (1)
Nearest Access Point: Cloverleaf			
Ramp-Crossroad	2.28	0.79	0.01
Ramp-Frontage Road	2.89	1.43	0.00
Frontage-Crossroad	3.26	0.99	0.00
Nearest Access Point: Diamond			
Ramp-Crossroad	4.06	1.20	0.02
Ramp-Frontage Road	2.00	0.84	0.00
Frontage-Crossroad	3.19	1.05	0.00
Nearest Access Point: Direct/Semi-Direct Connection			
Ramp-Crossroad	5.35	1.42	0.05
Ramp-Frontage Road	1.64	0.29	0.00
Frontage-Crossroad	15.98	2.86	0.00

(1) Per million vehicles through the unit.

APPENDIX E

EXAMPLES OF TRAFFIC OPERATIONAL AND SAFETY ANALYSES

Four examples have been developed to illustrate the recommended procedures for traffic operational and safety analyses and comparison of alternative improvements presented in Sections VII and VIII of this volume. These examples illustrate four common problems that occur at freeway-arterial interchanges and appropriate solutions for each. The problems presented here are:

1. Run-off-road accidents on a short-radius curve of an off-ramp.

2. Traffic backup onto the mainline freeway from a diamond off-ramp.

3. Operational problems and accidents associated with a frontage road intersection on the arterial crossroad located very close to a ramp terminal.

4. Deficient capacity for left-turns from the arterial crossroad.

The examples are presented in order of increasing complexity of the analyses that were performed. The first two examples present relatively simple problems where a single alternative solution is considered. The third and fourth examples are cases with more than one alternative solution. The third example illustrates a situation where two alternatives must be considered both seperately and in combination; the final example illustrates a situation where the alternatives are mutually exclusive and do not have to be considered together.

These examples are based on actual interchanges for which data were collected during the study. Actual interchanges were used to assure that the problems chosen as examples were realistic. Some changes in the basic traffic volume and accident data have been made to increase the illustrative value of the examples. The results presented here should not be considered to be a general evaluation of any particular countermeasure, because the cost-effectiveness of countermeasures are highly dependent on the traffic volumes and accident experience of the specific location under consideration.

EXAMPLE NO. 1

Background

Example No. 1 concerns a full cloverleaf interchange in the suburban portion of a major metropolitan area. The mainline freeway has 6 lanes, the arterial crossroad has four lanes and all ramps have one lane. The overall configuration of the interchange is illustrated in Figure 25.

The mainline freeway has an Average Daily Traffic (ADT) of 80,000 vehicles per day west of the interchange and 75,000 vehicles per day east of the interchange; the arterial crossroad has an ADT of 20,000 vehicles north of the interchange and 30,000 vehicles south of the interchange. The eastbound outer connection off-ramp has an ADT of 7,800 vehicles.

Operational and Safety Problems

The State highway agency has found a concentration of run-off-road accidents on the eastbound outer connection off-ramp. There is an average of 5 single vehicle, run-off-road accidents per year (including an annual average of 0.3 fatal, 1.7 injury and 3.0 property-damage-only accidents) on the outside of the first curve beyond the off-ramp gore. This curve has a radius of 76.2 m (250 ft) and the concentration of accidents at this location is attributed to the short radius of curvature and low superelevation of this curve.

Alternative Considered

The State highway agency is considering rebuilding the off-ramp curve and increasing its radius to 167.6 m (550 ft) as a countermeasure for the run-off-road accidents. Reconstruction of the curve would cost $250,000. The original off-ramp curve and proposed improvement are illustrated in the lower portion of Figure 25. The decision to rebuild the off-ramp curve should be based on an economic analysis of the anticipated operational and safety benefits.

Quantify Operational Effects

Although the proposed project is intended primarily to reduce accidents, some reduction in vehicle running costs is expected to result from increased radius of curvature of the ramp. An operational analysis was conducted using the procedure recommended in Section VII-A of this volume. The analysis limits included the mainline freeway adjacent to the off-ramp terminal and the off-ramp from the gore to the point of

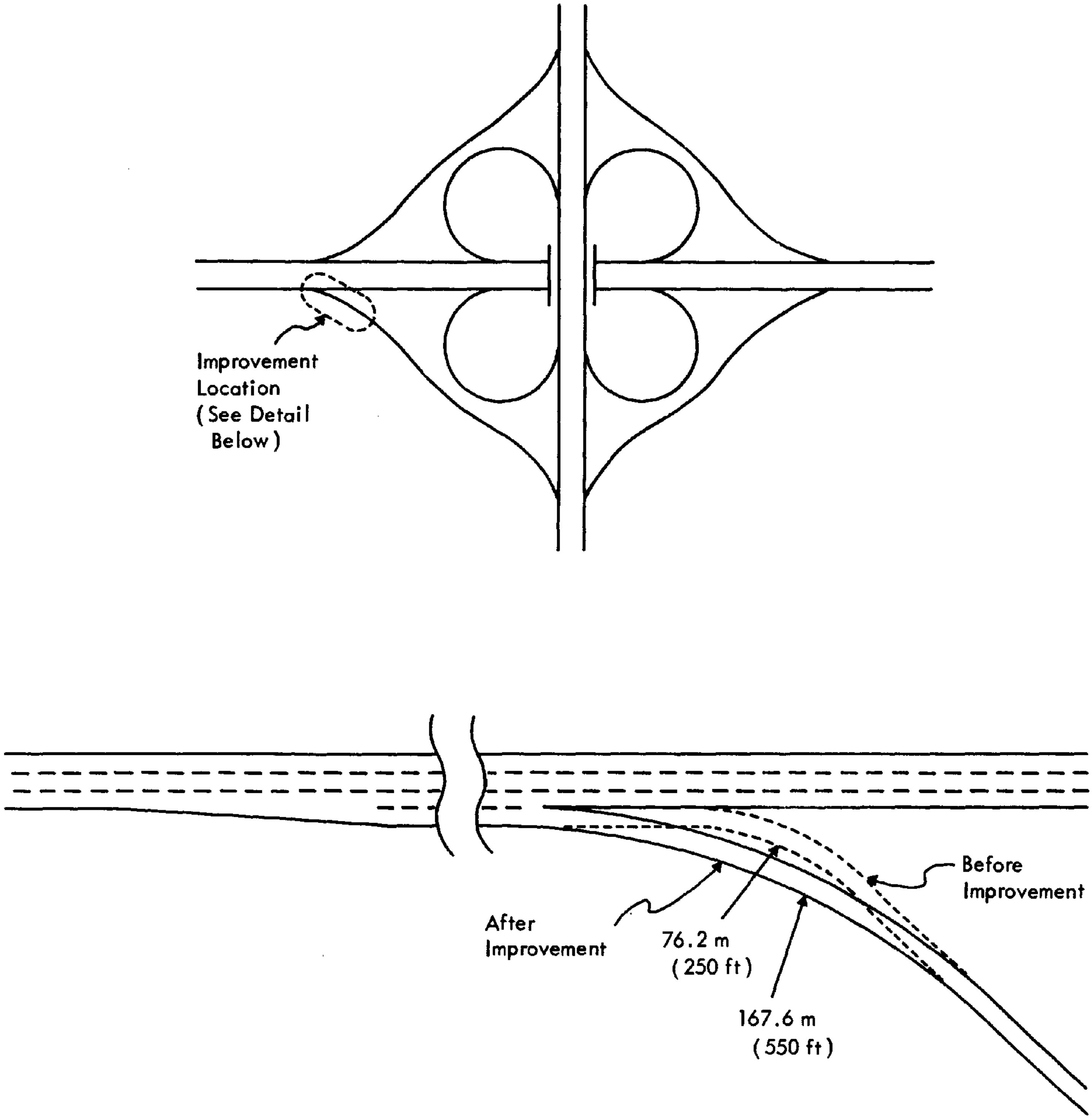


Figure 25 - Interchange Plan and Improvement Diagram

tangency (PT) of the reconstructed off-ramp curve. The analysis of the original configuration involved three roadway sections (one on the mainline freeway and two on the ramp) and one transition point; the analysis of the improved conditions required two roadway sections (one on the mainline freeway and one on the ramp) and one transition point. These analysis units, which include all portions of the interchange directly affected by the proposed improvement are illustrated in Figure 26.

The analysis considered traffic operational conditions during four periods of the day, whose traffic volumes were estimated as follows:

Period	Duration (hours)	Traffic Volume (vph) EB Freeway west of Interchange	EB to SB off-ramp
Morning Peak	2	5000	1000
Evening Peak	2	3000	600
Off-Peak (daytime)	12	1660	330
Off-Peak (nighttime)	8	400	80

The assumed traffic mix was 98 percent passenger cars, 1 percent single-unit trucks and 1 percent combination trucks. The effects of trucks were accounted for using the approximate procedure suggested in Step 6 of Section VII-A.

Table 28 presents the results of the travel time analysis. The proposed project will result in a slight increase in travel time at this location. The travel time savings resulting from a decrease in the length of the ramp are offset by an increase in the travel distance on the freeway resulting from the relocation of the off-ramp gore 35 m (115 ft) downstream. The overall increase in travel time is estimated to be 770 hours per year or about one second per vehicle.

Table 29 illustrates that, despite the slight increase in travel time, the proposed project will decrease running costs. The annual decrease in running costs is expected to be $15,140 or $0.005 per vehicle.

