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MAJOR INTERCHANGE DESIGN, OPERATION, AND TRAFFIC CONTROL

Vol. 1. Text of Report

J. I. Taylor and others

July 1973

Final Report

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16. Abstract

The objectives of this research project were to develop improved design procedures and guidelines for major (i.e., freeway-to-freeway) interchanges through the examination and analysis of existing design procedures and current freeway operational characteristics. Pertinent information was gathered through a review of the literature, conversations with and workshop participation by practicing design engineers and traffic operations specialists, and through written questionnaires.

The criteria and guidelines used in the design of major interchanges at both the overall configuration level and the individual component level (such as entrances, exits, lane drops, major forks) are reviewed; conclusions and recommendations for future practices are stated. Freeway traffic control systems are examined in the context of major interchange design and operation, and the implications of various systems are explained.

A methodology for interchange evaluation using decision theory and tradeoff analyses is presented, with example applications. Extensive case studies of a lane drop and exit ramps at a major interchange are described to illustrate the manner in which the recommended guidelines might be applied in practice. Two sample "Fact Sheets" illustrating the manner in which design experience information might be disseminated to the design community are included.

A bibliography of over 200 pertinent references accompanies the report. This report is in three volumes. The other volumes are:

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PREFACE

Acknowledgments

Most research projects progress along a standard course of data collection, analysis, and evaluation. The degree of success in such efforts depends primarily upon the planning skills, ingenuity, and diligence of the research staff and on the availability and pertinence of the input data. At all events, the burden of data acquisition is on the research staff and is relatively independent of external agencies.

Quite the contrary was the situation on this project. The contract specifically precluding the acquisition of new field data, our primary source (indeed, our only source) of data was people--experts in the business of designing and/or evaluating highway interchanges. Some people we had to search out for help; others contacted us and graciously volunteered their support. In either case, it was only their willingness to share with us the benefit of their experience, their convictions, and their insight that made conduct of this study possible.

It is, therefore, with deep appreciation that we gratefully acknowledge the assistance of:

• all the state highway officials who took the trouble to fill out the 40-item post-workshop questionnaire relating to design aids and innovations;

• the state highway engineers in California, Illinois, Kansas, Michigan, New York, Pennsylvania, and Texas who reshuffled their busy schedules to receive and be interviewed by members of our project staff;

• the state highway department engineers, traffic operations specialists, consulting design engineers, researchers in allied areas, and officials of the Federal Highway Administration who completed an extensive 45-item pre-workshop

questionnaire and subsequently participated in the workshop sessions with enthusiasm, candor, and not a little good humor;

• Mr. Charles R. Stockfisch, who, as FHWA's contract manager, provided valuable guidance and encouragement and arranged for the welcome cooperation of the Federal Highway Administration's far-reaching organization; and

• Messrs. Stanley R. Byington and Joseph W. Hess, FHWA Office of Research, under whose auspices the study was conducted.

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Chapter One

INTRODUCTION

The introduction of high-speed, limited access highways has required the best efforts of roadway designers to maintain an operational balance between such highways and the nodes that link them, the interchanges. Clearly, the advantage, efficiency, and convenience of expressways are lost or impaired if the interchanges are not of such nature as to accommodate traffic flows between them at an acceptable level of service. It follows, therefore, that the planning, design, and construction of interchanges are considerations of high priority if a well functioning highway system is to result; that optimal designs must be well understood and strived for; and that procedures must be adopted for identifying and reasonably pursuing this goal of optimization.

While designs of interchanges are often based on evolutionary changes of past designs or on modifications of existing designs to increase capacity, such improved designs tend to develop from experience and engineering judgment rather than from an objective ranking of alternatives in quantifiable terms, based on performance. The changes seen in recent demands for highway systems suggest that a more analytic approach is necessary.

Project Objectives

The objectives of this study were to develop improved design procedures for freeway-to-freeway interchanges through an analysis of the existing design procedures and operational characteristics, to develop design criteria and guidelines for interchanges as a whole and the major components, and to determine the feasibility of the various different freeway-to-freeway interchange configurations for inclusion in adaptive freeway control schemes.

Research Approach

The scope of the overall study was such that no new field operational data were to be gathered. Pertinent information was gathered through a review of the literature, conversations with and workshop participation by practicing design engineers and traffic operations specialists, and questionnaires. This information was analyzed in terms of the project objectives, and this report documents the outcome of these efforts.

(1) Orientation to the Specific Problems

An extensive search and review of the general literature and pertinent research reports was made. The bibliography in Appendix M includes more than 200 articles and reports which were abstracted and categorized according to key words pertinent to the project objectives.

In addition, project personnel visited design engineers and traffic operations specialists in California, Illinois, Kansas, Michigan, New York, Pennsylvania, and Texas to discuss the project goals and derive information on their procedures for major interchange design.

(2) Pre-Workshop Questionnaire

An extensive questionnaire, covering some 45 items related to the project goals, was mailed to those expected to attend the project workshops (discussed under (3) below). The questions were directed toward policies and practices of state highway departments. Completed questionnaires were returned by all the highway department personnel solicited, and a number of others were returned by representatives of consulting engineering firms, research agencies, universities, and personnel of the Federal Highway Administration.

(3) Project Workshops

Two workshops were held to obtain the views of practicing state highway department engineers, traffic operations specialists, consulting design engineers, other researchers in allied areas, and members of the Federal Highway Administration. These two workshops were three days in length. All presentations and discussions were recorded, and a summary transcript prepared. There were 16-18 participants at each, in addition to project personnel (the attendees are listed in Appendix C). In all, eleven working sessions, plus an Introduction and a Summary Session, were held. Each session was approximately 90 minutes in length. The working sessions were:

- Standardization; Classification; and Adaptability
- Configuration Evolution
- The Design Sequence; Checklists
- Trade-Offs; Level-of-Merit Concept
- Visibility Analyses; Driver Perception; Design Aids
- Exits
- Entrances
- New Designs and Design Concepts
- Lane Drops; Lane Balance
- Route Continuity; Ramp Arrangement
- Local Access; Freeway Control; Bus Lanes.

The general session format consisted of the following:

- An introduction by one of the project personnel, acting as discussion leader.
- A short summary of the information derived from the literature, interviews with highway department personnel, and the completed questionnaires.
- Presentation of a set of prepared questions, and an invitation to the participants to express their opinions and to relate accounts of pertinent experience. In addition, a reasonable amount of "open" discussion in the general topic area was encouraged.
- Following the discussions, distribution of a prepared "session questionnaire." The participants were asked to provide written answers or opinions to specific questions.

(4) Post-Workshop Questionnaire

In order to follow-up some of the ideas generated by the workshop participants, a second set of questionnaires was developed and distributed to all of the states through the regional offices of the Federal Highway Administration. A forty-item questionnaire dealing with design aids was returned by 32 states. The response to an accompanying letter requesting examples of innovative designs for entire interchanges or complement parts was quite limited.

(5) Development of Interim Reports 2 and 3

On the basis of the aforementioned questionnaires, supplemented with information from the workshops and the literature, two independent Interim Reports were prepared. (Interim Report 1 was primarily a working document -- the pertinent material is integrated into this report.)

Interim Report 2 describes three broad categories of design aids -- three-dimensional models, design checklists, and computer graphics, and reviews the design community's attitudes toward and utilization of these aids.

Interim Report 3 draws attention to novel interchange designs or design features in the interests of disseminating these ideas among the engineering community for consideration in future interchange configurations.

(6) Study of Related Topics

It was necessary to supplement the information gathered thus far in the study. In particular, more information on traffic control as it relates to major interchanges, accident experience at major interchanges, and the general area of decision theory as it applies to complex problems were required. This further information was derived largely from pertinent literature sources; accident data from previous studies were provided by the Federal Highway Administration.

(7) Development of the Final Report (with Appendices)

This report presents a summary of the information gathered, with considerable analysis, and lists the conclusions reached within the study.

Design guidelines and procedures in a number of forms were developed and are presented -- particular attention is directed to Chapters Two, Three, and Four, and Appendices E, G, H, and I. These guidelines and procedures apply on a project basis and/or with respect to individual interchange elements.

THE MAJOR INTERCHANGE DESIGN PROCESS

Operational problems encountered in existing major interchanges can probably be traced back to failures in one or more of four design process components. First, there may not have been an adequate allocation of funds to provide an interchange adequate to serve the forecasted traffic volumes and characteristics. Second, traffic forecasts themselves may have been inaccurate. A third potential reason for an unsatisfactory design may be that the criteria employed in the geometric design were based on insufficient understanding of the effects of certain design parameters on traffic operations. Finally, the interchange "failure" may be due to inadequacies in the design process itself; it is this latter subject that is addressed in this chapter.

Included in the discussion are the subproblems of interchange location, alternative configuration generation and subsequent evaluation, public input into design process and the operational feedback mechanism. An evaluation of the process is made and recommendations for constructive changes are given.

Overview of the Highway Planning Process

During the first phases of the highway system design process, interchange and highway corridor design are considered as a whole. Therefore, initially the design process for an interchange is really a parallel sub-process which cannot be broken out independently from the larger problem of highway system planning.

Figure 2-1 is a conceptual view of the highway design process. The motivating forces in determining highway needs are the wishes of the general public, both local and at a larger scale, and the need for better

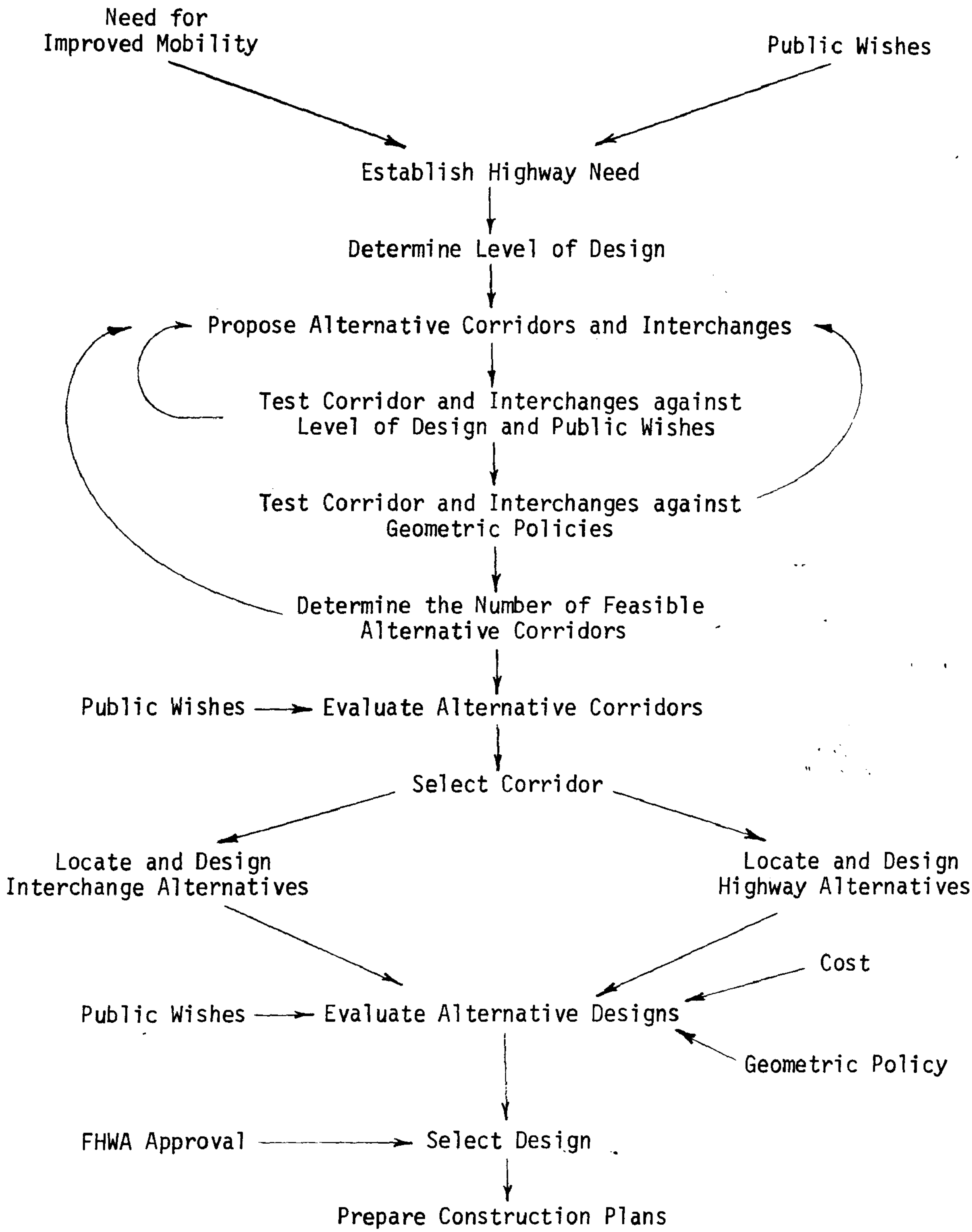


Figure 2-1. Conceptual Highway Design Process

transport services or improved mobility. These two forces are channeled through a government agency or commission (e.g., a state department of transportation) which determines that the most appropriate means for satisfying the public wishes and needs is through the highway mode. At this juncture other methods for satisfying the transportation needs are considered and may be formally investigated. If alternate modes are to be evaluated against the highway solution, a parallel design process for these modes would begin. The evaluation procedure shown in the latter part of the process is then broadened to include not only alternative highway corridors but also alternative mode designs. The figure shown is limited to highways only, however.