Quantify Safety Effects

Based on similar projects at other locations, the highway agency expects that 75 percent of the run-off-road accidents on the off-ramp curve will be eliminated by the improvement. The safety benefits can be determined on the basis of the 1975 NHTSA accident costs presented in Section VIII, increased by 49 percent to account for cost increases

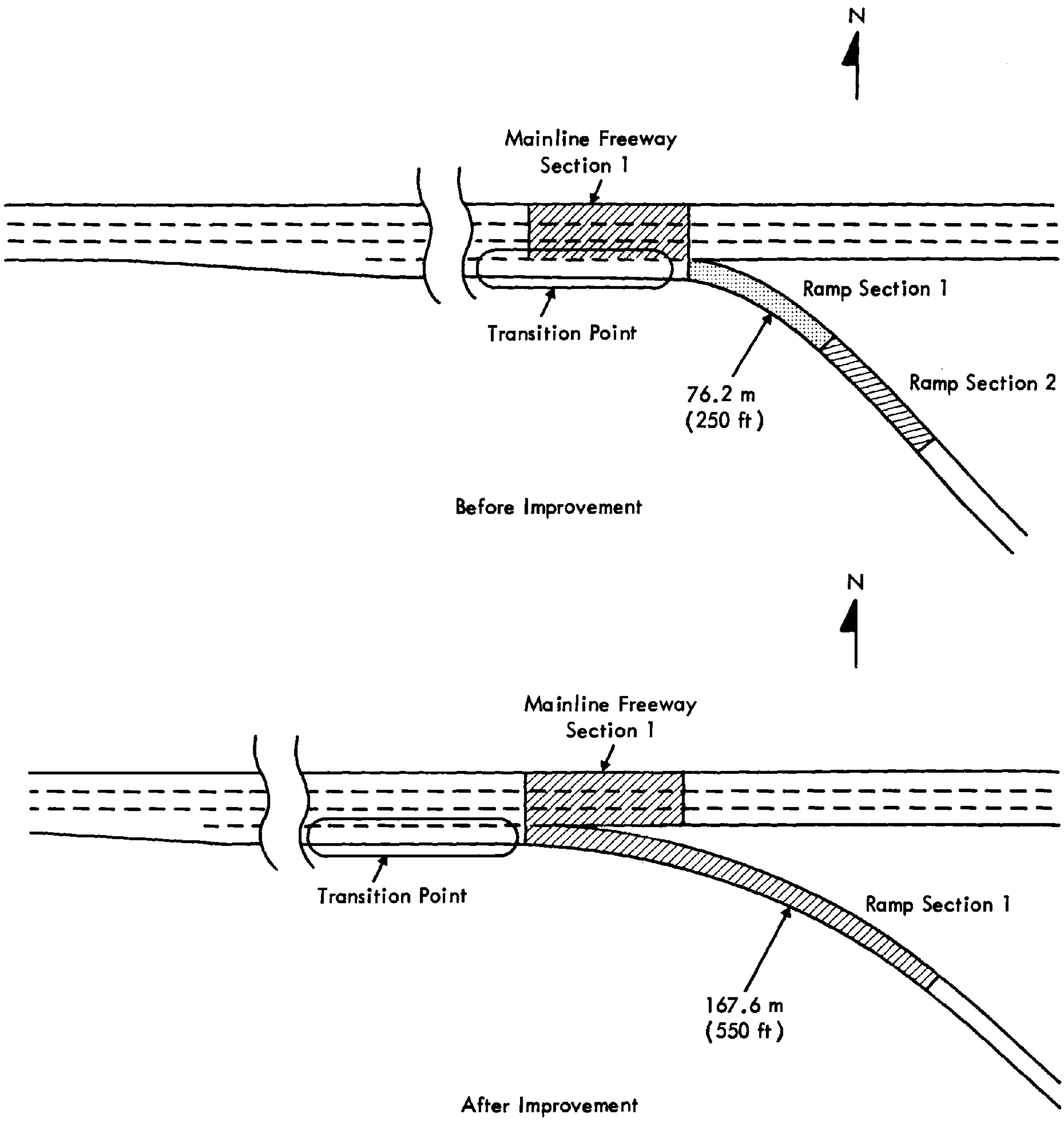


Figure 26 - Operational Analysis Units for Ramp Curve Reconstruction Project

TABLE 28

TRAVEL TIME ESTIMATES

Alternative	Daily Total Travel Time (hours) Mainline Freeway	Ramp	Total	Yearly Total Travel Time (hours)
Do-Nothing	12.99	21.65	34.64	12,640
Reconstruct Curve	16.59	20.16	36.75	13,410

TABLE 29

RUNNING COST ESTIMATES

Alternative	Daily Total Running Cost (dollars) Mainline Freeway	Ramp	Transition Point	Total	Yearly Total Running Cost (dollars)
Do-Nothing	65.82	79.15	87.55	232.52	84,870
Reconstruct Curve	81.66	58.04	51.33	191.03	69,730

since 1975. Table 30 illustrates that the proposed project is expected to reduce 0.225 fatal, 1.275 injury and 2.25 property-damage-only accidents per year, for an annual accident cost savings of $146,700.

Net Return Analysis

A net return comparison of the ramp widening and do-nothing alternatives is shown in Table 31. The annual costs of travel time, vehicle operation and accidents are those derived in Tables 28, 29 and 30. The cost of travel time is estimated as $4.50 per vehicle-hour (i.e., $3.00 per vehicle-hour recommended by the AASHTO manual[2] increased by 49 percent to account for inflation). The $250,000 construction cost has been annualized at an 8 percent interest rate over an anticipated service life of 15 years. Table 31 shows that the reduced accident costs and vehicle running costs outweigh the increased travel time cost and the construction cost to produce an expected annual net return of $132,875 for the project. The project is economically justified, since its net return is greater than zero.

Conclusion

The proposed project will result in a substantial decrease in accident costs, a moderate decrease in vehicle running costs and a slight increase in travel time. The project is economically justified and its construction is recommended if sufficient funds are available.

TABLE 30

ESTIMATED SAFETY BENEFITS

Severity Level	Average Annual Accident Frequency	Anticipated Percent Accident Reduction	Number of Accidents Reduced	Accident Cost (dollars)	Cost of Accidents Reduced (dollars)
Fatal	0.3	75%	0.225	$525,000	$118,125
Injury	1.7	75%	1.275	18,000	22,950
Property-Damage-Only	3.0	75%	2.25	2,500	5,625
					$146,700

TABLE 31

NET RETURN ANALYSIS

Alternative	Annual Travel Time Cost	Annual Vehicle Running Cost	Annual Accident Cost Savings	Other Annual Costs	Annualized Construction Cost	Net Return
Do-Nothing	$56,880	$84,870	-	0	0	
Reconstruct Curve	60,345	69,730	146,700	0	25,500	
Difference	-3,465	15,140	146,700	0	-25,500	$132,875

EXAMPLE NO. 2

Background

Example No. 2 concerns a full diamond interchange located in the suburban portion of a medium-sized metropolitan area. The mainline freeway has 6 lanes through the interchange area; the arterial crossroad has 4 lanes, with left-turn lanes at the ramp terminals in both directions of travel. All four diamond ramps are single-lane ramps 5.5 m (18 ft) wide. The interchange configuration is illustrated in the upper portion of Figure 27.

The mainline freeway has an Average Daily Traffic (ADT) of 45,000 vehicles south of the interchange and 29,000 north of the interchange; the arterial crossroad has an ADT of 22,000 east of the interchange and 19,000 west of the interchange.

Operational and Safety Problems

The State highway agency has identified a major operational problem at this interchange in the evening peak period resulting from the backup of traffic from the northbound off-ramp onto the mainline freeway. The evening peak period volume on the northbound off-ramp is 885 vph (270 vph turning left and 615 vph turning right). The turning movements at the east ramp terminal in the morning and evening peak periods are illustrated in Figure 28. The northbound off-ramp volume exceeds the capacity of the northbound approach to the crossroad ramp terminal, forcing right-turning vehicles to use the right shoulder as a turning lane. Despite the storage of vehicles on the shoulder, vehicle queues frequently back onto the mainline freeway in the evening peak period, resulting in an average of 15 accidents per year (no fatalities, 5 injury accidents and 10 property-damage-only accidents).

Alternatives Considered

The State highway agency is considering the possibility of widening the northbound off-ramp to increase the capacity of the ramp terminal and increase the vehicle storage available on the ramp. The widened ramp would have 3-3.7 m (12 ft) lanes--two for right-turns and one for left-turns. The original and proposed configurations are illustrated in the lower portion of Figure 27. Improved signal hardware would be installed on the crossroad to accommodate the improved geometrics. The cost of the project would be $350,000 and it is anticipated to reduce delays and vehicle running costs on the ramp, eliminate vehicle queues from the mainline freeway and reduce the frequency of rear-end accidents. The decision to implement this project should be based on an economic analysis of the expected costs and benefits.

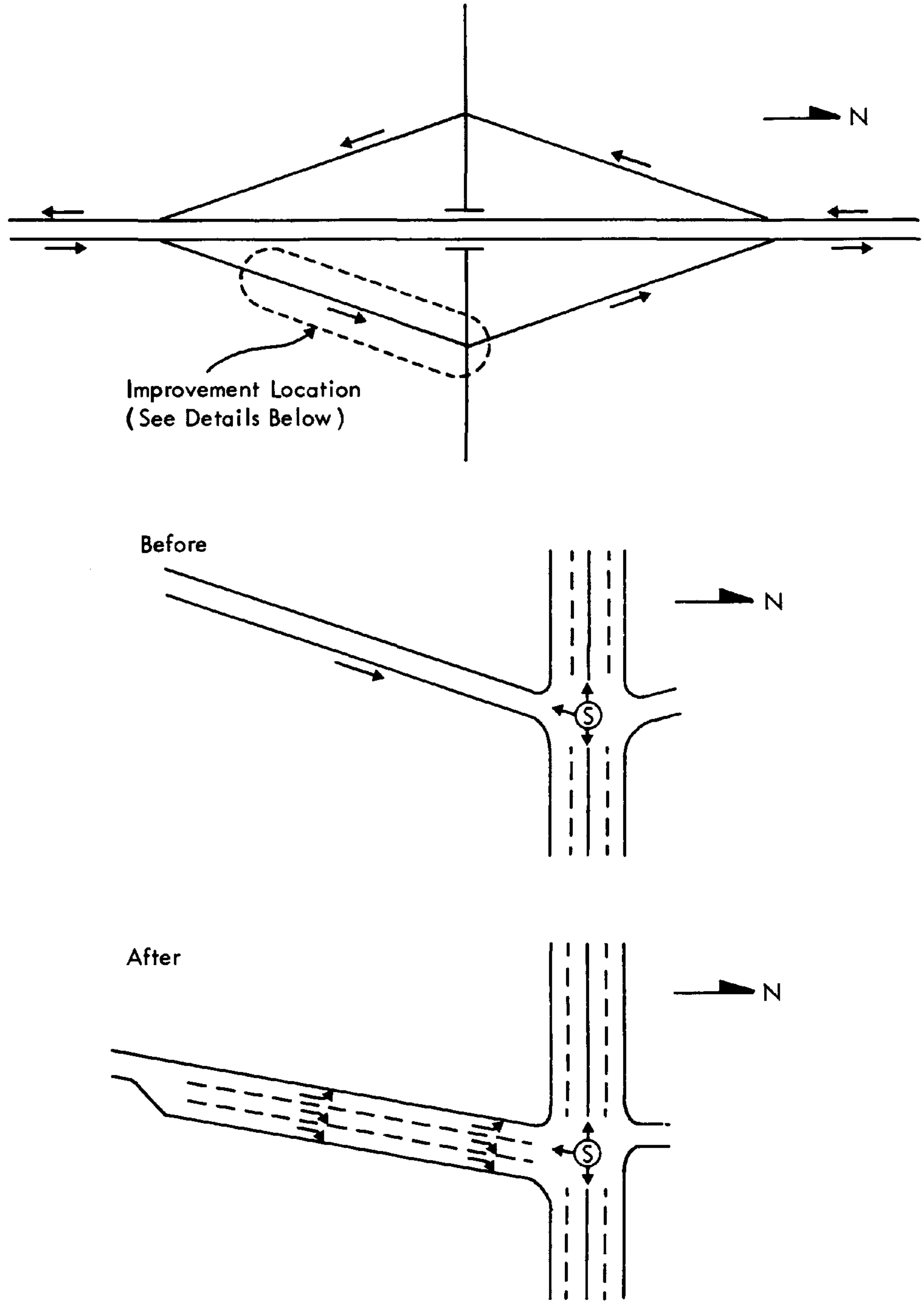


Figure 27 - Interchange Plan and Improvement Diagram

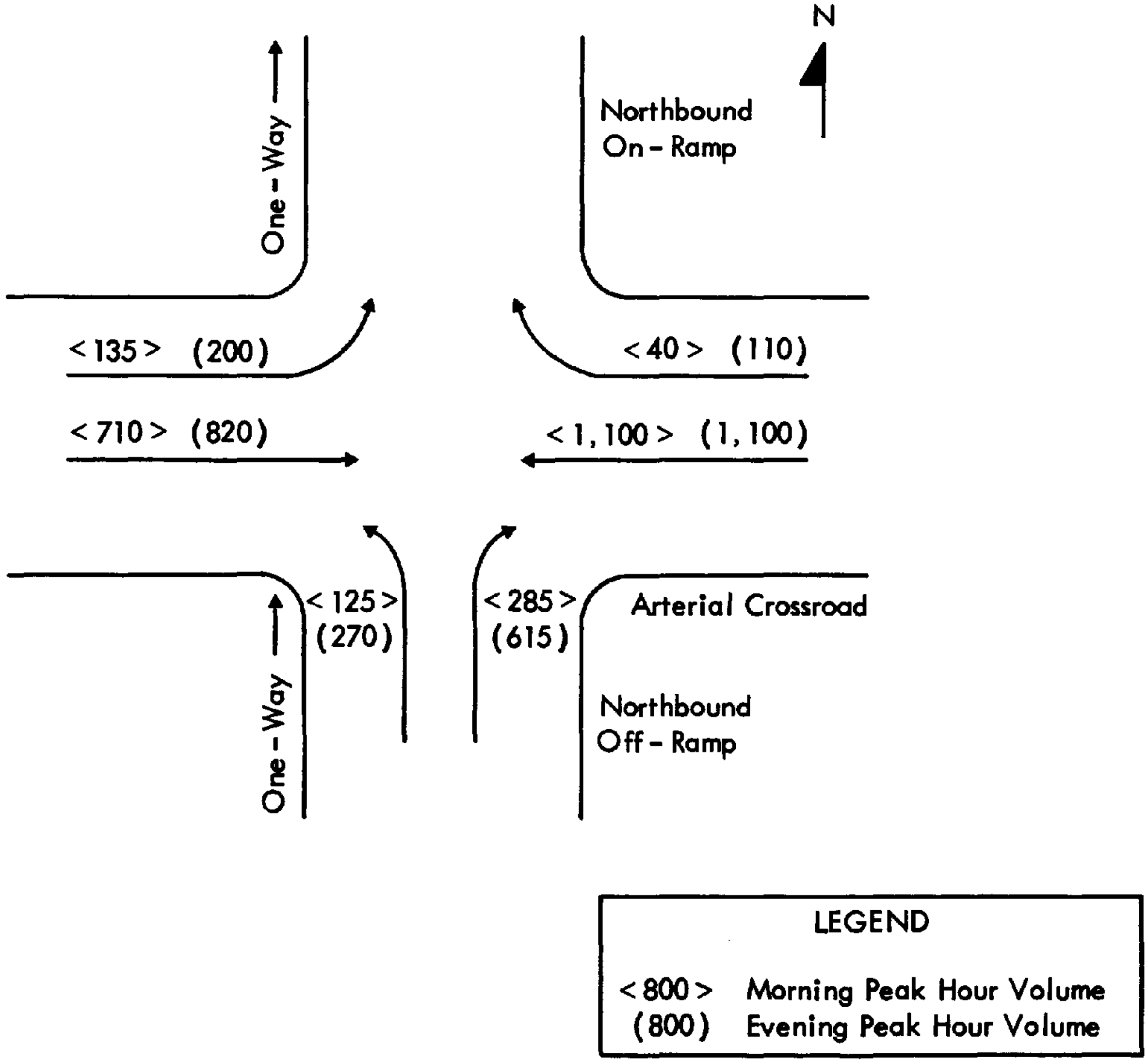


Figure 28 - Peak Hour Traffic Volumes at East Ramp Terminal

Quantify Operational Effects

The operational effects of the improvement alternatives were analyzed using the procedures recommended in Section VII-A. For comparison of the original and improved conditions, the interchange was divided into 4 analysis units that were expected to be operationally affected by the improvement; the four analysis units were 2 roadway sections that together comprise the northbound off-ramp, one intersection (the east ramp terminal) and one transition point (the transition from the mainline freeway to the northbound off-ramp). Operational effects on the mainline freeway were not considered.

Each alternative was considered for four periods of the day whose traffic volumes were defined as:

Period	Duration (hours/day)	Traffic Volumes
AM Peak	2	as defined in Figure 28
PM Peak	2	as defined in Figure 28
Off-Peak (Daytime)	12	42% of average peak period volume
Off-Peak (Nighttime)	8	12.5% of average peak period volume

The assumed traffic mix was 95 percent passenger cars, 3 percent single-unit trucks and 2 percent combination trucks. The effects of trucks were accounted for using the approximate procedure suggested in Step 6 of Section VII-A.

The results of the travel time analysis are shown in Table 32. The daily total travel time shown there is the sum of the travel time for the roadway section and the additional travel time due to the presence of the intersection. The yearly total travel time is based on 250 working days per year. The analysis shows that the ramp widening project would result in a substantial reduction in travel time of 26,000 vehicle-hours per year or 7.5 seconds per vehicle.

The results of the running cost analysis are presented in Table 33 in a similar manner. The ramp widening project was found to decrease vehicle running costs by $86,270 per year or $0.007 per vehicle.

If only the evening peak period is considered, the yearly travel time savings would be 17,600 hours or 28 seconds per vehicle and the running cost savings would be $17,700 or $0.007 per vehicle. Thus, the delay reduction benefits appear to be concentrated in the evening peak period, while the vehicle operating cost reductions appear to be more equally spread throughout the day.

TABLE 32

TRAVEL TIME ESTIMATES

Alternative	Daily Total Travel Time (hours) Roadway Sections (Ramp)	Intersection	Total	Yearly Total Travel Time (hours)
Do-Nothing	36.47	329.19	365.66	91,415
Widen Ramp	12.63	248.99	261.62	65,405

TABLE 33

RUNNING COST ESTIMATES

Alternative	Daily Total Running Cost (Dollars) Roadway Sections (Ramp)	Intersection	Transition Point	Total	Yearly Total Running Cost (Dollars)
Do-Nothing	209.23	750.41	48.26	1007.90	252,000
Widen Ramp	71.87	543.69	47.37	662.93	165,730

Quantify Safety Effects

Table 34 illustrates the computation of the safety benefits for the ramp widening project. It is assumed that the project will eliminate all rear-end accidents on the mainline freeway, although some rear-end accidents may still occur on the off-ramp itself. Overall, a 75 percent reduction in rear-end accident experience is anticipated. The accident costs used in Table 34 are those presented in Section VIII, increased by 49 percent to account for cost increases since 1975.

The results presented in Table 34 show that the ramp widening project is expected to prevent 3.75 injury accidents and 7.5 property-damage-only accidents per year, for a total annual accident cost savings of $86,250.

Net Return Analysis

A net return comparison of the ramp widening and do-nothing alternatives is shown in Table 35. The annual costs of travel time, vehicle operation and accidents are those derived in Tables 32, 33, and 34. The $350,000 construction cost has been annualized at an 8 percent interest rate over an anticipated service life of 15 years. The analysis results indicate that the ramp widening project is economically justified and has an annualized net return of $247,835.

Conclusion

Widening of the northbound off-ramp will reduce travel time, reduce vehicle running costs and reduce accidents. Construction of the ramp widening project is economically justified from a traffic operations and safety viewpoint and is recommended if sufficient funds are available.

TABLE 34

ESTIMATED SAFETY BENEFITS

Severity Level	Average Annual Accident Frequency	Anticipated Percent Accident Reduction	Number of Accidents Reduced	Accident Cost (dollars)	Cost of Accidents Reduced (dollars)
Fatal	0	75%	0	$525,000	0
Injury	5.0	75%	3.75	18,000	$67,500
Property-Damage -Only	10.0	75%	7.50	2,500	18,750
					$86,250

TABLE 35

NET RETURN ANALYSIS

Alternative	Annual Travel Time Cost	Annual Vehicle Running Cost	Annual Accident Cost Savings	Other Annual Costs	Annualized Construction Cost	Net Return
Do-Nothing	$408,625	$252,000		0	0	
Ramp Widening	292,360	165,730	$ 86,250	0	$ 40,950	
Difference	$116,265	$ 86,270	$ 86,250	0	$-40,950	$247,835

EXAMPLE NO. 3

Background

Example No. 3 concerns a full diamond interchange located at the edge of a small urban community. The mainline freeway is a 4-lane, divided, fully-access-controlled facility, while the arterial crossroad is a 4-lane, divided facility without access control. All four diamond ramps are single-lane ramps, 5.5 m (18 ft) wide. The interchange configuration is illustrated in Figure 29.