The level of design determination sets the quality of service which the highway facility is to provide. For the purposes of this project this level is limited to controlled access freeways, but the "determination" step is included in the conceptual development for completeness.

With the highway need and level of design established, a corridor to connect the two project endpoints is proposed. This corridor plan sets the approximate location of the proposed highway and generally identifies the interchange points. This corridor is tested to insure that it meets with the public wishes and the level of design criteria, and is consistent with the controlling geometric policies. If the proposal fails to meet any of these controls, the corridor is rejected and a new plan is advanced.

At some point a decision is made as to the number of reasonable alternatives that should be considered in the evaluation process. The common number of corridor alternatives considered varies between three and five and usually contains a "do-nothing" approach. This do-nothing alternative generally means that no improvements are made on the subject transport network, but in certain instances a do-nothing alternative might include minor control type improvements. This process continues until various alternative "solutions" are generated which satisfy the need for mobility.

The next step is to evaluate the alternative corridors in order to make a rational, objective choice between plans. An American Association of State Highway Officials publication notes that seven principal factors should be considered in the choice analysis (AASHO, 1960). Recent Federal environmental quality legislation resulted in the issuance of further criteria for project evaluation. The FHWA PPM 20-8 has directed that highway projects must be evaluated on 23 categories through the application of an Environmental Impact Statement.

The changing emphasis in highway design evaluation is evident in the two lists. The criteria recommended by AASHO are clearly more cost-conscious, while the 23 factors given in PPM 20-8 stress "quality-of-life" kinds of considerations. The latter factors are less tangible and for the most part more difficult to measure objectively. Therefore, the designer is required to handle many factors subjectively; and he must communciate their measure through description rather than numbers.

Alternative evaluation at the corridor stage is ultimately accomplished through considering a mixture of quantifiable and unquantifiable factors. The designer attaches numbers to construction and maintenance costs and user time savings while the other evaluative categories are simply written about. No explicit weighting procedure is used to combine all the factors into one measure of corridor quality. Yet, finally, a choice is made as to the "best" alternative. The designer unconsciously combines the factors, employing his implicit weighting scheme, to arrive at a conclusion.

This recommended corridor and the other alternatives are presented to the local community in the corridor public hearing. The evaluation categories are explained and the measurement of each alternative is given to the citizenry and their comments solicited. This public input is the final information source before the corridor is ultimately settled upon.

Following the corridor selection, the proposed highway passes into the route location phase of the design process. The designer generates alternative highway locations within the corridor and develops them to a point refined enough to make more accurate cost estimates than were made in the corridor evaluation phase. Usually this is the point at which the interchange design can be split off somewhat independently from the overall highway design.

The alternative locations within the chosen corridor are evaluated in the same manner as were the alternative corridors. The evaluative criteria are applied to each design (more critically and in more detail than in the corridor selection process), a second public hearing is held, and the highway location is chosen. Due to the high relative costs and large land requirements of interchange, the locations of the major interchanges will probably play an important role in the final highway alignment.

For each proposed location within the corridor, different interchange configurations can be tried. Usually a major interchange has very few feasible locations within a corridor so that the number of possible configurations are tractable. The remainder of the chapter will focus primarily on the interchange design process following the location studies.

Design Policy

The Policy-Making Process

All interchanges must meet certain standards related to geometrics and location which are commonly known as design policy. This policy is formulated at the national level through a coalition of the Federal Highway Administration and various national organizations such as the American Association of State Highway Officials, (AASHO), the Highway Research Board, (HRB), and the Institute of Traffic Engineers, (ITE). Policy is promulgated in several manuals which generally must be approved by the state highway departments through balloting procedures. These manuals include A Policy on Arterial Highways in Urban Areas (AASHO, 1957), (the "Red Book"), A Policy on Geometric Design of Rural Highways, (AASHO, 1965), (the "Blue Book"), A Policy on Design Standards -- Interstate System (AASHO, 1967), the Manual on Uniform Traffic Control Devices (USDOT/FHWA, 1971), (the MUTCD), and the Highway Capacity Manual (Highway Research Board, 1965).

These policy manuals are revised and updated periodically to incorporate increased knowledge and experience and to reflect changing priorities and community standards. The procedure employed in these large-scale revisions is slow and not clearly defined. In general, AASHO, ITE, and HRB committees, and FHWA staff members who work with them to produce a volume, collect comments and criticism on their work over a period of time. When the prevailing committee feels that enough new information has been gathered to warrant publication of a new manual, one is prepared for approval by the cognizant agencies. No set criteria, other than the collective expert judgment of the committee, are employed to determine when a policy is outdated.

No specific office exists for major policy revision nor is any common process employed to prepare the revised edition. Each revision is treated as an individual problem rather than a continuing task of updating and improving. This is not to say that continuity does not exist between successive revisions of policy manuals since much of the federal and state staff involved remains intact and any new group has the old manual to start from.

Policy modifications are also effected through the Policy and Procedures Memorandum (PPM) system. The FHWA distributes these modifications to specific design procesures or policy to the state agencies on a continuing basis. This system, therefore, changes design policy more rapidly than a policy manual can be revised and adopted.

Design Policy Conclusions and Recommendations

Three problems can be identified in the policy making process. First, no well oiled machinery exists for the periodic revision and updating of design policy manuals. Second, the time required for a major revision of a policy manual is excessive, and third, the tie between policy-makers and highway researchers is, at best, informal.

Despite these problems, however, positive steps toward improved design are being taken. Design criteria, developed from more enlightened policy, have been made more responsive to driver needs and community wishes. Interchanges are safer, more efficient, and a truer reflection of community wishes today more than ever before; and it appears that this trend will continue. Additionally, the policy-makers have succeeded in gaining nearly nationwide acceptance of design guidelines, resulting in a desirable degree of uniformity across the country.

It is recommended that the cognizant agencies create staffs whose purpose would be to consider and draft revisions to the four basic policy guides, the "Red" and "Blue" Books, the Highway Capacity Manual and the Manual on Uniform Traffic Control Devices. The advantages in establishing a continuing staff for such a purpose will include:

(1) minimization of "start-up" and organization time required for policy revision;

(2) the possibility for more formal coordination of the four policy manuals;

(3) a permanent deposit for criticism and suggestions of existing policy; and

(4) the ability to establish a uniform policy revision procedure to derive maximum benefit from experience and research findings.

These offices should not usurp the policy-making powers held by the national organizations, but rather should serve a staff function in policy-making by submitting drafts for approval and adoption by the respective agencies.

Close ties with the research agencies within the Federal Highway Administration and the Highway Research Board should be maintained by these staffs. This would insure rapid transformation of research results into design policy and serve to identify problem areas requiring additional empirical study.

Consideration should be given to revising the national documents into loose leaf formats, and integrating them with the PPM procedure, thereby making revision by section a viable alternative to the total revision currently practiced. This procedure would lead to national policies which are more indicative of current experience and research findings and minimize the time lag involved with policy revision. Many state agencies have already adopted such a system and it would appear natural for the national agencies to follow suit.

Finally, these policy staffs should give high priority to establishing normal feedback systems which would transmit operating information back to the policy makers. Such a system might take the form of establishing monitoring programs of major interchanges which would produce objective records of operating experience including accidents, speeds, volumes, delays, and user and non-user attitudes. This information, when combined with the physical characteristics of the structure, could go a long way toward improving the policy to be used on future designs. A "Fact Sheet" approach to this information collection and dissemination is outlined fully in Appendix I but an example is provided on the following four pages.

EXAMPLE "FACT SHEET"

Entrance Ramp, Two Lane

Location:

- US 6/75 (Douglas Street) entrance ramp to Route I-480 at Missouri River Bridge connecting Omaha, Nebraska, and Council Bluffs, Iowa.

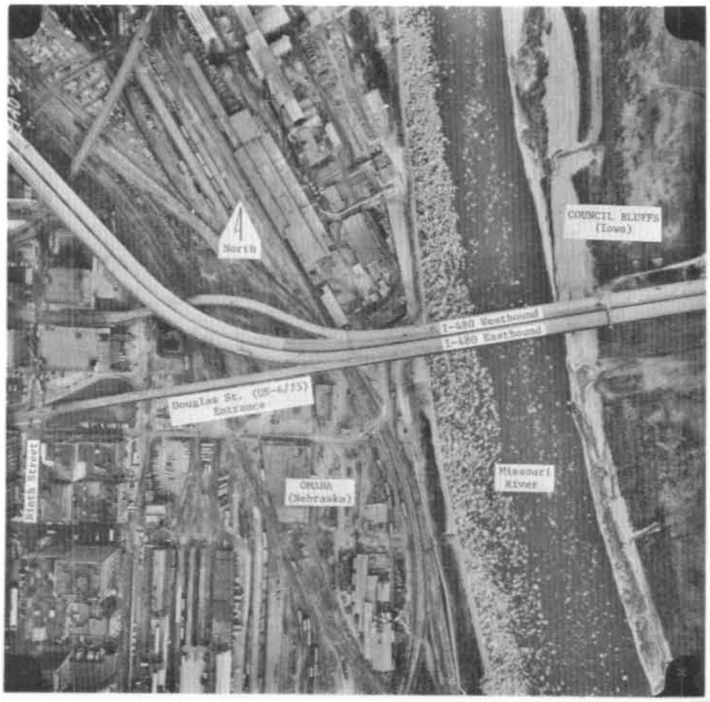


Figure 1. Location

General Design Features:

- Average Daily Traffic on the bridge at the time of design (1962) -- 30,830 vehicles. All eastbound traffic to be carried solely on the entrance ramp.
- Projected volumes for 1984 -- ADT of 76,000 and DHV of 8,665. Entrance ramp DHV of 2,630.
- Trucks = 4%
- Directional Split = 63%
- Design Speed, through road = 50 mph
- Nearest Exit -- 0.35 miles downstream
- Nearest Entrance -- 0.39 miles upstream

- Area Type -- Urban with heavy commuter traffic
- Combined population of muncipalities = 360,000

Special Conditions:

- Due to staged construction, the entrance ramp was required to carry all eastbound bridge traffic until completion of the up-stream through lanes.
- The bridge is limited to four lanes in each direction.
- The elevation of the bottom and the length of the entrance ramp were dictated by the location of railroad tracks at Ninth Street at the beginning of the ramp. (See Figure 1.)

Final Design:

- See Figure 1.
- The ramp is on a 6% upgrade and has two 15-foot lanes.
- There are three through lanes prior to the merge and four lanes, total, after the merge.
- There are 3-ft. solid parapets along both the elevated freeway and bridge.

Operational Evaluation:

- After the through lanes were opened, a merging problem was created by the limited sight distance; a result of the grade on the entrance ramp and the parapets on the roadways.
- The reduction in the number of lanes soon after the merge point added to the problem. Evaluation based mainly on public opinion and accident records. (Figure 2 shows the accidents definitely traced to the merging problem.)