The land surrounding the interchange is commerically developed with highway-related businesses and the arterial crossroad has strip commercial development to the east of the interchange toward the Central Business District (CBD) of the community. A frontage road, which is also commercially developed, intersects the crossroad approximately 30 m (100 ft) east of the east ramp terminal. Both the east ramp terminal and the frontage road intersection are signalized.

The mainline freeway has an Average Daily Traffic (ADT) of 25,000 vehicles both north and south of the interchange; the arterial crossroad has an ADT of 7,000 vehicles east of the interchange and 6,500 vehicles west of the interchange. The morning and evening peak hour volumes at the east ramp terminal and frontage road are given in Figure 30.

Operational and Safety Problems

The State highway agency has identified two safety problems at this interchange. First, the close proximity of the east ramp terminal and the frontage road intersection has been found to confuse drivers. Erratic maneuvers including vehicles backing up on the northbound on-ramp and vehicles making U-turns across the freeway median have been observed. These erratic maneuvers are apparently related to vehicles which intend to enter the frontage road and turn onto the northbound on-ramp by mistake. An accident history related to these erratic maneuvers has been documented.

Second, the interchange has been found to have high nighttime accident experience. An average of 6 night accidents per year occur in the vicinity of the east ramp terminal and frontage road intersection, including 2 nighttime pedestrian fatalities within the past 30 months.

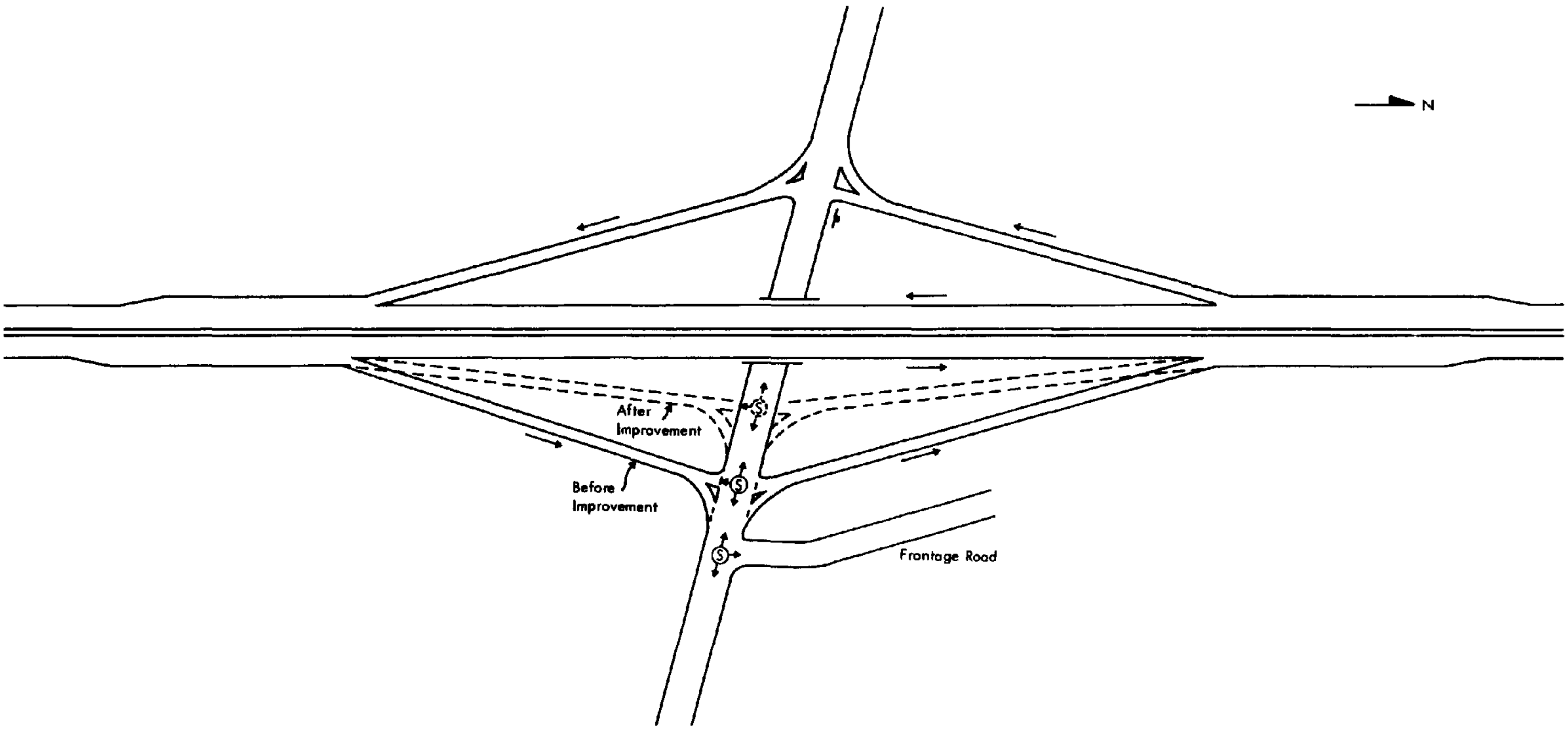


Figure 29 - Interchange Plan and Improvement Diagram

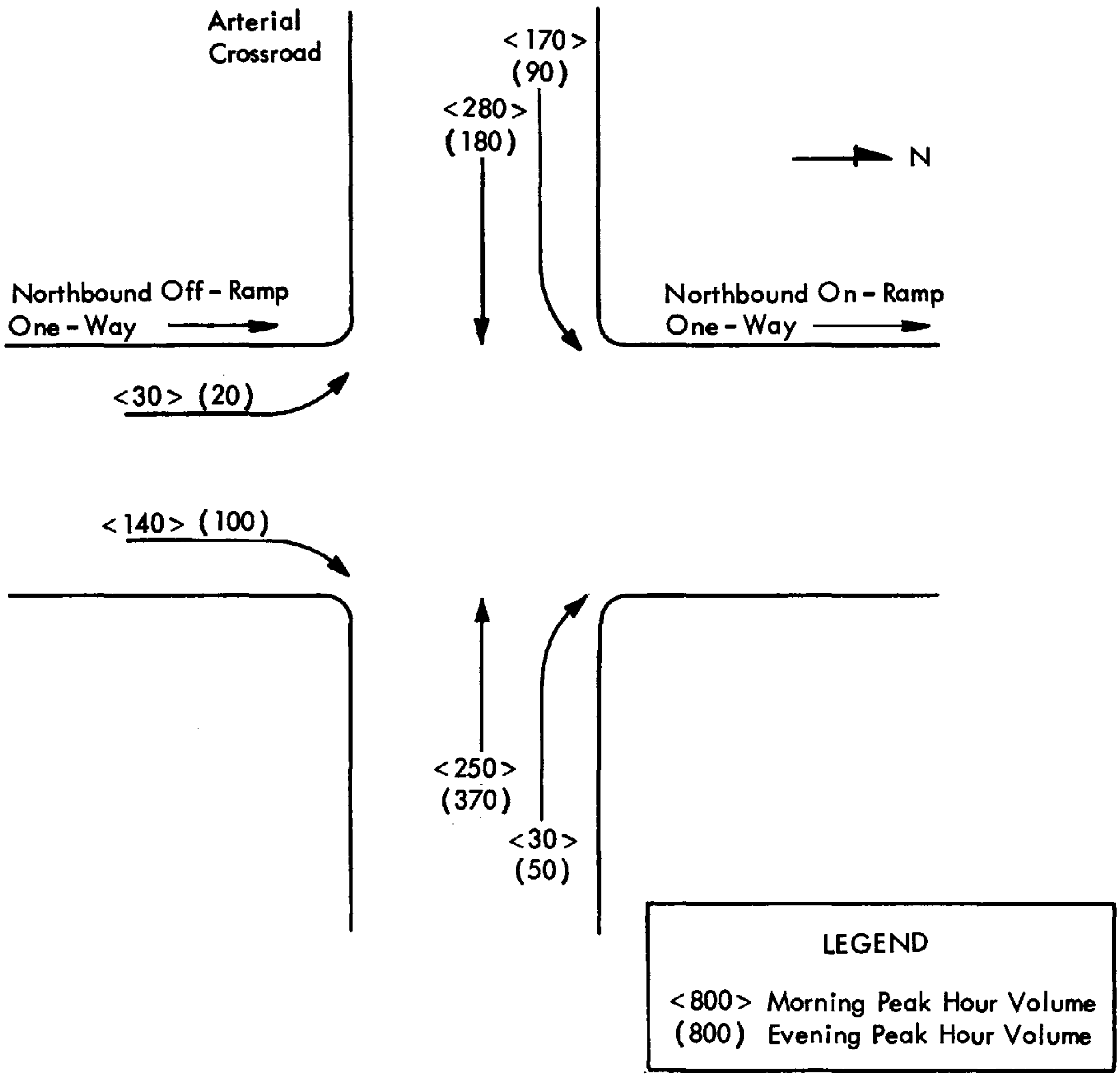


Figure 30 - Peak Hour Traffic Volumes at East Ramp Terminal

Alternatives Considered

The State highway agency is considering two possible improvements that may be implemented seperately or in conjunction with one another. The first proposed countermeasure, intended to reduce driver confusion, is to increase the separation between the east ramp terminal and the frontage road intersection from 30 m (100 ft) to 60 m (200 ft). This will be accomplished by reconstructing the northbound on- and off-ramps to relocate the east ramp terminal closer to the freeway overpass structure. The cost of the ramp reconstruction alternative is estimated to be \$500,000. This countermeasure is illustrated in Figure 29. The second countermeasure being considered is to install tower lighting to reduce the night accident experience. The cost of the lighting alternative is estimated to be \$100,000. The decision to implement either or both of these countermeasures should be based on an economic analysis of the expected safety benefits. Consideration should also be given to any operational effects of the ramp reconstruction alternative.

Quantify Operational Effects

The operational effects of the ramp reconstruction alternative were analyzed using the procedures recommended in Section VII-A. For comparison of the original and improved configurations, the operational analysis limits were selected to include the northbound on- and off-ramps and the arterial crossroad from the new ramp terminal location to the frontage road intersection. This portion of the interchange was divided into 7 analysis units (one roadway section on each ramp, 4 roadway sections on the arterial and one intersection, i.e. the east ramp terminal).

Both the do-nothing and ramp reconstruction alternatives were considered for four periods of the day, whose traffic volumes were defined as:

Period	Duration (hours/day)	Traffic Volume
AM Peak	2	As defined in Figure 30
PM Peak	2	As defined in Figure 30
Off-peak (Daytime)	12	42% of average peak period volume.
Off-peak (nighttime)	8	12.5% of average peak period volume.

The assumed traffic mix was 96% passenger cars, 2% single-unit trucks and 2% combination trucks. The effects of trucks were accounted for using the approximate procedure suggested in Step 6 of Section VIII-A.

Tables 36 and 37 present the effects of the ramp reconstruction alternative on travel time and vehicle running cost, respectively. This alternative is expected to increase travel time by 29 vehicle-hours per year and decrease vehicle running costs by $29 per year. These effects are both of trivial magnitude and are partially offsetting.

Quantify Safety Effects

The average accident experience for the northbound on- and off-ramps, the east ramp terminal and the frontage road intersection is presented in Table 38. An average of 14.7 accidents per year, including 1.2 fatal accidents, occur in this portion of the interchange. Approximately 41% of the total accidents and 67% of the fatal accidents occur at night.

Table 39 illustrates the computation of safety benefits for the three alternatives: ramp reconstruction, lighting, and ramp reconstruction and lighting combined. The ramp reconstruction alternative is assumed to reduce 10% of all accidents at the interchange, while the lighting alternative is assumed to reduce 50% of the nighttime accidents. When these countermeasures are combined, the anticipated effectiveness is a 10% reduction of daytime accidents and a 55% reduction of nighttime accidents. The accident costs used in the analysis are those present in Section VIII, increased by 49% to account for cost increases since 1975.

The results presented in Table 39 show that the ramp reconstruction alternative would result in $74,900, the lighting alternative in $229,700, and the ramp reconstruction and lighting alternatives combined in $281,600 of safety benefits.

Evaluate Alternatives and Select the Best

The three alternatives under consideration were compared with the do-nothing alternative through a net return analysis. The results of this analysis are shown in Table 40. The annual costs of time, vehicle operation and accidents are those derived in Tables 36, 37, and 39. In accordance with Section VIII, the $3.00 per vehicle-hour cost of travel time has been increased by 49% to allow for cost increases since 1975. The construction cost for each alternative has been annualized at an 8% interest rate over the anticipated 20-year service life of the project.

TABLE 36

TRAVEL TIME ESTIMATES

Alternative	Daily Total Travel Time (hours) Ramps	Arterial	East Ramp Terminal	Total	Yearly Total Travel Time (hours)
Do-nothing	17.16	7.46	81.25	105.87	38,643
Ramp Reconstruction	17.12	7.58	81.25	105.95	38,672

TABLE 37

RUNNING COST ESTIMATES

Alternative	Daily Total Running Cost (dollars) Ramps	Arterial	East Ramp Terminal	Total	Yearly Total Running Cost (dollars)
Do-nothing	77.58	33.52	184.39	295.49	107,854
Ramp Reconstruction	77.16	33.86	184.39	295.41	107,825

TABLE 38

ANNUAL ACCIDENT EXPERIENCE FOR NORTHBOUND RAMPS, EAST RAMP TERMINAL AND FRONTAGE ROAD INTERSECTION

Severity Level	Daytime Accident Frequency	Nighttime Accident Frequency	Total Accident Frequency
Fatal	0.4	0.8	1.2
Injury	3.8	1.7	5.5
Property-Damage-Only	4.5	3.5	8.0
Combined	8.7	6.0	14.7

TABLE 39

ESTIMATED SAFETY BENEFITS

Alternative	Severity Level	Time of Day	Average Annual Accident Frequency	Anticipated Percent Accident Reduction	Number of Accidents Reduced	Accident Cost (dollars)	Cost of Accidents Reduced (dollars)
Ramp Reconstruction	Fatal	Combined	1.2	-	0.12	525,000	63,000
		Day	0.4	10%	0.04		
		Night	0.8	10%	0.08		
	Injury	Combined	5.5	-	0.55	18,000	9,900
		Day	3.8	10%	0.38		
		Night	1.7	10%	0.17		
	Property-Damage-Only	Combined	8.0	-	0.80	2,500	2,000
		Day	4.5	10%	0.45		
		Night	3.5	10%	0.35		
							74,900
Lighting	Fatal	Day	0.4	0%	-	-	-
		Night	0.8	50%	0.40	525,000	210,000
	Injury	Day	3.8	0%	-	-	-
		Night	1.7	50%	0.85	18,000	15,300
	Property-Damage-Only	Day	4.5	0%	-	-	-
		Night	3.5	50%	1.75	2,500	4,400
							229,700
Ramp Reconstruction and Lighting	Fatal	Combined	1.2	-	0.48	525,000	252,000
		Day	0.4	10%	0.04		
		Night	0.8	55%	0.44		
	Injury	Combined	5.5	-	1.315	18,000	23,700
		Day	3.8	10%	0.38		
		Night	1.7	55%	0.935		
	Property-Damage-Only	Combined	8.0	-	2.375	2,500	5,900
		Day	4.5	10%	0.45		
		Night	3.5	55%	1.925		
							281,600

TABLE 40

NET RETURN ANALYSIS

Alternative	Annual Travel Time Cost	Annual Vehicle Running Cost	Annual Accident Cost Savings	Other Annual Costs	Annualized Construction Cost	Net Return
DO-NOTHING	173,893	107,854	-	0	0	
RAMP RECONSTRUCTION	174,024	107,825	74,900	0	51,000	
Difference	-131	29	74,900	0	-51,000	23,798
DO-NOTHING	173,893	107,854	-	0	0	
LIGHTING	173,893	107,854	229,700	0	10,200	
Difference	0	0	229,700	0	-10,200	219,500
DO-NOTHING	173,893	107,854	-	0	0	
RAMP RECONSTRUCTION AND LIGHTING	174,024	107,825	281,600	0	61,200	
Difference	-131	29	281,600	0	-61,200	220,298

The analysis results show that all three alternatives have net return greater than zero and are economically justified. The alternatives in order of increasing net return are: ramp reconstruction ($23,798); lighting ($219,500); and, ramp reconstruction and lighting combined ($220,298). These results demonstrate that ramp reconstruction alone is the least desireable alternative. Therefore, whatever is done, lighting should be part of the project. The net return for the combined alternative is slightly higher than for lighting alone, so the combination is recommended if sufficient funds are available. If $600,000 is not available to construct the combined project, then $100,000 should certainly be invested in the lighting project alone.

Conclusion

Implementation of the lighting project or the ramp reconstruction and lighting projects combined is recommend to reduce accidents at this location.

EXAMPLE NO. 4

Background

The interchange in Example No. 4 is a full diamond located on a circumferential freeway in the suburban portion of a metropolitan area with over 1,000,000 population. The arterial crossroad is a radial route and is commercially developed. Most of the existing commercial development is north of the interchange, although the area south of the interchange is expected to develop rapidly in the next 10 to 15 years.

The existing interchange configuration is illustrated in Figure 31. The mainline freeway has 6 lanes through the interchange area. The arterial is a 4-lane divided highway with 3.7m (12 ft) lanes and partial control of access in the vicinity of the ramp terminals. Frontage roads, with intermittant openings to the arterial, are provided for local access on both sides of the arterial both north and south of the interchange. Both off-ramps have 2 - 3.7m (12 ft) lanes, while both on-ramps have a single 5.5m (18 ft) lane.

The State highway agency has adopted a long-range plan to upgrade the arterial crossroad to 6 lanes by 1990 for a 8.1 km (5 mile) section including the interchange. It is assumed that this improvement will be made whether or not other improvements are made at the interchange.

The upper portion of Figure 32 illustrates the current ADT and peak volumes for the interchange area. These volumes are expected to grow by the year 2000 to those illustrated in the lower portion of Figure 32. The freeway east of the interchange has a current ADT of 45,000, a morning peak volume of 4,600 vph and an evening peak volume of 4,480 vph; west of the interchange, the freeway has an ADT of 33,900, a morning peak volume of 3,480 vph and an evening peak volume of 3,300 vph.

Identification of Operational and Safety Problems

The interchange was selected for further study by the State highway agency based on existing traffic operational problems in the evening peak hour. In the evening peak hour, the traffic waiting to make the southbound-to-eastbound left-turn at the south ramp terminal backs through the north ramp terminal and partially blocks the southbound through lanes. This blockage reduces the capacity for southbound through traffic, resulting in Level of Service F conditions and further backups. In addition, traffic volumes at this interchange are expected to more than double over the next 20 years. The State has projected that both ramp terminals will be extremely congested in the year 2000, even if the arterial crossroad is widened as planned.