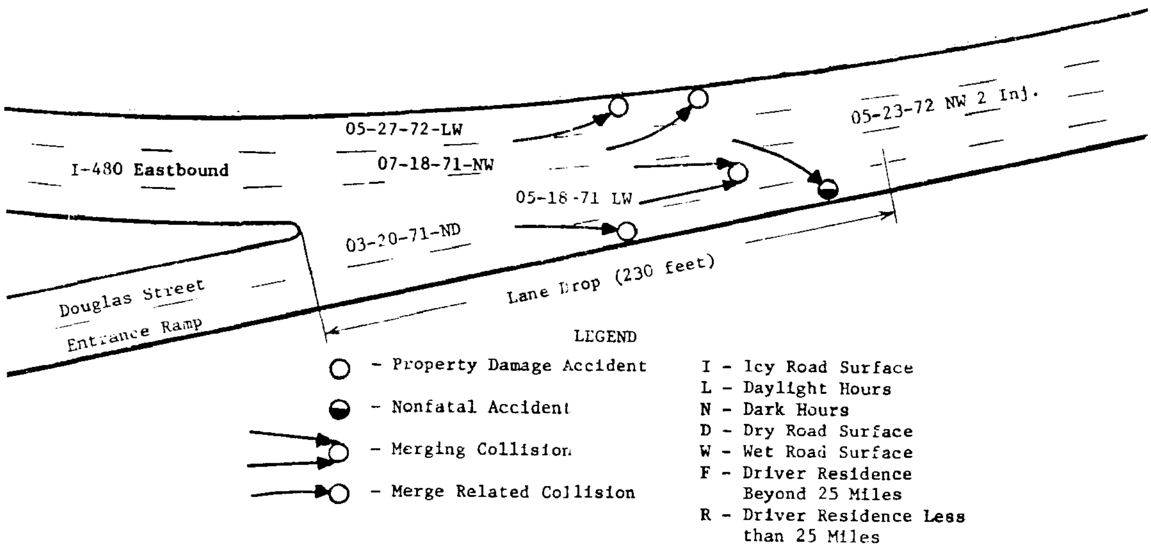
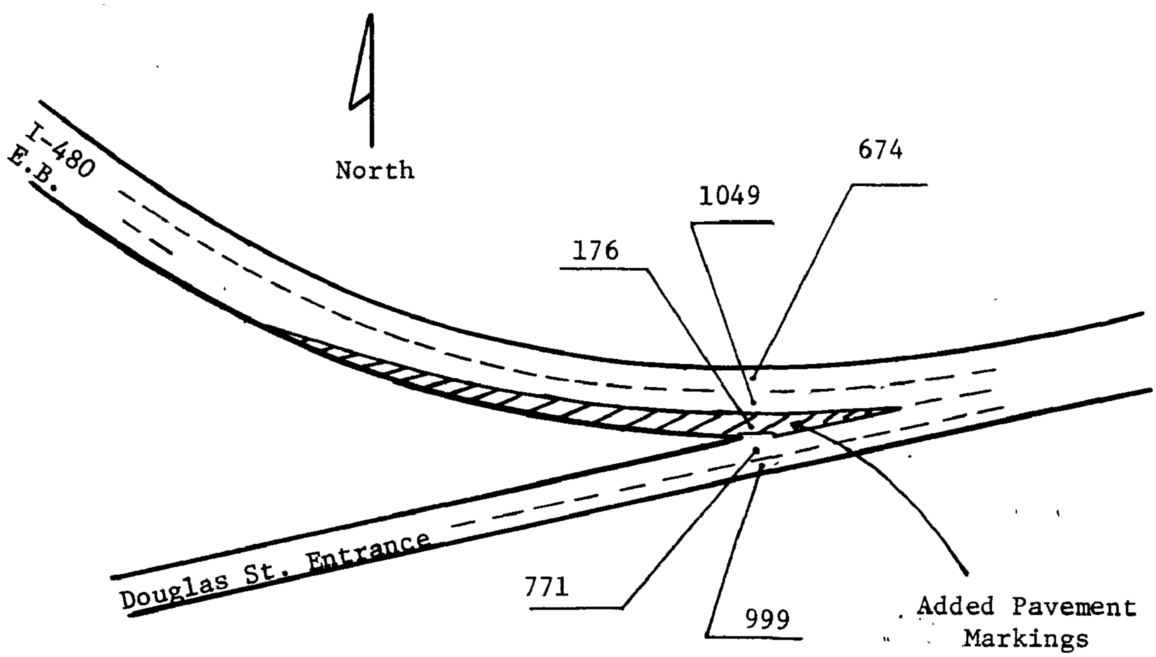


Figure 2. Collision Diagram

Remedial Action:

- A lane-by-lane vehicle volume count was made to determine the actual traffic distribution. The results of this count are shown in Figure 3.
- Based on this traffic data, pavement markings and signs were installed to discourage through I-480 traffic from using the lane nearest the entrance ramp. The pavement markings are shown in Figure 3.



Afternoon peak-hour volume counts before pavement markings were added.

Figure 3. Volume Counts and Corrective Measures

Evaluation After Remedial Action:

- Not available at this time.

Lessons Learned:

- Lane Drop. The lane drop very near the merge point (see Figure 2) was unsatisfactory. A decision on a second maneuver (lane change due to lane drop) was required immediately after completion of the first maneuver (merge), with virtually no time for information processing. Remedial action at this location included moving the lane drop -- eliminating the merge. (See Figure 3.)
- Grades, Entrance Ramp. The relatively steep grade (six percent) was a major factor in the unsatisfactory operations. It added to the merge problem created by the restricted sight distance and sudden lane drop.

- Sight Distance. Sight distance at the merge point was not sufficient. It was restricted by the use of parapets on the entrance ramp and elevated through roadway. This problem was compounded by the fairly steep (six percent) grade on the entrance ramp.

- Entrance, Two Lane. The two-lane entrance ramp did not function adequately. It might have functioned better if the lane drop had been moved further downstream and the rear and forward sight distances had been longer.

Key Words for this Fact Sheet:

- Entrance Ramp
- Grade
- Lane Drop
- Sight Distance
- Two-Lane Entrance

Interchange Location

The location of a major interchange can either be fixed by the location of the two freeways involved, or the interchange can control for freeway alignment. That is, the location of the two freeways can be settled, thereby fixing the interchange area, or the interchange can be located first and the intersecting highways bent to conform to the interchange location.

The question of which location is determined first, the highway alignment or the interchange, is often dependent upon whether the proposed facility is to be built in an urban or rural area. For a densely developed urban area, the alignment of a freeway is often determined by where interchanges can be constructed. In rural areas, however, the land costs are less and there exists more flexibility in moving freeway corridors. Therefore, freeway alignment is usually set first and the location of the interchange follows at the intersection point.

Location Conclusions

The locations of major interchanges are the result of a series of compromises between the community and the corridor planner. The evaluation of the interchange location is performed primarily from a non-user standpoint, i.e., the interchange is built where community opposition will be least. Because no person wants to be forced to relinquish his home or place of business, it is inevitable that conflicts requiring compromise solutions will arise. The location evaluation, then, is not an exercise to determine the optimal decision but rather a problem of selecting the least objectionable of a set of objectionable alternatives.

Unfortunately, the location compromise may affect the interchange user by restricting the geometrics of the resulting facility. That is, certain "untouchable" land parcels may dictate a design which is less than satisfactory to the driver population but meets the requirements or affords some relief to the non-user public sector immediately surrounding the interchange.

Generation of Alternative Designs

After the location of the proposed interchange is tentatively set, the designer must come up with alternative plans or sketches for the interchange configuration. The information required at this stage of the process includes:

(1) Topographic map of interchange location,

(2) An estimate of the proposed and/or existing freeway alignment,

(3) Forecasted traffic volumes for each movement,

(4) A set of design criteria based upon a desired level of service, and

(5) Knowledge of serious right-of-way or environmental constraints.

A conceptual view of the interchange configuration generation process which delineates the inputs required is shown in Figure 2-2. The five major inputs are considered simultaneously by the designer to arrive at one alternative design for evaluation. The diagram shows an adjustment process for the freeway alignment, the traffic forecast, and the design criteria. If the designer discovers that the interchange cannot be fit into the area available, he may elect to adjust certain variables previously assumed to be fixed. For example, if a suitable configura-

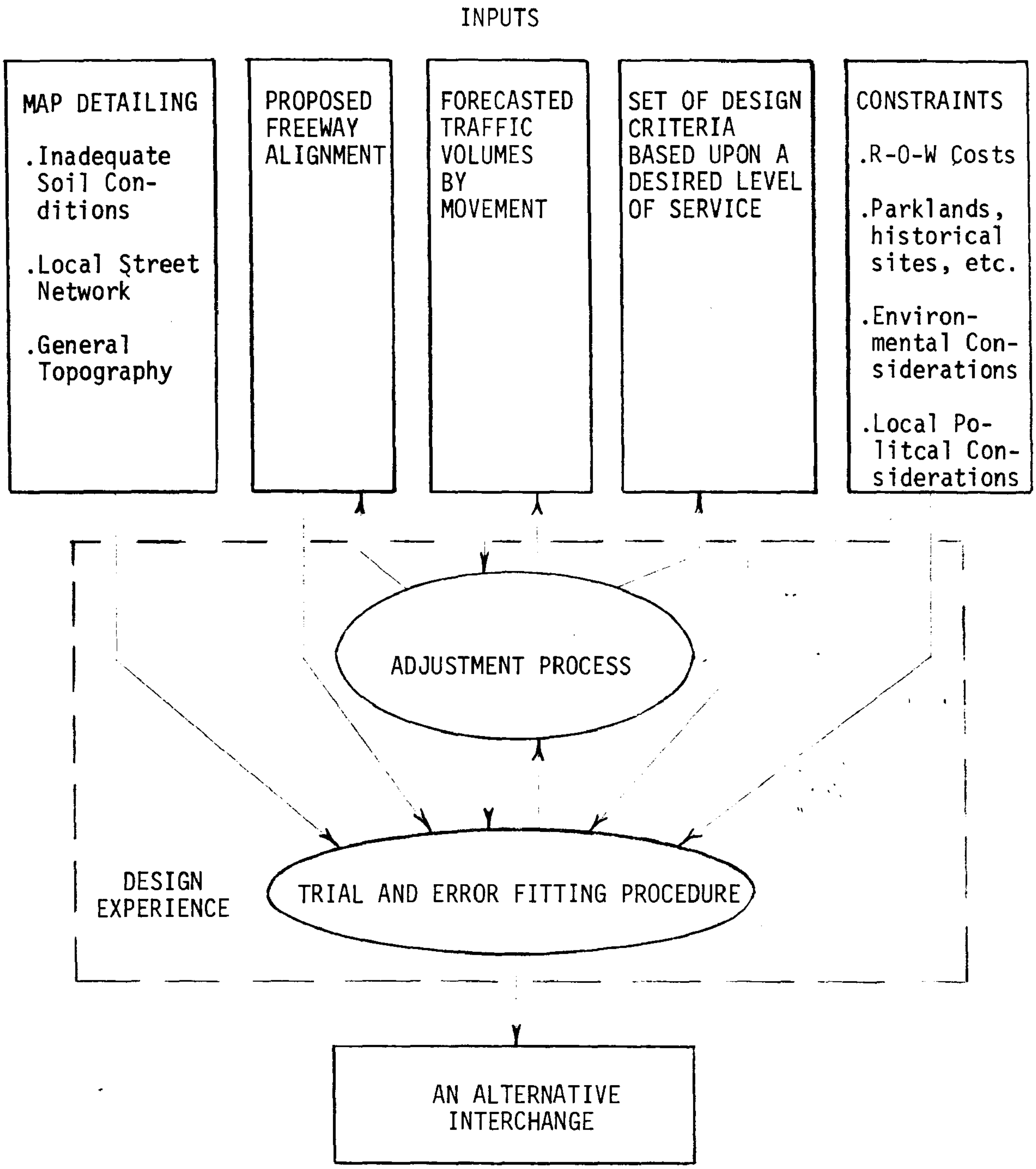


Figure 2-2. Conceptual Alternative Generation Process

tion cannot be developed within the given land area, the designer may elect to reduce his design standards or consider altering the freeway alignment slightly.

Implicit within the input adjustment and trial-and-error fitting portion of the alternative design is the experience of the design engineer or geometrician responsible for generating alternatives. The importance of experience at this stage of the total design process (and the other phases as well) is probably the point of most agreement among the workshop participants. The engineers were virtually unanimous that the person who handles this portion of design should not be without extensive highway design and operation experience.

The designer is required to weigh a large number of factors which bear upon the adequacy of an interchange. He must consider public sentiment, costs, and available resources, political "clout," traffic operations, engineering materials suitability and higher level approval mechanisms, to name only a few considerations. Certainly the ability to produce a workable alternative interchange within this complex environment is a highly valuable skill.

Alternative Generation Conclusions and Recommendations

The ability to keep in mind all the parameters, constraints and guidelines, with enough understanding and clarity to produce a feasible interchange design, is an ability which is more artistic than scientific. The generation process is not a "cookbook" type of procedure which can be handled by an inexperienced individual with a set of instructions or by a computer programmed to design interchanges. The subject is too complex, the parameters too numerous, and the constraints too unique to enable one to program a feasible solution, much less attempt to optimize a final choice.

Therefore, the key to successful alternative design generation lies with the individuals charged with the responsibility for "solving" the interchange problem. The interchange designer should be required to undergo a rigorous and complete personnel training and screening process. To improve the design it is imperative that the most talented and qualified individuals be assigned to the task.