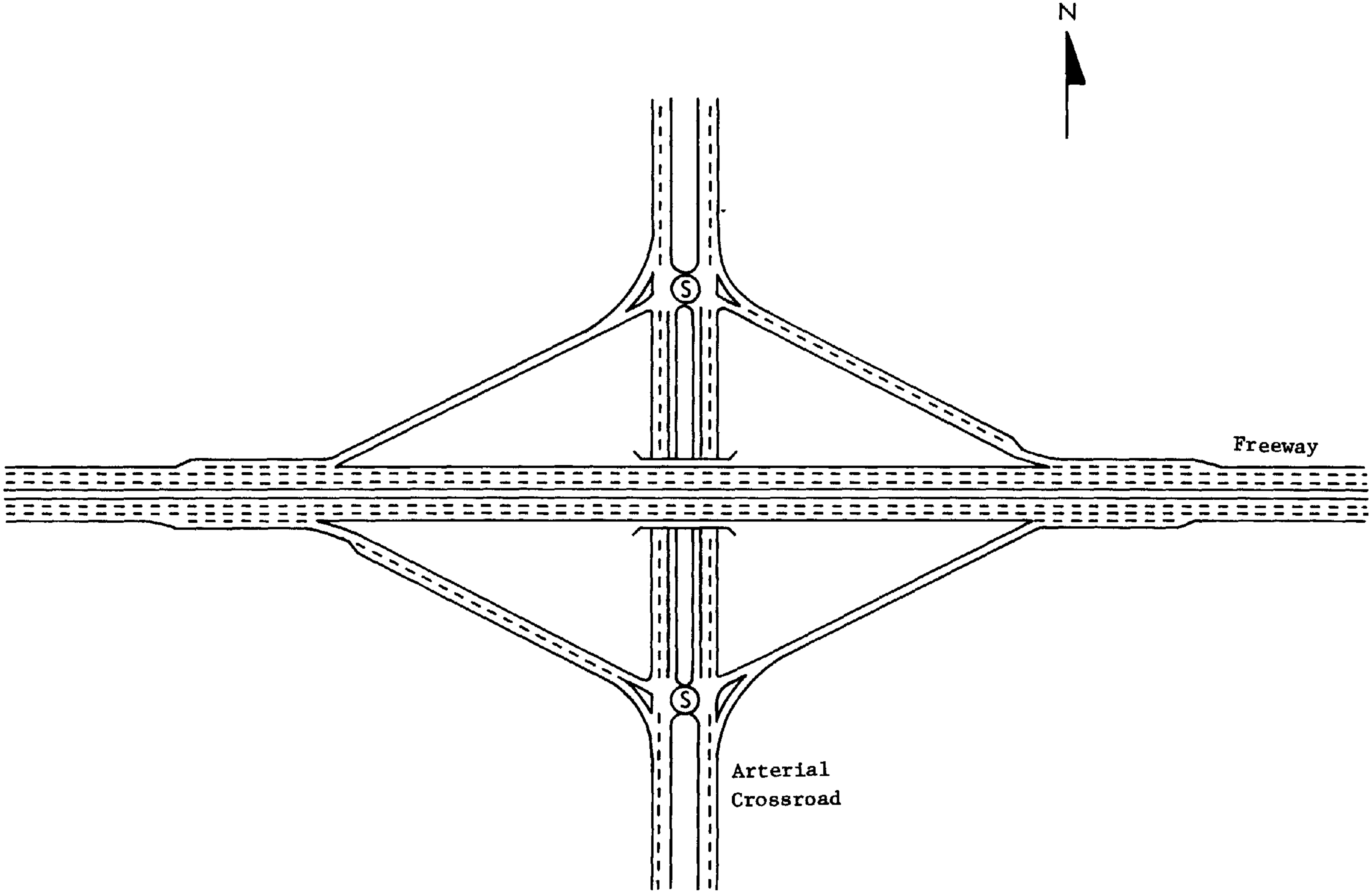


Figure 31 - Current Interchange Configuration

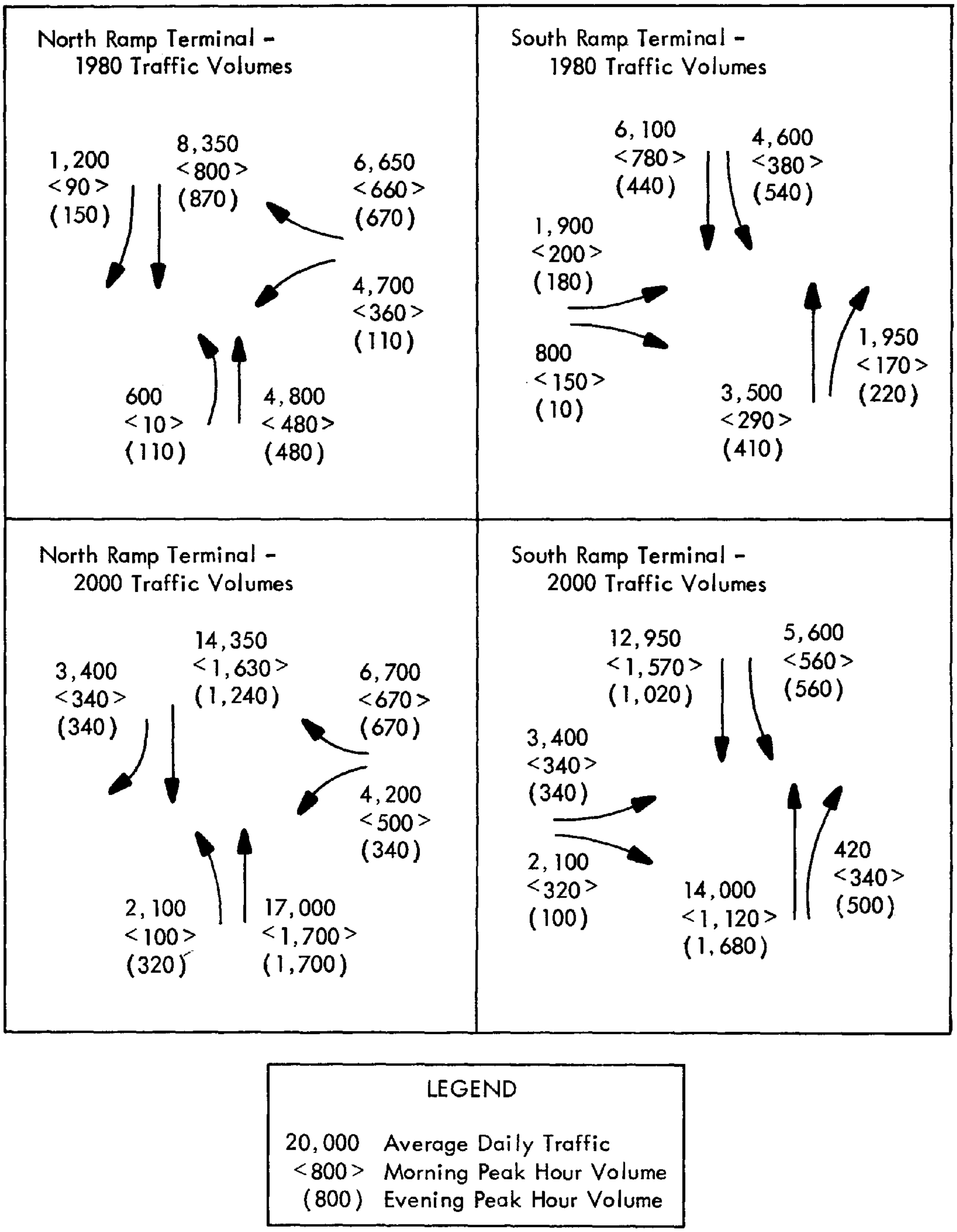


Figure 32 - Current and Proposed Traffic Volumes

The interchange was not found to be a high-accident location in the Statewide review of interchange safety conditions. There are, however, an annual average of 10 accidents at the north ramp terminal and 20 accidents at the south ramp terminal. Potential safety benefits from reduction of these accidents should be considered in the evaluation of operational improvements.

Alternatives Considered

The operational problems at the interchange were identified as: (1) excessive delay for through traffic on the arterial and (2) excessive delay for left-turns from the arterial. Operational analyses focused on the high-volume southbound-to-eastbound left-turn as the key to the solution of operational problems. The highway agency engineers consulted the chart presented in Table 6 of Section VI to determine appropriate improvements for diamond interchanges. The following potential improvements were selected from the chart.

- Add loop on-ramp
- Add directional on-ramp
- Optimize existing signal
- Add through lanes to arterial
- Add double left-turn lane to arterial
- Lengthen left-turn storage
- Increase distance between ramp terminals

Several of these alternatives were immediately eliminated from consideration. It was determined that the timing of the existing signal was already optimal for the existing geometrics and could not be improved further. The addition of through lanes to the arterial was not considered explicitly because a decision to widen the arterial had already been made. Also, it was found that, due to adjoining development, it was infeasible to increase the distance between the ramp terminals. This left four feasible alternatives, arranged below in order of increasing construction cost:

Alternative	Construction Cost
Lengthen left-turn storage	$50,000
Add double left-turn lane	$100,000
Add loop on-ramp	$700,000
Add directional on-ramp	$3,000,000

The do-nothing alternative was also considered.

For purposes of this example, the do-nothing, double left turn lane and loop on-ramp alternatives have been evaluated in detail. The configuration of the latter two alternatives is illustrated in Figures 33 and 34.

Quantify Operational Effects of Improvement Alternatives

The operational effects of the improvement alternatives were analyzed using the procedures recommended in Section VII-A of this volume. For the do-nothing and double left turn lane alternatives, the interchange was divided into 22 analysis units, as shown in Figure 35. These included 6 mainline freeway sections, 6 arterial crossroad sections, 4 ramp sections, 4 mainline ramp terminals (transition points), and 2 crossroad ramp terminals (intersections). Twenty-seven (27) analysis units were required for the loop ramp alternative because a new ramp and two new transition points were added, while two existing sections had to be subdivided.

Each alternative geometric configuration was considered for two analysis years (1980 and 2000) and four periods of the day. The traffic volumes for these periods of the day were defined as follows:

Period	Duration(hours/day)	Traffic Volumes
AM Peak	2	As shown in Figure 32
PM Peak	2	As shown in Figure 32
Off-Peak (Daytime)	12	42% of the average peak hour volume
Nighttime	8	12.5% of the average peak hour volume

The computations exactly followed the procedures of Steps 1 through 18, using the approximate adjustments for truck effects in Step 6, as recommended in the AASHTO Manual on User Benefit Analysis of Highway and Bus Transit Improvements-1977, so that separate computations for each vehicle type were not required.

All of the analyses, including that for the do-nothing alternative, assume that the arterial crossroad will be widened as planned.