Interchange designers should continuously keep abreast of current developments in the fields crucial to interchange design. The pertinent categories are not restricted to traditional engineering disciplines such as foundations, materials and construction techniques, but include social, economic and environmental factors as well. Technical updating is being provided adequately through the efforts of professional societies which distribute technical literature and sponsor conferences on engineering subjects, and through in-house seminars, training programs, etc., by FHWA and many state highway agencies.

In recent years the "soft" sciences are being given considerable more attention by the individual state highway agencies, AASHO, FHWA, ASCE, HRB, ITE, etc. However, this emphasis varies from state to state, and sometimes only by the top and middle management levels.

In many cases the designers's contact with assessments of the socio-economic impacts of his work is still sketchy at best. A major effort should be mounted, therefore, to substantially increase the intensity and quality of knowledge on social and environmental factors available to the highway designer.

Evaluation of Alternatives

The classical evaluative technique for assessing alternative designs is known commonly as benefit-cost analysis. More recently the term systems analysis has been applied, denoting a larger frame of reference in which to assess costs and benefits. In either case, the rationale employed is to choose that alternative which has the highest ratio of benefits to costs.

Conceptually, this is a rational approach to evaluate interchange alternatives. However, the complexity of the highway problem makes quantitative assessment of many relevant costs and benefits impossible at the present time; therefore, the designer generally quantifies what he can and simply judges the effects of the rest. His decision as to the best of the alternatives is based on numerically explicit costs and benefits as well as intangible costs and benefits assessed through judgment.

Alternative Evaluation Conclusions

It appears that systematic, rational evaluation of alternative solutions to the interchange problem suffers from a lack of quantitative information and from the absence of a suitable evaluative tool or method. This need for improved techniques and information affects the interchange design at two levels. First, and most basically, the wrong alternative can be chosen from those investigated. Second, a lack of both technique and information makes convincing an ever more sophisticated public that the proposed interchange is the "best," a formidable task. The first effect can be demonstrated by pointing out recent designs which have high accident rates. The second consequence is evidenced by the increasing mistrust of the highway design community by the general public.

The highway designer does make a conscientious effort to evaluate user-related costs and benefits for various alternatives. Likewise, safety and driver convenience are assessed by members of specialized disciplines such as traffic and human factor engineers. More recently, the evaluative framework is widening to include a great deal of non-user criteria for specific major interchange designs.

A final conclusion is that the resistance to adopting an evaluation methodology is formidable. The decision makers themselves appear very cautious, even defensive, when the concept of aiding their decision-making is discussed. They maintain that the evaluation of something as complex as a major interchange is too large and too intricate to trust to any type of "numbers game." Even if the evaluation strategy is presented as an aid, not a decision-making device, designers appear to be very unenthusiastic. The reluctance on the part of professionals trained in a physical science to accept a method for quantifying judgment is perplexing, particularly in light of the mounting public questioning of the validity of their decisions.

<u>Alternative Evaluation Recommendations</u>

The interchange design community should follow the lead of the business world and the military by applying some of the Bayesian Decision Theory techniques to the evaluation problem. The underlying principle of such techniques is that by having a decision maker express his judgment about particular interchange evaluative categories, the probability that he will make the right decision will be increased. Therefore, a decision theory methodology seeks not to replace judgment in evaluation, but to assess alternatives in an explicit and systematic manner.

Decision theory advocates recognize that there are two basic types of decisions -- decisions based on complete, accurate information, and decisions based on uncertain information. It is in the latter category that evaluations and choices between alternative interchange designs must be made since so many of the projected effects of the facility are unknown or can only be grossly predicted. The advocates of the decision theory approach recognize that the choice has to be made under uncertainty and seek to structure the problem so as to incorporate estimates of the uncertain factors rather than ignoring them.

Uncertainty usually affects most of the variables which we combine in assessing and evaluating alternative interchange designs. Sometimes this uncertainty is dealt with by combining conservative values for each of the variables. In other cases, the "best estimate" value for each variable is selected. Unfortunately, the first approach is likely to result in an "over conservative" evaluation, while the second approach disregards the consequences of any variation around the best estimate value.

The purpose of the proposed decision theory approach is to eliminate the need for restricting one's judgment to a single optimistic, pessimistic, or "best" evaluation by carrying throughout the analysis a complete judgment on the possible range of each variable and on the likelihood of each value within this range. The product of the analysis is not just a single value for the "worth" of each design alternative, but a judgment on the possible range of the "worth" around this value, and a judgment on the likelihood of each value within this range.

Initially, the "goals" to be achieved by constructing the interchange must be specified, and then the attributes variables which contribute to meeting or defeating these goals must be listed. Judgments regarding the proper value for each of the variables then take the form of probability distributions.

Probability can be thought of in two contexts, mathematical and subjective. The mathematical concept of probability relies on frequency data to produce expected percentages of occurrence. The Bayesian statistician admits to another kind of probability, labeled "subjective probability." This kind of probability is derived from a person's intuition about a particular event which is about to occur, but whose outcome cannot be predicted mathematically. (Most "subjective" judgments obtained from experts are based on some sort of "objective" experience. For example, usually the past record of similar events leads the expert to attach more importance to one outcome than to another.)

If a person says that he feels there is a 35% chance of rain tomorrow, then he has given a subjective probability of 0.35 that it will rain. A Bayesian's statistical approach will admit this kind of subjective information into a subsequent analysis and will, in effect, place a value on the decision-maker's judgment.

From a mathematical point of view, the Bayesian decision theory approach consists of aggregating the probabilities assigned to each of the many variables. There are a number of ways in which this can be done -- the Monte Carlo simulation technique was selected for this study.

The basic idea underlying the decision theory approach is relatively simple. Through the analysis, a probability distribution for the "total worth" (a measure of the success of the particular design alternative in meeting the goals established for the interchange) is developed. From this graph, one can say that an alternative has a 60% chance of having a total worth of "6.0" or more. The interpretation is that if a great number of similar projects were constructed, it is expected that about 60% of them would have a worth exceeding 6.0. Conversely, if a great number of similar interchanges were constructed and if 60% of them had a worth exceeding 6.0, it could be said that the probability of exceeding 6.0 is 60%. Hence, mathematically, the simplest technique is to build a great number of projects with the characteristics of the one being investigated, and see how many of them have worth values exceeding 8.0, 7.0, 6.0, etc. In practice, the value of each of the uncertain variables is chosen by random selection, and the worth of the interchange is computed for the project defined by these values. The process is repeated many times and the results statistically analyzed. In a sense, then, a large number of similar projects are built on paper, using probability distributions for the input variables rather than single values, and the resulting payoff distribution indicates the probability of achieving a total worth exceeding any given value within the range of possibilities.

It is felt that comparison of payoff distributions developed for the various alternatives is more meaningful than comparisons of single "conservative" or "best estimate" values. The technique will become somewhat clearer through an example.

Step 1: Establish a Goal Hierarchy

The important evaluative attributes for any decision can be logically arrived at by considering the goals of the action as a hierarchy of increasing explicit subgoals. The goal structure may be visualized as a tree which becomes more defined in its terminology as one moves out the branches. The lowest level subgoals become the attributes of the evaluation procedure and are later represented as performance measures.

Step 2: Establish a Performance Measure for Each Lowest Level Goal

A performance measure must be adopted which reflects how closely each alternative design comes to satisfying the goal. For example, one goal may be to keep construction costs low, with the performance measure in dollars. Another goal may be to keep the noise level in the community low, and the attendent performance measure might be decibels at some prescribed distance from the edge of pavement.

It is desirable to express goals or goal attainment in terms of physical measures. Unfortunately, this is not always possible, either because no measure exists or the goal is not expressed at a fine enough level. An example of an attribute with no performance measure might be neighborhood disruption. In this case, the performance measure may have to be a direct worth estimate of the value of the alternative rather than a physical measure.

Figure 2-3 presents an example of the goal hierarchy concept and the matching performance measure. It is not intended to be a recommended format for all projects, but is given only to illustrate the output of a goal hierarchical structure and performance measure procedure.

Step 3: Generate Alternatives

Major interchange design is essentially a search and selection procedure with the generation of alternative designs constituting the search and an evaluation procedure for selecting the best of the alternatives. Alternative designs should be generated to cover the wide range of goals which appear in the goal structure.

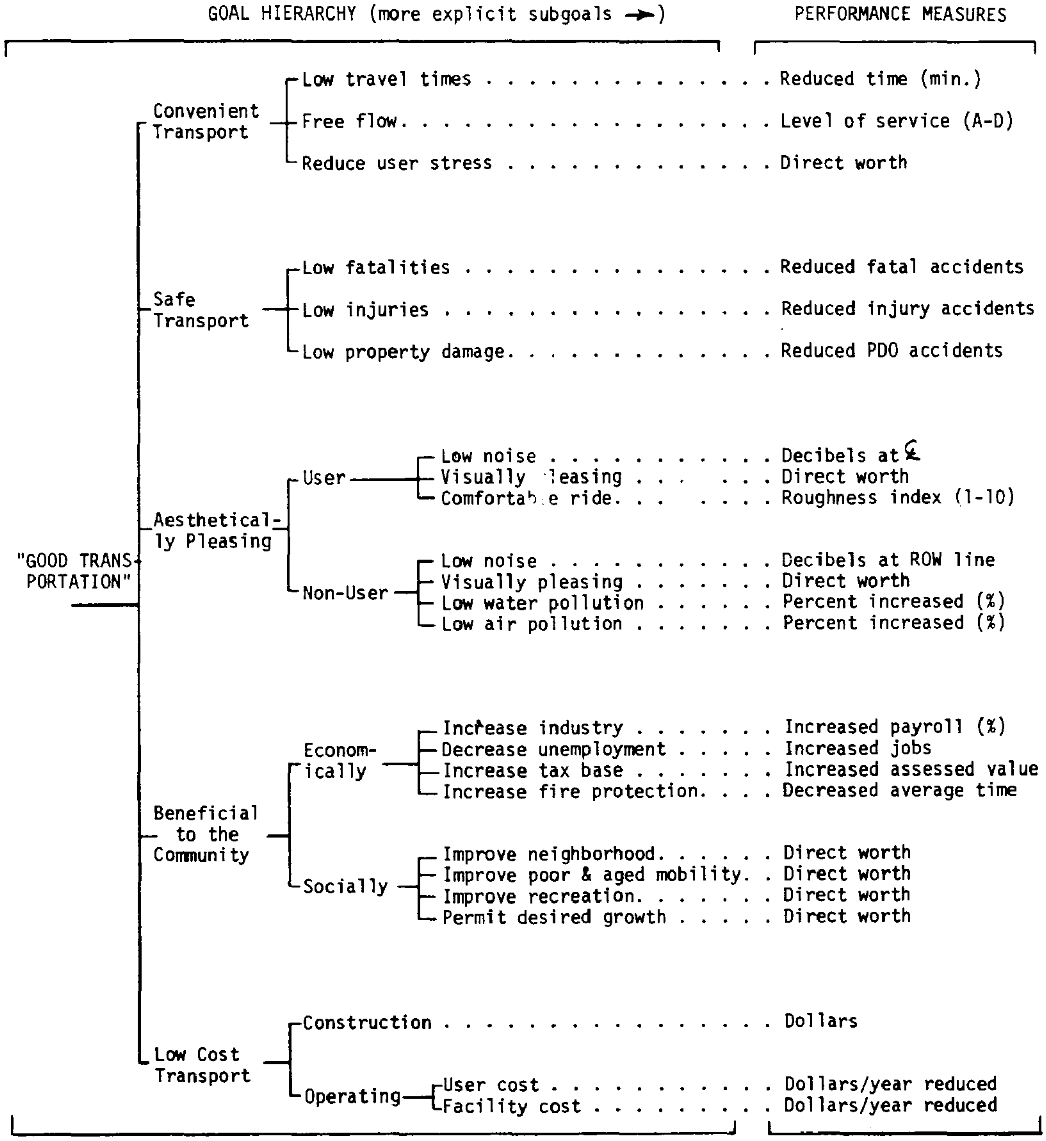


Figure 2-3. Example Goal Structure and Performance Measures

Step 4: Obtain Performance Distributions

Given the set of alternatives and the measures and goals with which to evaluate them, the decision maker must predict how each alternative will "score" in each performance category.

Certain tools are available for making rational estimates of future performance. These range from the Highway Capacity Manual which predicts level of service, to presentation models which can be used to assess non-user and user visual impacts.

Because the performance measures are predicted, rather than measured after the fact, a degree of uncertainty exists as to their values. The uncertainty is a function of the accuracy of the predictive device and can be expressed as a probability distribution. Whenever sufficient doubt exists as to the experts' predictive power the point value estimate should be discarded in favor of a distribution. The ranges of these distributions increase with an increase in uncertainty. Construction costs may be predicted with relatively little uncertainty because of the existence of good historical data, but accident predictions at a particular interchange may vary greatly.

Step 5: Obtain Worth Transformation Functions

In order to combine the performance measures of the individual attributes into a single over-all measure for the entire facility, the performance units must be transformed to worth or utility. Utility theory in general and the applications to the types of transformations needed in the interchange design process are discussed in Appendix E.

Examples of performance distributions and worth transformations are shown in Figures 2-4 and 2-5. No attempt has been made to ensure that the distributions are illustrative of two real world alternatives; rather, the distributions are intended to demonstrate the variety of shapes which might be encountered in a typical analysis.

Step 6: Generate a Number of Weighting Schemes

Different weighting schemes should be devised to reflect the diversity of opinion throughout the affected community. Examples of different schemes might be (1) a safety-conscious scheme, (2) an aesthetic-conscious strategy, or (3) a cost-conscious scheme. These would be constructed to give heavier weights to areas of safety, aesthetics, or costs, respectively, so as to give advantages to the alternatives with high scores in such attributes.

Figure 2-6 illustrates the application of different weighting schemes. The numbers separated by slashes are 3 alternative weighting schemes which are derived from giving different weights to the five second-level goals. These lead to three sets of final individual weights at the 24 lowest-level goals.

Step 7: Assign Prior Probabilities to the Weighting Schemes

For the example in Figure 2-6 it is assumed that the probability of Scheme A being representative of community desires is 0.3; Scheme B is 0.2; and Scheme C is 0.5. This would mean that Scheme C, the community-benefits-oriented strategy has the highest likelihood of representing the public's wishes, followed by user convenience and then safety.

These prior probability assignments must be made based on public inputs to the design decision maker through public hearings, local

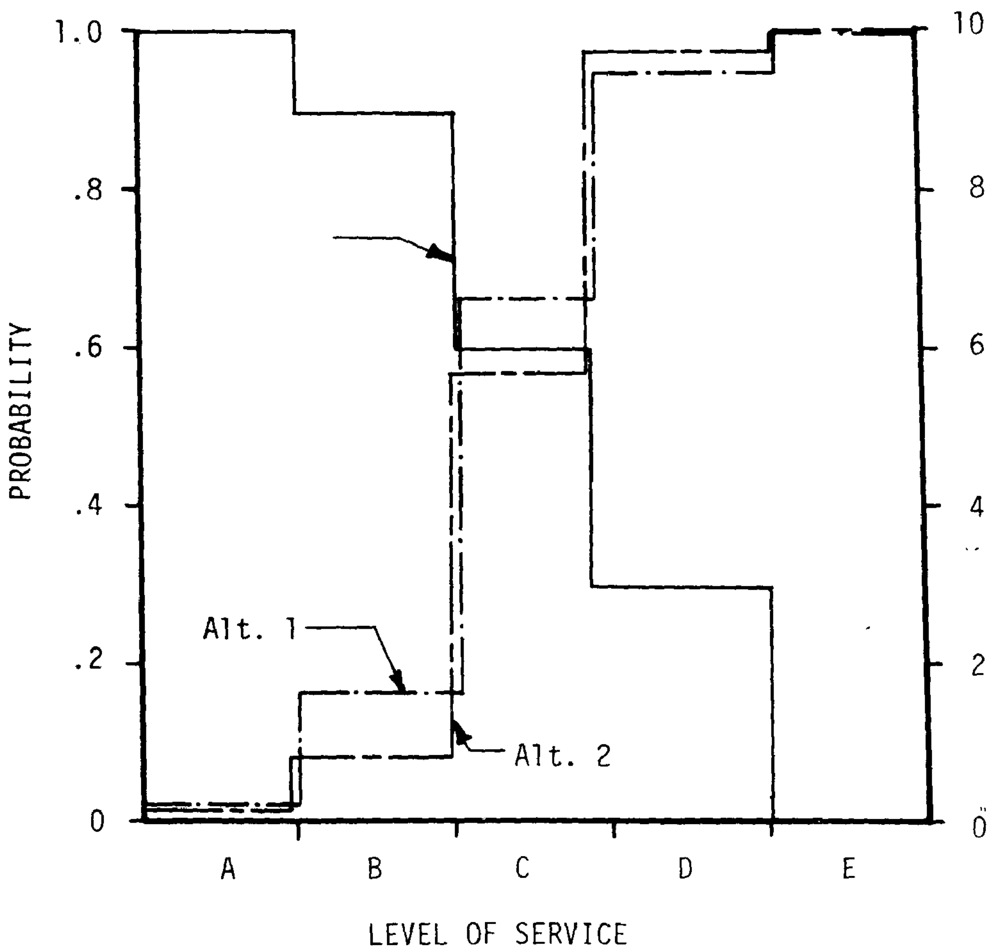


Figure 2-4 Sample Performance and Worth Distributions

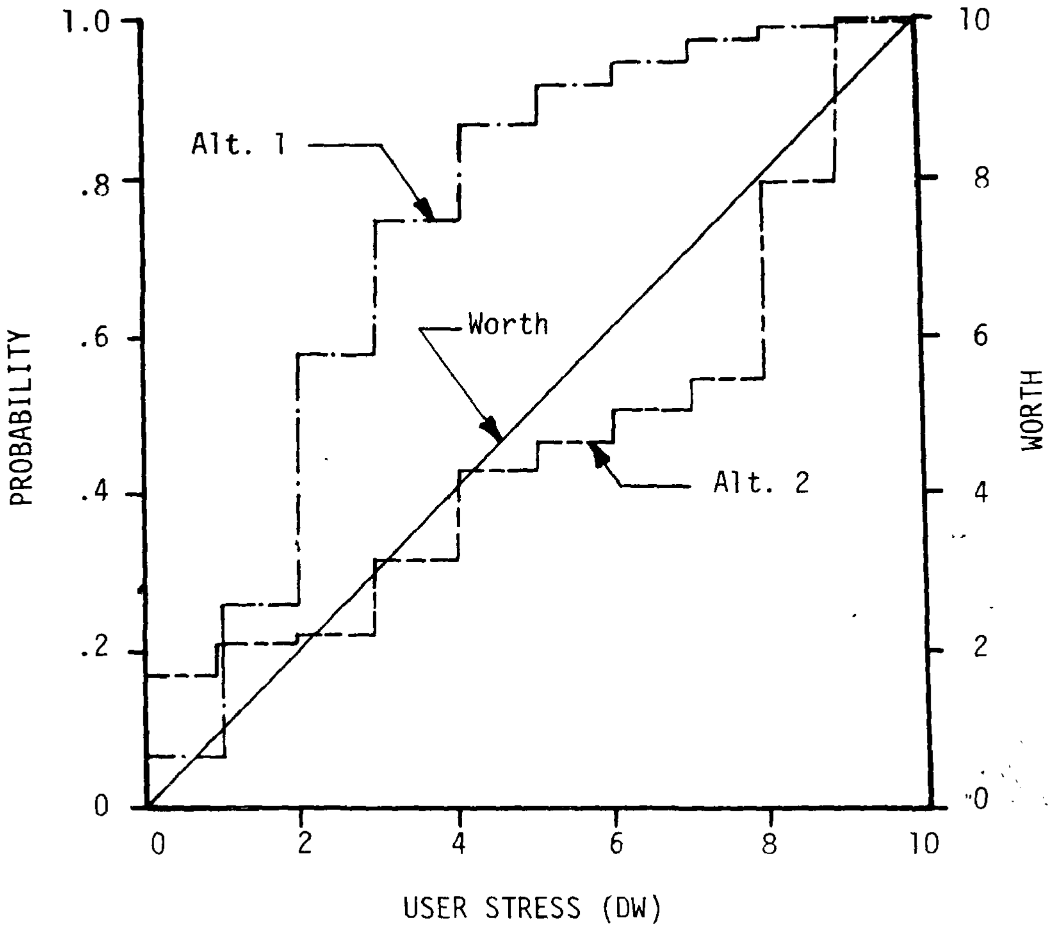


Figure 2-5 Sample Performance and Worth Distributions

GOAL WEIGHTS				FINAL INDIVIDUAL WEIGHTS
"GOOD TRANSPORTATION	Convenient Transport [.4/.1/.2]*		Low travel times [.3]	[.12/.03/.06]
			Free flow [.4]	[.16/.04/.08]
			Reduce user stress [.3]	[.12/.03/.06]
	Safe Transport [.3/.4/.2]		Low fatalities [.6]	[.18/.24/.12]
			Low injuries [.3]	[.09/.12/.06]
			Low property damage [.1]	[.03/.04/.02]
	Aesthetically Pleasing [.1/.3/.2]	User [.4]	Low noise [.3]	[.012/.036/.024]
			Visually pleasing [.4]	[.016/.048/.032]
			Comfortable ride [.3]	[.012/.036/.024]
		Non-User [.6]	Low noise [.3]	[.018/.054/.036]
			Visually pleasing [.3]	[.018/.054/.036]
			Low water pollution [.2]	[.012/.036/.024]
			Low air pollution [.2]	[.012/.036/.024]
	Beneficial to the Community [.15/.1/.35]	Economically [.5]	Increase industry [.3]	[.0225/.015/.0525]
			Decrease unemployment [.4]	[.03/.02/.07]
			Increase tax base [.2]	[.015/.01/.035]
			Increase fire protection [.1]	[.0075/.005/.0175]
		Socially [.5]	Improve neighborhood [.2]	[.015/.01/.035]
			Improve poor & aged mobility [.2]	[.015/.01/.035]
			Improve recreation [.2]	[.015/.01/.035]
			Permit desired growth [.4]	[.03/.02/.07]
	Low Cost Transport [.05/.1/.05]	Construction [.2]		[.01/.02/.01]
		Operating [.8]	User cost [.6]	[.024/.048/.024]
			Facility cost [.4]	[.016/.032/.016]

Figure 2-6. Sample Weighting Procedure

government, and special interest groups in the framework of the existing design process. Perhaps, in the future, the accuracy of these weighting scheme probabilities can be improved through the application of public opinion gathering devices.

Step 8: Monte Carlo Sample the Performance Measure Distributions

The decision maker now has before him a set of attribute performance distributions for each alternative, means for transforming them into worth functions, and a distribution of weighting strategies to combine the worth of all attributes. He must combine these distributions into a single distribution of a single payoff variable. A Monte Carlo sampling technique for both performance measure distributions and the weighting schemes can be used.

This technique (described in more detail in Appendix E) will yield one performance measure for each of the attributes, which can subsequently be transformed to a single worth value. If this procedure is followed 100 times one will, in effect, generate 100 interchanges with performance measures following the previously specified performance distributions.