The results of the travel time analysis are shown in Table 41. The daily total travel time for the interchange is the sum of the travel time for roadway sections and the additional travel time due to the two at-grade intersections. The yearly total travel time is based on 250 working days per year. The analysis shows that the double left-turn lane and loop on-ramp alternatives result in a nearly identical reduction in delay from the do-nothing condition. The anticipated reduction in travel time is equivalent to a savings of approximately 6 seconds per vehicle over the 20 year analysis period.

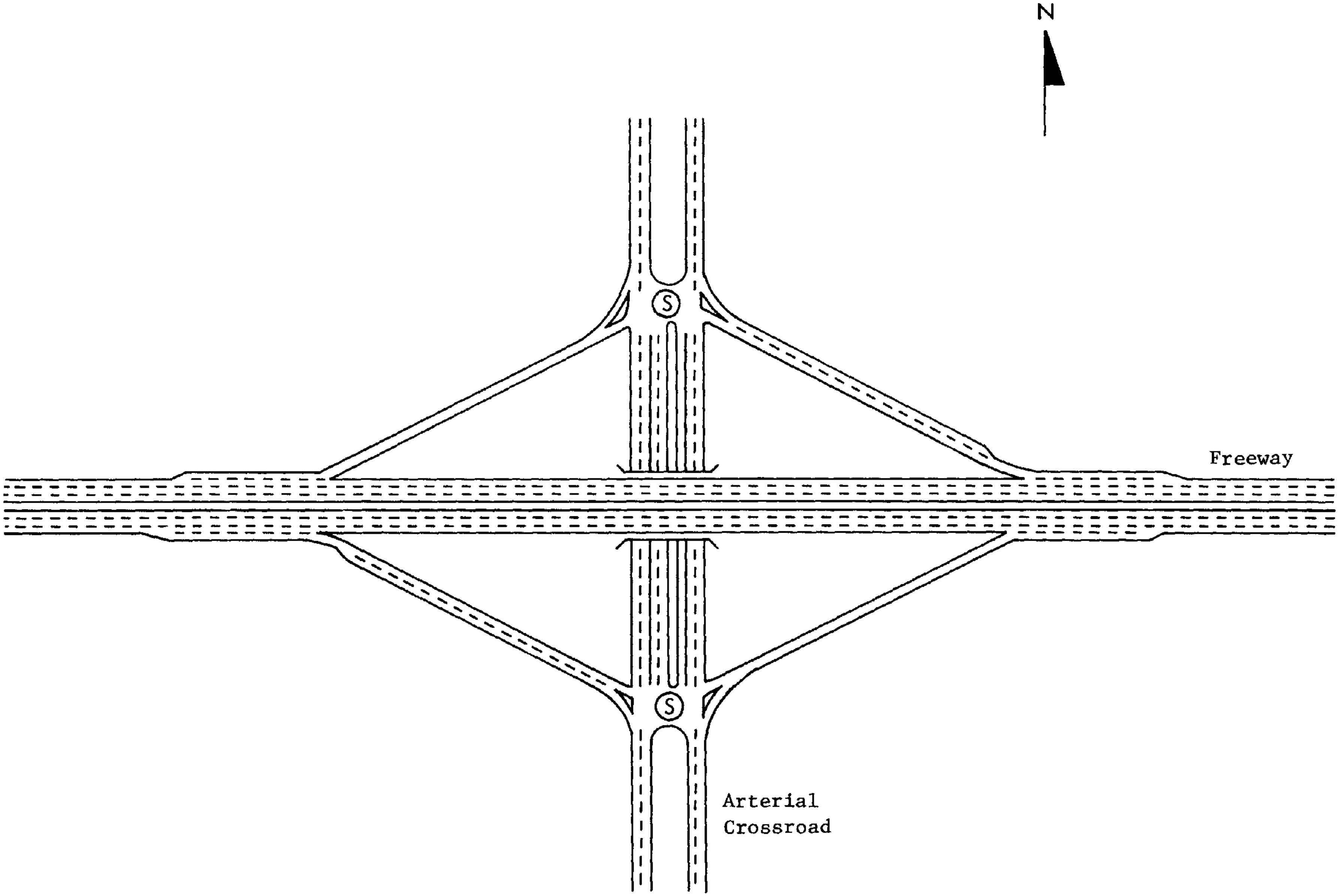


Figure 33 - Double Left-Turn Lane Alternative

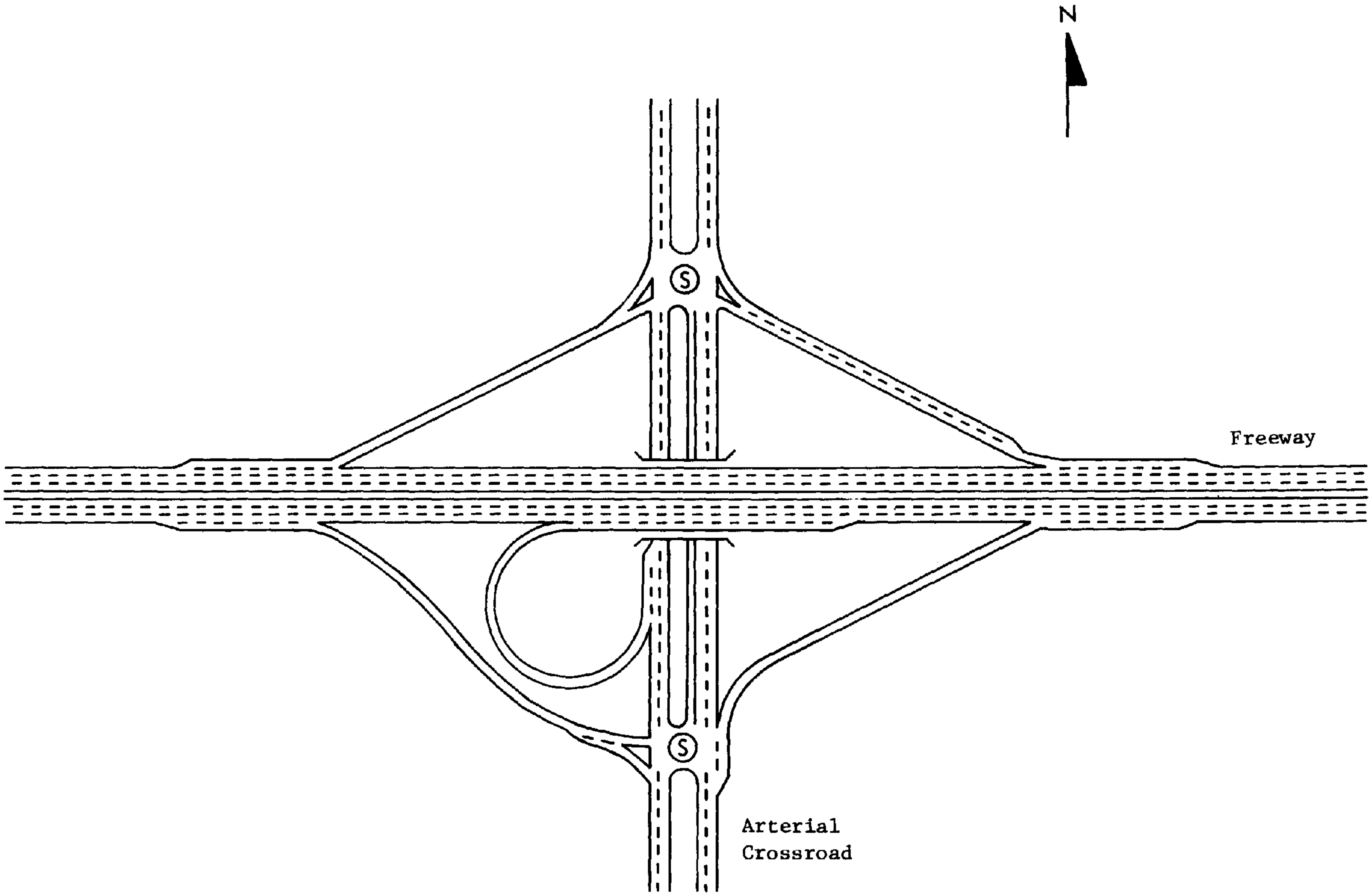


Figure 34 - Loop On-Ramp Alternative

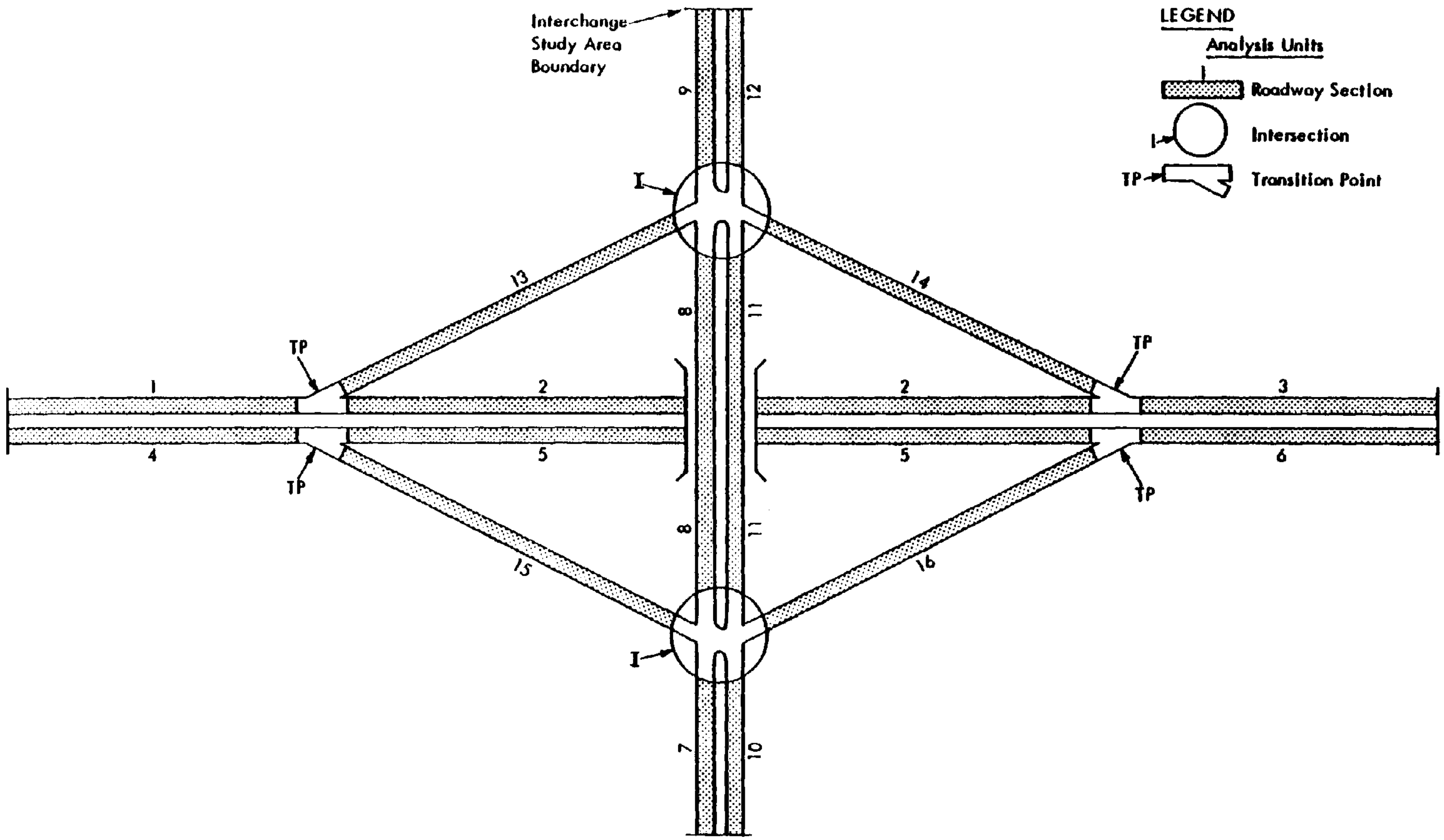


Figure 35 - Operational Analysis Units for Do-Nothing and Double Left-Turn Lane Alternatives

TABLE 41

TRAVEL TIME ESTIMATES

Alternative	Analysis Year	Daily Total Travel Time (hours) Roadway Sections	Inter-sections	Total	Yearly Total Travel Time (hours)	Annualized[a] Total Travel Time (hours)
Do-Nothing	1980	1,228	314	1,542	385,500	546,750
	2000	1,844	1,147	2,991	747,750	
Double Left-Turn Lane	1980	1,228	228	1,456	364,000	520,798
	2000	1,844	1,021	2,865	716,250	
Loop On-Ramp	1980	1,339	213	1,552	388,000	520,873
	2000	1,997	749	2,746	686,500	

[a] Annual amount equivalent to a uniformly increasing series from the 1980 value to the 2000 value.

The results of the running cost analysis are presented in Table 43 in a similar manner. The double left-turn lane alternative was found to decrease vehicle running costs, while the loop on-ramp alternative resulted in increased running costs because of the increased travel distance for each left turning vehicle.

The operational analysis showed that both the double left-turn lane and loop on-ramp alternatives would eliminate Level of Service F conditions in 1980. However, it was found that Level of Service F conditions would recur in 2000 whichever alternative is adopted.

Quantify Safety Effects of Improvement Alternatives

The accident experience for the south ramp terminal is shown in Table 43. The table, organized by accident types, shows both the average annual accident experience for 1980 and the projected accident experience for 2000, based on a 124% increase in entering traffic volume. It is assumed that 0.5% of all accidents involve a fatality and 20% of all accidents involve a personal injury.

The estimated safety benefits for the double left-turn lane and loop on-ramp alternatives are shown in Table 44. Engineering judgment was used to develop accident reduction estimates for these countermeasures. These countermeasures have not been explicity evaluated, but the estimation of their safety effectiveness appears appropriate, especially since the operational effects are expected to be predominant at this interchange. The expected effectiveness of the double left-turn lane alternative is a 20% reduction in rear-end accidents, while the expected effectiveness of the loop on-ramp alternative is a 100% reduction in left-turn accidents.

The cost of accidents used for this example are:

. Fatal Accidents	$352,400
. Injury Accidents	12,100
. Property-Damage-Only Accidents	1,700

These accidents costs reflect 1975 cost levels. When updated to 1980 cost levels using the transportation component of the Consumer Price Index, as described in Section VII-A, the resulting accident costs are:

. Fatal Accidents	$525,000
. Injury Accidents	18,000
. Property-Damage-Only Accidents	2,500

TABLE 42

RUNNING COST ESTIMATES

Alternative	Analysis Year	Daily Running Cost (Dollars): Roadway Sections	Daily Running Cost (Dollars): Inter-sections	Daily Running Cost (Dollars): Transition Points	Daily Running Cost (Dollars): Total	Yearly Total Running Cost (Dollars)	Annualized[a/] Total Running Cost (Dollars)
Do-Nothing	1980	6,618	708	85	7,411	1,852,750	2,159,500
	2000	8,228	1,836	102	10,166	2,541,500	
Double Left-Turn Lane	1980	6,618	688	85	7,391	1,847,750	2,147,000
	2200	8,228	1,749	102	10,079	2,519,750	
Loop On-Ramp	1980	7,056	650	116	7,822	1,955,500	2,268,750
	2000	8,977	1,517	144	10,638	2,659,500	

a Annual amount equivalent to a uniformly increasing series from the 1980 value to the 2000 value.

TABLE 43

ACCIDENT EXPERIENCE AT SOUTH RAMP TERMINAL

Accident Type	1980 Annual Accident Frequency	Projected 2000 Annual Accident Frequency	Percent Fatal Accidents	Percent Injury Accidents
Rear-End	8.0	17.9	0.5	20.0
Left Turn	5.0	11.2	0.5	20.0
Right-Turn	3.0	6.7	0.5	20.0
Sideswipe	2.0	4.5	0.5	20.0
Other	2.0	4.5	0.5	20.0

TABLE 44

ESTIMATED SAFETY BENEFITS