Step 9: Monte Carlo Sample From the Weighting Distribution

The same type of sampling can be used to choose a weighting strategy. After one set of attribute worth measures are extracted from the previous step, a weighting procedure can be chosen randomly according to the prior probability distribution. Application of such a scheme would result in one payoff point in a distribution of points for each alternative. If 100 interchange worth sets were multiplied by 100 weighting schemes, a

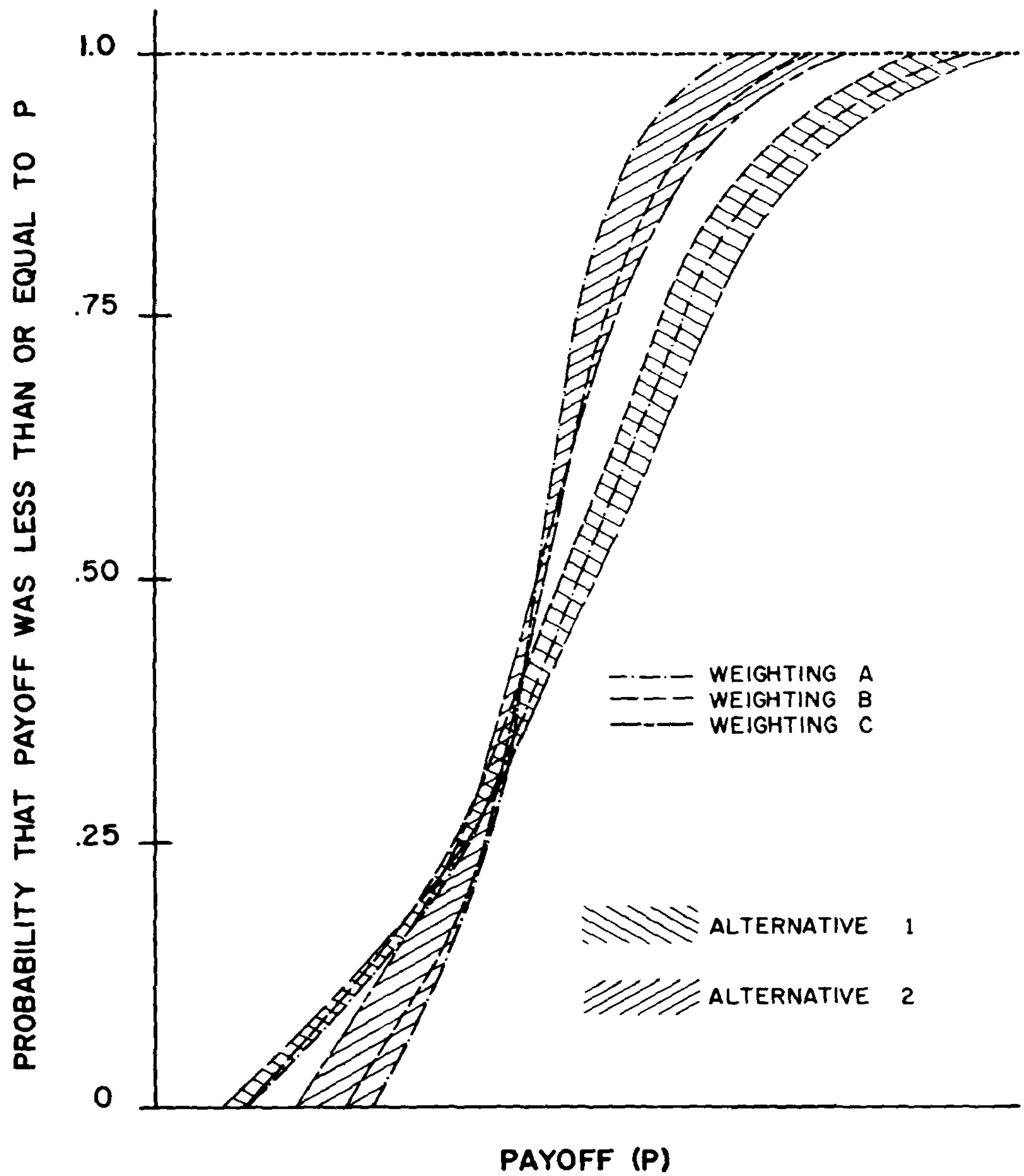


Figure 2-7. Sample Payoff Distributions with Weighting Bands

distribution of payoffs would result for each alternative. These curves would form the basis for decision.

Step 10: Produce Payoff Distributions

The final step (excepting the decision itself) is to graph the computed worths for each alternative in a cumulative format. An example is given in Figure 2-7 for two alternatives evaluated under a group of equally likely weighting schemes. The decision maker is presented with much more information than a simple "mean" payoff value. The payoff distribution gives ranges and the shape of the entire function.

In the example, the decision maker sees that Alternative 1 has the highest payoff most of the time -- it is the best alternative in about 70% of the simulated cases. However, it can be seen that there is a chance that Alternative 1 will yield the lowest worth of the two Alternatives under some combinations of values of the input variables.

The bands represent the outer limits of each alternative as defined by the individual weighting strategies. It can be seen that Alternative 2 is generally more sensitive to the different weighting schemes since the band widths of payoffs are larger.

The probability analysis gives us a complete picture of the evaluations of the alternative designs. It can be seen that Alternative 1 will probably give the higher payoff, but there is some risk that a very low value of payoff may be obtained -- considerably lower than the lowest "possible" payoff from Alternative 2. One can then accept Alternative 1 on the basis that it is most likely to produce the higher payoff, or Alternative 2 on the basis that at least some minimum payoff will be assured.

In the simpler cases, where all the performance measures can be expressed in dollar terms, the advantage of the payoff distribution presentation is more obvious, as benefit/cost ratio or rate of return can be substituted for "payoff." In that case, the decision would be whether to choose the alternative with the most promising benefit/cost ratio, or the one which will assure at least a minimum value (perhaps 1.0).

It is important to realize that the interchange under consideration will give only one value when constructed. One cannot say that Alternative 1 will perform better than Alternative 2 seventy percent of the time, but rather that Alternative 1 would be the better choice in 70% of a set of similar choices.

To implement such a procedure, two changes are required. First, the highway department should modify its organizational structure to become an information gathering agency for public opinion on transportation goals. A decision theory approach would entail some quantification of public opinion in order to be meaningful. Traditionally, the highway department has attempted to act in the best interests of the public they serve. It is desirable to improve the present perception of public wishes, and this can only be accomplished through extensive and continuous public surveys. The public hearing process is a step in this direction, but generally fails to correctly assess the feelings of the majority.

A second necessary step in implementing a decision theory evaluation approach would be the establishment of a management information system (MIS) or operations research (OR) group within the highway department. The function of this section would be to develop techniques to aid decision makers in their complex decisions. The management of highway departments should have the latest state-of-the-art methods for analysis available to them just as executives of large corporations do. This MIS or OR group would be responsible for keeping current with analysis techniques used in other fields and adapting them to highway department use. These decision aids or tools should then be made available to the decision maker.

Public Input

Information about the public wishes is obtained at three points in the highway design process, (1) project initiation, (2) corridor selection, and (3) design location. The last of these three, the design location, has most impact on the design of a major interchange although all three have some effect.

Three avenues of citizen input exist at each point in the process. These include: (1) the at-large public in the public hearings, (2) local government, and (3) special interest groups. Contact with the first two groups is usually initiated by the highway department but special interest groups are generally not officially contacted by the highway officials and therefore have to establish their own communication channels.

The highway design community is becoming increasingly aware of the public wants and needs for quality transportation. The public does have a strong voice in the final design of a freeway facility, particularly in the urban areas where special interest groups are well organized. Public participation is currently less in rural areas but is catching up to the metropolitan experience. Highway departments are accordingly increasing their efforts to interact with the local areas to insure that the design will be acceptable to the affected community.

Public Input Conclusions and Recommendations

Previous highway designs have emphasized user safety and convenience, sometimes at the expense of the non-user of surrounding community. Evidence of this can be found in the growing public mistrust of highway department decisions and increasing local opposition to new projects. The public hearing process has become a major consideration in interchange design, due to the public's increasing anti-highway sentiment coupled with the development of effective legal channels in which to express their opinions.

Highway departments approach this "impediment" to the design process in two ways. Either the public hearing is regarded as an information gathering mechanism which can be used to assess public goals, or the public hearing is treated as a sales presentation in which the best solution is sold to the community.

The second major public input device, local government, is effective in altering highway design by expressing its opinion. The possibility exists, however, that the government's position is not truly reflective of the general public's opinion; therefore, total reliance on government wishes by the highway department may lead to misleading and potentially expensive conclusions about public goals.

The function of the public hearing should be clarified at each design level. It would seem that the corridor public hearing should be an information gathering device, intended to elicit community feelings on transportation goals, existing and future transport needs, and desired level of service in meeting these needs. For a freeway situation where the impacted area is large, sometimes metropolitan in scale, a public hearing or even many hearings is not an effective tool to accomplish this task. A town meeting is no way to assess the desire for a major transport artery which may affect millions of people indirectly.

An alternative method should be developed for gaining public input on an expanded scale and in a more objective manner. A referendum approach should be seriously considered as such an alternative. The referendum could be designed to yield considerable public preference data, not only in the transportation phase of public life, but also on many other social issues.

The location public hearing is probably best approached as a sales presentation if community goals are well understood and documented. The designer who understands the public's goals really can design with them in mind; and, hence, is in the best position to trade off technical characteristics against social goals. If such goal information were quantified in some acceptable form, it could be used as supporting evidence in justifying his decisions. Therefore, once designers fix on what the public wants as to the general corridor and level of service specifications along with their trade off functions for aesthetics versus transport efficiency and safety factors, more rational designs can be generated. The location public hearing then becomes a sales exercise aimed at convincing those most closely affected, and certain special interest groups that the proposed interchange is truly in the community's best interest.

Neither of these public hearing functions, information gathering at the corridor level or sales at the location level, are the special province of the typical design engineer. It should not be expected that he can best gather public opinion data or best sell his final design. The designer should act as the coordinator, participating in both functions, but should not be responsible for carrying them out. Highway departments should develop a staff level similar to the marketing staff of a large corporation. The expertise should be available to more effectively assess public opinion and transfer the information into a format that is both understandable and useful to the design engineer; and then, convert the proposed design into information which is understandable and logical to the general public.

Operational Feedback

The term "feedback" refers to the information regarding the operation of the major interchange which is transmitted back to the design engineer. Through a comparison of the actual interchange operations to the predicted operations, the designer is able to learn and improve his design skills. The operational feedback to the design engineer can be divided into five categories. These include: (1) personal experience, (2) criticism from the public, (3) congestion information, (4) accident data, and (5) traffic operations research. It is almost always informal. The only data which are actively collected after an interchange is constructed are accidents and traffic volume counts.

The views of the public are not actively solicited by the highway department in either the user or non-user segments. Frequently, citizen complaints about interchange design, signing and safety become known to the designer, but only through the active efforts of the public, not at the request of the highway department. Complaints of nonusers are generally presented before construction; relatively few comments on aesthetic impacts are received after the opening of the facilities.

Feedback Conclusions and Recommendations

The interchange designer must be made aware of the impacts that his design has made both on the driver and on the non-user. A great deal more attention should be given to the important feedback loop of the design process so that future solutions can incorporate past successes and avoid past mistakes. This should be accomplished by establishing a review procedure aimed at evaluating the operation and community impact of the facility. To spend twenty to seventy million dollars to construct a major interchange and then have information only trickle back to the designer seems not to be a wise allocation of resources.

The feedback process would be enhanced by: (1) insuring that designers see letters of public complaints, (2) collecting and disseminating public opinion data on both the user and non-user impacts of the facility, (3) summarizing accident data into formats specifically designed for designers needs, and (4) performing interchange bottleneck analyses which attempt to point out geometric flaws leading to congestion.

The first step, insuring that designers see the letters would be simple to implement. Design offices should maintain files of these letters by location on the highway system.

Collecting public opinion after construction could be accomplished by the same group that collects opinion prior to the corridor hearing.

Summaries of the results should be made available to the design office.

Accident data are already being collected but are summarized in a format which often is not useful to the designer or policy-maker preparing safer design criteria. A restructuring of the data could be very helpful in evaluating design criteria from a safety point of view. The format must be concise and understandable to the designer.

Finally, bottleneck analyses could be carried out by the traffic engineering section and the findings transmitted to the designer. Inferences as to probable cause, design inadequancy or inaccurate forecasting could be drawn and used to refine design standards.

The sum total of such feedback would be case histories of interchange operation and impact. User and non-user opinion, safety and congestion data could be combined to form a scenario which could be a valuable learning device for interchange designers. A potential format for this feedback process is given in Appendix I.

Chapter Three

DESIGN CRITERIA AND GUIDELINES

General

The considerations which enter into the design of freeway-to-freeway interchanges are not vastly different from those involved in any highway interchange design, be it freeway-to-expressway, expressway-to-arterial, or arterial to local road. Nevertheless, those slight differences -- designing for higher traffic volumes, providing for higher design speeds and for no stops in the turning movements, and maintaining generally higher levels of service for both turning and through traffic -- are significant in deriving a properly functioning design.

One of the major objectives in the development of interchange designs should be to provide a facility which permits the driver to perform his driving task with a minimum of discomfort, indecision, and frustration. In short, the design should make it easy for the driver to perform properly and difficult to perform improperly. Conversely, when the driver uses poor judgment or makes a mistake, he should not be penalized too greatly. To achieve this goal, the communicative characteristics of the configuration should be an integral part of the geometric design. The driver who knows where he wants to go should be able to move easily through the interchange to his destination by using the directional cues incorporated in the design.

Many existing major interchanges are characterized by poor operational characteristics, as evidenced by high accident rates and traffic congestion. Some designers blame the poor performance of older interchanges on the fact that too little funds were available to build good designs. In the last fifteen years, however, more money for construction has become available, design standards have undergone continuous upgrading, and still some recently completed interchanges show poor operating characteristics. It behooves the highway community to understand and appreciate that factors other than financing function to yield a good design. This chapter attempts to come to grips with some of those factors.

The text that follows deals both with the overall configuration and with the principal design elements of major interchanges. Consideration is given to what constitutes desirable designs, what problems and limitations are associated with such designs, and what alternatives may be sought under specific circumstances. Appendix K presents the workshop discussion on each of these topics.

Standardized Design Criteria

A primary consideration of this study has been to investigate the feasibility of establishing a nationally acceptable set of design criteria and procedural guidelines that all states and the Federal Government might utilize. The administrative advantages of such a system are obvious; and, indeed, from an abstract point of view, it would likely not be difficult to develop criteria that would promote free flow of traffic at high speeds through directional interchange configurations having nearly ideal operating characteristics.

Regrettably, our conversations and discussions with state and federal officials have convinced us that adapting the general, ideal design to the gamut of specific sites across the nation is not possible and that establishing a standardized set of criteria is, hence, not feasible. Aside from the great variations in topography, land use, traffic volumes, population densities, real estate values, and availability of financing, we found that the philosophic and political approaches of the various highway departments spanned such a wide range as to make a common solution impossible. Even within a single state, the disparaties between rural and urban areas, between driver populations, and the like, are such that they demand different solutions to the design problems.

The standardization of individual design elements such as ramp terminals, lane drops, etc., has considerable merit in terms of satisfying driver expectancy. For such standardization to be effective, however, it would have to be applied to all interchanges and not merely to major interchanges. The expenditure level that this would entail mitigates against nationwide conformity as a viable goal within the next two decades. On the other hand, there does appear to be a trend toward uniformity at the local level, and this should have a positive effect on driver behavior over the long term.

Sight Distance

One of the most important design-related factors in communicating with the driver is to provide sufficient sight distance so that the driver will have ample time and information to make a choice or decision before any given maneuver is required.

In 1971, AASHO adopted a new "Policy on Design Standards for Stopping Sight Distance" which includes a "desirable" stopping sight distance as well as a "minimum" stopping sight distance. These standards are shown in Table 3-1.

TABLE 3-1

REVISIONS TO A POLICY ON GEOMETRIC DESIGN ON RURAL HIGHWAYS FOR DESIRABLE STOPPING SIGHT DISTANCES

Design Speed, mph	30	40	50	60	65	70	75	80
Stopping Sight Distance, feet								
Minimum	200	275	350	475	550	600	675	750
Desirable	200	300	450	650	750	850	950	1050

Some agencies are currently discussing the use of "anticipatory sight distance" in highway design. Anticipatory sight distance has a relationship to the point on the road ahead at which the driver generally focuses his sight at various speeds. The distances shown in Table 3-2 have been suggested by Jack E. Leisch as appropriate values for anticipatory sight distance.

TABLE 3-2

SUGGESTED ANTICIPATORY SIGHT DISTANCES

Design Speed, mph	30	40	50	60	70	80
Anticipatory Sight Distance, feet	600	800	1100	1500	2000	3000

In the design process, it is essential that the behavioral and psychophysiological character of drivers be considered and accommodated. Available information on driver response, awareness, visual acuity, and other physical and mental capacities should affect design decisions. Particular attention should be given not only to what the driver can do but also to what he is likely to do under various circumstances. It is recommended that the AASHO desirable sight distance be regarded as minimums, and that sight distances approaching the suggested anticipatory values be utilized wherever practicable.

This is not to imply that sight distance is, in all cases, the only consideration worthy of attention. On the contrary, the need for adequate overall visibility should be heeded in the design process. Not only should the driver have sufficient sight distance that he has ample time to react, but also, he should have a sufficient "view" or "preview" of the situation so that he has advanced information regarding the manner in which he will have to react.

Design Speed

Most drivers using a freeway system desire to travel as rapidly as possible for the given roadway conditions. When two high speed freeways intersect it would be highly desirable if the driver could change from one freeway to the other with no appreciable reduction in speed caused by either geometric effects or driver uncertainty. In practice, economic considerations and site limitations restrict the designer to geometrics which seldom permit the attainment of unabated design speeds on interchange ramps. The 1965 AASHO Policy on Geometric Design of Rural Highways provides guide values for ramp design speed as they relate to highway design speed (see Table 3-3).

TABLE 3-3

GUIDE VALUES FOR RAMP DESIGN SPEED AS RELATED TO HIGHWAY DESIGN SPEED

Highway Design Speed, mph.	50	60	65	70	75	80
Ramp Design Speed, mph						
Desirable	45	50	55	60	60	65
Minimum	25	30	30	30	35	40

Recommendations. It is highly desirable to provide the highest feasible ramp design speed in major interchanges. However, cost considerations must play an important role in the design of any major highway facility. Therefore, it is recommended that every effort be made to attain the "desirable" ramp speeds in Table 3-3 for the high turning volume movements, and design speeds as much above the minimum values as economically feasible for ramps servicing lower turning traffic volumes.

Travelled Roadway and Shoulders

A 12-foot lane width is universally accepted as the design criteria for traffic lanes on multi-lane freeways and ramps. The design criteria for single lane ramps on tangent vary among the states from a minimum of 12 feet (California) to a maximum of 16 feet (Illinois), with additional widening on the smaller radii curves. A high type, heavy duty pavement is used for the travelled roadway on freeways and their interchange ramps in all the states. Some states use a contrasting pavement surface on ramps for better delineation, but the current trend is to use the same type of pavement on major interchange ramps as on the freeway mainline.

Until very recently, the width and type of shoulders on freeways has varied considerably throughout the country, and even within the individual states. It was not uncommon to find ten-foot shoulders on the right of the mainline roadway reduced to three or four feet across structures to reduce construction costs. Since shoulder areas provide a recovery opportunity for errant vehicles as well as a refuge area for the disabled vehicle, it is recommended that uniform, full width shoulders be provided on all elements of the highway, including on the structures.

In 1965, AASHO recommended stabilized or surface treated shoulders. When the driving characteristics of the shoulder surface vary significantly from the mainline pavement, there is a tendency for vehicles leaving the travelled roadway to lose control. Therefore, recent recommendations to provide flush type, paved shoulders with similar riding qualities to mainline pavement have been adopted by many of the states (color and textural differences are usually recommended, however).

The width of paved shoulders on the right of the through freeway lanes is generally ten feet, although a few states prescribe twelve feet to provide refuge for stalled trucks. On the left, or median, side, paved shoulders measure four to six feet on four-lane freeways, five to ten feet on six-lane freeways, and ten feet on eight-lane freeways. Shoulder widths on single-lane ramps vary considerably among the states, ranging from six to ten feet on the right and from two to five feet on the left. The width of shoulders on two-lane ramps are normally the same as those constructed on four-lane freeways.

Recommendations. It is recommended that flush, high type paved shoulders (with textural and color contrast) be provided on both sides of all mainline roadways, turning roadways, and ramps in major interchanges. The minimum width of paved shoulders should be as shown in Figure 3-1. No reduction in shoulder width should be permitted on structures, regardless of their length.

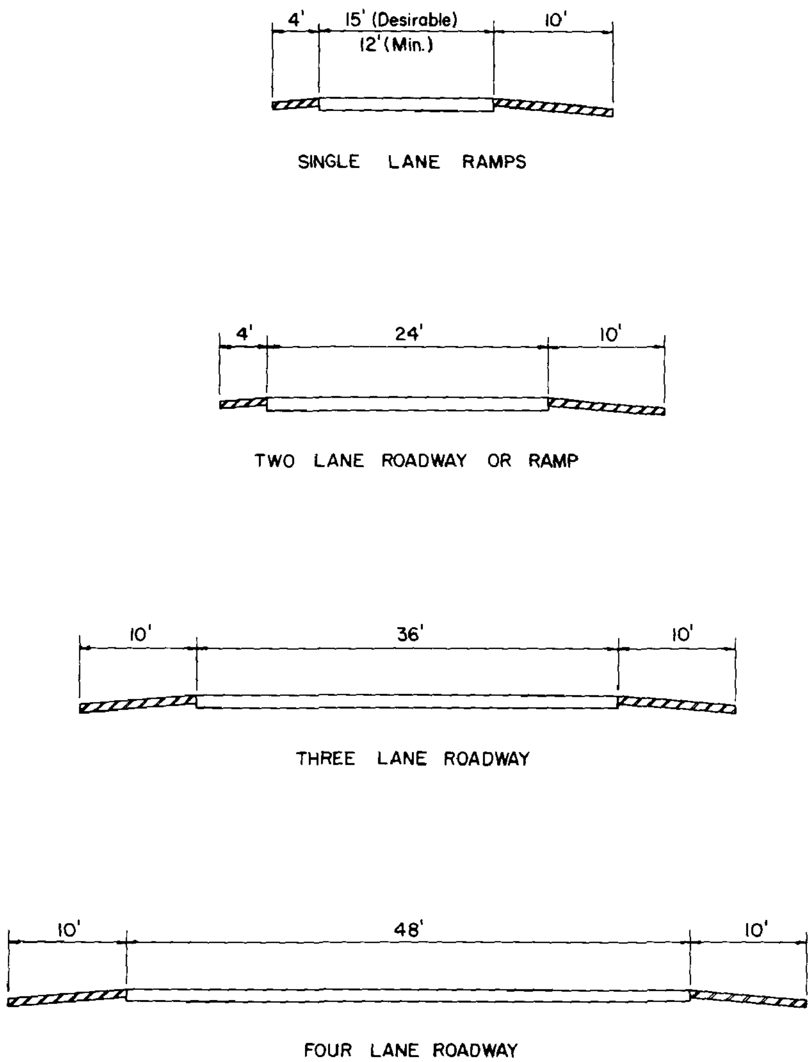


Figure 3-1. Minimum Shoulder Widths

Interchange Configurations

If it were possible to assume an ideal, or at least a standardized, set of initial conditions (topographic features, right-of-way, traffic demands, etc.) associated with every new interchange, it might then be argued that there is one particular arrangement of mainline throughways and turning roadways that is theoretically perfect in the sense of providing faultless operating characteristics, complying with the most stringent requirements for safety, satisfying the most onerous demands for minimal maintenance, and conforming to all the principles of sound engineering practice. In the real world, unfortunately, initial conditions vary so widely that no such a uniform set can be assumed. Moreover, attempts to adapt any single standard configuration to meet all these varying conditions would require such large expenditures and be likely to produce such unpromising compromises in operating features as to subvert the very goals of standardization. In short, there is no evidence to contradict the assertion of highway designers that every interchange must be individually tailored to fit the specific controlling conditions -- physical, political, environmental, and fiscal -- of its proposed site.

Certainly, from a topological standpoint there are several hundred geometric configurations which, in the abstract at least, may be conceived as possible design alternatives. In a practical sense, however, there are only a few dozen which have been perceived and evaluated as feasible solutions to the problem of producing safe, efficient interchanges. The discussions that follow are aimed at assisting the designer in selecting the most desirable configuration for a given set of conditions.

Ramp Types

Any discussion of ramp types in freeway-to-freeway interchanges generally focuses on the left turning movements, since right turning movements ordinarily involve uncomplicated designs having exits from the right on one freeway leading to entrances on the right on the second freeway.

There are five basic types of left-turn ramps. Figure 3-2(a) illustrates a direct left connection; Figures 3-2(b), (c), and (d) are examples of semidirect connections; and the loop, or indirect, ramp is shown in Figure 3-2(e). The ramps in Figures 3-2(a) and (b) employ left-hand exits from the first freeway; those in Figures 3-2(a) and (c) are characterized by left-hand entrances to the second freeway; while those in Figures 3-2(d) and (e) both exit from and enter on the right.

Left Exits

The geometric design problems encountered in single lane, and even multi-lane, left hand exits are no greater than for comparable right hand exits. In some cases there may be considerable cost advantabe in providing left hand exits in freeway-to-freeway interchanges. However, studies of the operational characteristics and accident records at interchanges indicate the following potential problems where left hand exits are utilized.