Alternative	Analysis Year	Number of Accidents Reduced [a]			Cost of Accidents Reduced (dollars)	Annualized Cost [b] of Accidents Reduced (dollars)
		Fatal	Injury	PDO		
Double Left Turn Lane	1980	0.008	0.32	1.27	13,135	19,700
	2000	0.018	0.72	2.85	29,535	
Loop On-Ramp	1980	0.025	1.00	3.98	41,075	61,600
	2000	0.056	2.24	8.91	91,995	

a Based on effectiveness for double left-turn lane of:
20 percent reduction in rear-end accidents and,
for the loop on-ramp:
100 percent reduction in left-turn accidents.

b Annual amount equivalent to a uniformly increasing series from the 1980 value to the 2000 value.

The safety benefits were found to increase each year from 1980 through 2000, since the constant percent reduction estimates are applied to the increasing accident experience. When the increasing safety benefits are annualized over the 20 year analysis period, it was found that the double left-turn lane alternative had annual safety benefits of $19,700 and the loop on-ramp alternative had annual safety benefits of $61,600.

Construction Costs

The construction costs for the alternatives are shown below in Table 45, including both the initial cost and the annualized cost spread over the 20-year analysis period, based on an interest rate (or minimum attractive rate of return) of 8%.

TABLE 45

CONSTRUCTION COSTS

Alternative	Initial Construction Cost (dollars)	Annualized Construction Cost (dollars)
Double Left-turn Lane	$100,000	$10,200
Loop On-Ramp	$700,000	$71,400

Evaluate Alternatives and Select the Best

The double left-turn lane and loop on-ramp alternatives were compared with the do-nothing alternative through a net return analysis. The results of the analysis are shown in Table 46. This analysis, based on the annualized or "averaged" costs for the years 1980 and 2000 is appropriate for evaluating the long-term merit of each alternative. The results show that the double left-turn lane alternative is economically justified from a traffic operations and safety viewpoint and has an annual net return of $138,800. The loop on-ramp alternative is not economically justified because it has an annual net return less than zero (-$2,550). This low net return is due primarily to high vehicle running costs resulting from the increased travel distance for left-turn vehicles. It should be noted, however, that this net return is very close to zero, and the loop on-ramp alternative could become economically justified if minor changes were made in the analysis assumptions. For example, this alternative would have a net return greater than zero if the interest rate (minimum attractive rate of return) were decreased from 8% to 7%.

TABLE 46

NET RETURN ANALYSIS

Alternative	Travel Time Cost	Vehicle Running Cost	Accident Savings	Other Cost	Construction Cost	Net Return
Do-Nothing	2,460,400	2,159,500	-	0	0	
Double Left Turn Lane	2,343,600	2,147,000	19,700	0	10,200	
Difference	116,800	12,500	19,700	0	-10,200	138,800
Do-Nothing	2,460,400	2,159,500	0	0	0	
Loop On-Ramp	2,343,900	2,268,750	61,600	0	71,400	
Difference	116,500	-109,250	61,600	0	-71,400	-2,550

This example also illustrates the importance of considering all of the major traffic operational safety factors: travel time (delay), vehicle running costs and accidents. If the analysis presented above had been based solely upon travel time and safety, the loop on-ramp alternative would be almost as attractive as the double left-turn lane alternative. However, in this case, the consideration of the increased vehicle running costs makes the loop on-ramp alternative look much less desirable.

The analysis results show that the double left-turn lane is clearly preferable to the do-nothing and loop on-ramp alternatives and should be constructed if sufficient funds are available. However, the operational analysis shows that even if this alternative were adopted significant operational problems would exist in the year 2000. Level of Service F conditions were found at both ramp terminals in both peak hours, resulting in delays that could be eliminated if some further means of increasing capacity were adopted. Thus, the directional ramp alternative discussed earlier deserves a complete analysis. Other alternatives that would divert traffic from the interchange or change the basic interchange configuration from a diamond to a cloverleaf or fully directional interchange should be considered.

Conclusions

The double left-turn lane alternative is preferable to the others investigated. It is, therefore, recommended as the short-term solution to operational problems at this interchange. In the long run, the addition of a directional ramp or major reconstruction of the interchange should be considered to eliminate Level of Service F conditions.

*U.S. GOVERNMENT PRINTING OFFICE: 1982 522-046/8522 1-3