(1) The left lane of a freeway generally contains the fastest moving traffic and frequently the highest lane volume when the freeway is operating at levels of service C or D. Since vehicles anticipating an exit from the freeway have a tendency to slow down prior to entering the deceleration lane and this slow down occurs in the high speed lane, the potential for rear-end collisions and sideswipes

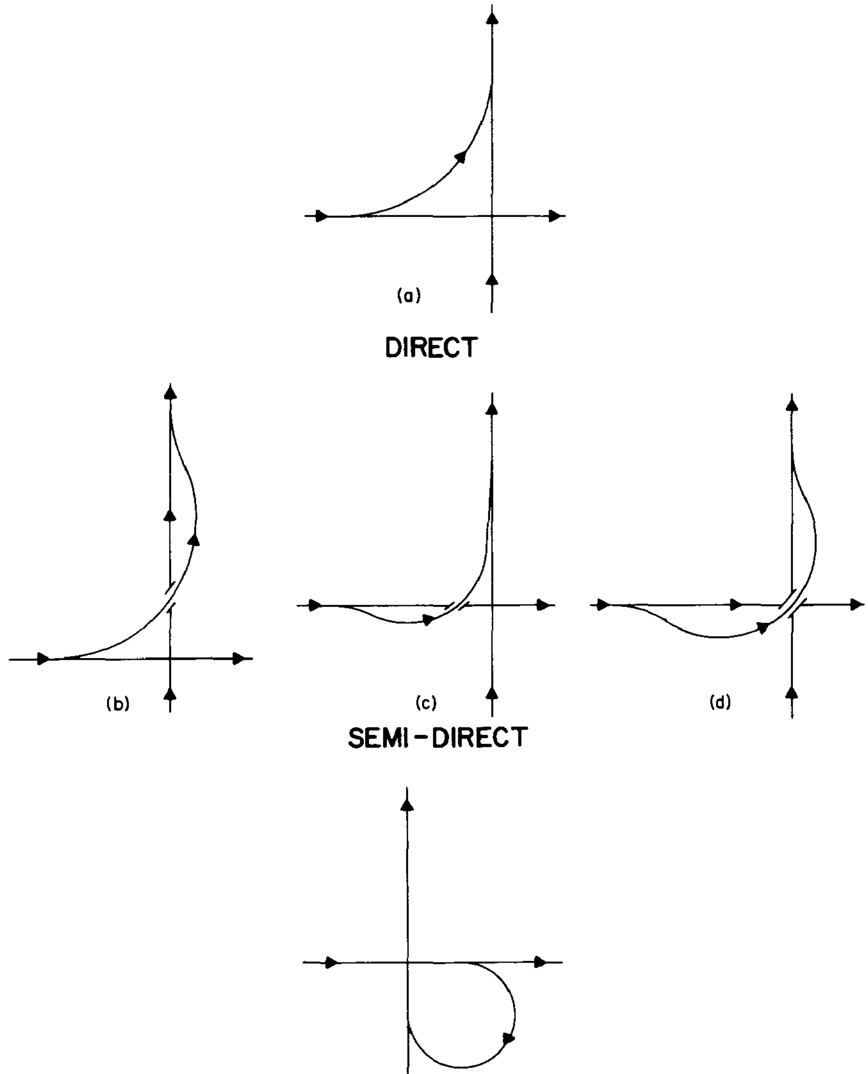


Figure 3-2. Left-Turn Ramp Types

from through traffic moving right to avoid the slowing vehicles is increased.

(2) The slower moving vehicles, particularly trucks, normally travel in the right lane. The left hand exit forces these slower moving vehicles to weave across the faster moving traffic to perform the exit maneuver. With three or four lanes in one direction, this requires several weaving maneuvers. When the approach to the left exit is on an ascending grade, the hazardous effect of the commercial vehicles weaving across the high speed lanes is further accentuated.

(3) The unfamiliar driver frequently does not anticipate left exits, regardless of the advanced signing. The hesitancy associated with the unexpected frequently results in hazardous maneuvers in the vicinity of left exits.

(4) On heavily travelled commuter freeways, where a majority of the drivers are very familiar with the roadway, the left exits do not present any particular problems for these drivers.

(5) Many of the operating problems associated with existing left exits can be related to inadequate advance signing and insufficient sight distance on the approach to the exit. When the left exit is readily visible to the driver at least one half mile in advance of the turn-off, "surprise" effect is reduced considerably.

Synthesis and Recommendations. On the basis of the research literature, workshop discussions, and project questionnaires, the following observations may be made:

(1) Left-hand exits are undesirable for low to moderate volume left-turn movements and should be avoided except where the economic advantage is of considerable magnitude.

(2) For high volume left turn movements requiring turning road-

ways with two or more lanes, especially where the primary numbered route turns left, an exit to the left is an acceptable solution, a major fork configuration being the preferred design.

(3) When left exits are to be incorporated in a freeway-to-freeway interchange, the designer should provide ample signing beginning at least two miles in advance of the exit to warn the driver of the left exit ahead.

(4) A careful investigation of the sight distance in advance of the proposed left exit should be conducted. If the anticipatory sight distance for the design speed of the highway cannot be provided, a right hand exit should be substituted.

(5) A left exit should not be used when the approach to the exit is on an ascending grade which will reduce the normal speed of heavily loaded trucks.

Left Entrances

In some configurations, the use of entrances on the left side of a through roadway may permit elimination of one or more costly structures and result in sizeable savings in cost. However, the poor operational characteristics of left-hand entrances may more than offset the cost advantages in all but a few cases. The principal problem associated with left entrances is the merging and subsequent weaving of the slower moving entering vehicles with the high speed through traffic in the left lane of the freeway.

To the average driver, the most familiar merging or weaving maneuver is for the slower moving vehicle to merge into or weave across the faster moving traffic from the right, using his rear view mirror to locate gaps. Blind spots in his rear vision prevent the driver from locating these gaps when merging from the left. This weaving maneuver is even more difficult for truck traffic because of the lower rate of truck acceleration and the greater relative speeds of the weaving and merging vehicles.

Synthesis and Recommendations. From the workshop discussion and the lierature review, it is recommended that:

(1) Left entrances should be used only when an extra freeway lane is added on the left downstream from entrance to eliminate a forced merge within a short distance, (See Figure 3-4) Left entrances with tapers as shown in Figure 3-3 should be avoided.

(2) Left entrances should not be used when a right exit follows within one mile of the entrance.

(3) The design speed of the ramp entering on the left should be comparable to the mainline speed so the relative speed of the merging vehicles will be reduced to a minimum.

(4) Left entrances should not be used when the entrance ramp or acceleration lane is on an ascending grade.

Loop Ramps

Except in highly developed urban areas, with their high right-of-way costs, the least expensive connection that provides the left turn movement between two freeways is generally the loop ramp. The only "structure cost" on a loop ramp is for the relatively small widening and lengthening of the bridge carrying one freeway over the other. Because loop ramps are less expensive than direct or semi-direct ramps, a great number of loop ramp or cloverleaf interchanges have been constructed throughout the country.

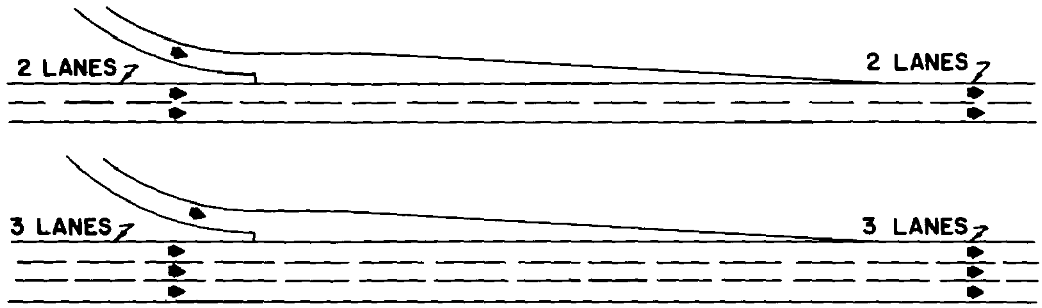


Figure 3-3. Left Entrance with Tapers

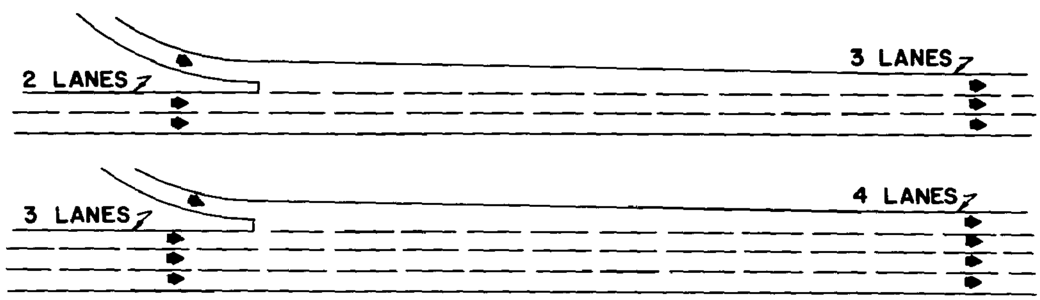


Figure 3-4. Left Entrances -- Lane Added

Large loop ramp radii result in unreasonable right-of-way requirements and longer travel distances while permitting only slight increases in design speed. Thus, the maximum practical loop ramp radius is between 230 and 300 feet. These radii limit the design speed of loop ramps to between 30 and 35 miles per hour. With freeway design speeds of 70 miles per hour and higher, long acceleration and deceleration lanes are required when loop ramps are used for freeway interchanges.

Studies of the driving characteristics on "flattened" loop ramps indicate that where the central arc is of a greater radius than either the initial or final arc, drivers accelerate prematurely on the flat arc and have difficulty in retaining vehicle control at the point of compound curvature with the succeeding sharper radius. Therefore, single radius loop ramp are preferred to multi-radii loops.

Pinnell and Burr observe that the capacity of an isolated single lane loop ramp, as illustrated in Figure 3-5, is approximately 800 vehicles per hour, and that a restriction of 1000 vehicles per hour is placed on the total capacity of two adjacent loop ramps because of the weaving maneuver. However, when a collector-distributor road is used, as shown in Figure 3-6, the weaving capacity increases to about 1,500 vehicles per hour. These capacity values limit the locations where loop type ramps, with or without collector-distributor roads, may be utilized.

Recommendations.

(1) Two-lane loop ramps should not be considered for freeway-to-freeway interchanges.

(2) Isolated loop ramps for minor turning movements may be

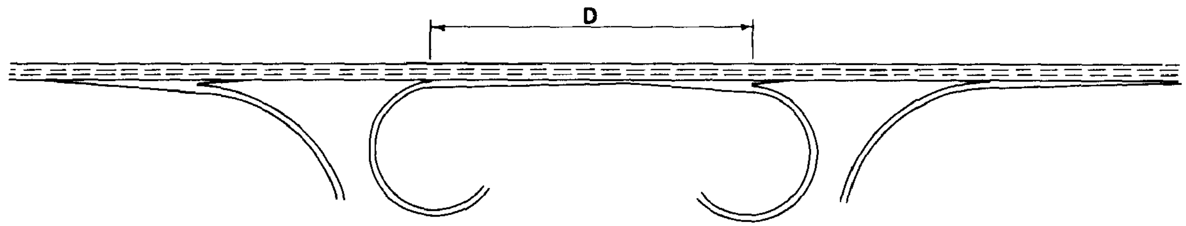


Figure 3-5. Adjacent Loop Ramps

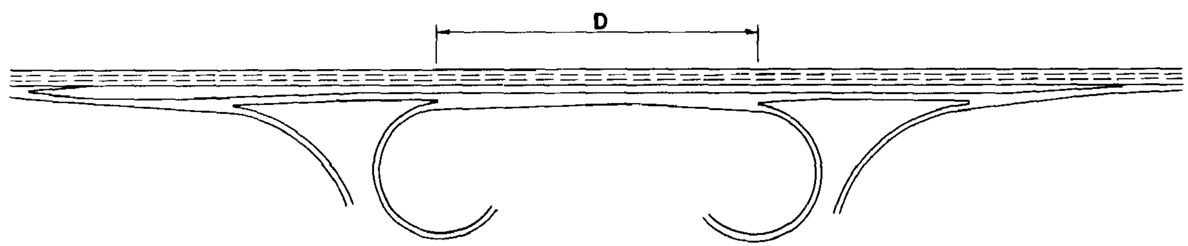


Figure 3-6. Loop Ramps with C-D Roads

utilized in high volume major interchanges provided adequate-acceleration and deceleration lanes can be provided for the lower design speed.

(3) All loop ramps should have a constant radius as multi-radii loops result in poor driving characteristics.

(4) With two adjacent loop ramps, the nose to nose distance should be as long as possible to provide adequate weaving length. See Table 3-4 for minimum and desirable values.

(5) Cloverleaf interchanges should only be used in rural areas, and preferably should have collector-distributor roads throughout.

Exit Ramp Configurations

When both right and left turning movements are to be provided at an interchange, the turns may be taken off the mainline separately (examples of which are seen in Figures 3-7(a), (b), and (c)); or the turns may be taken off together and followed by a fork, as shown in Figure 3-7(d). This fork may be part of a collector-distributor road in a cloverleaf interchange, or the left branch may lead to a semi-direct left turn roadway. A left hand exit for the left turning movement is a special case of the two exit configuration and has been discussed previously. Only under extreme right-of-way constraints would a single exit for both movements take off from the left of the through freeway lanes.

With the two-exit design, the exiting driver must make two decisions while in the high speed freeway traffic: first, he must decide that he wants to exit at this interchange· and second, he must decide on whether to go left or right. With the single exit design, the first decision is made on the freeway and the second decision on the

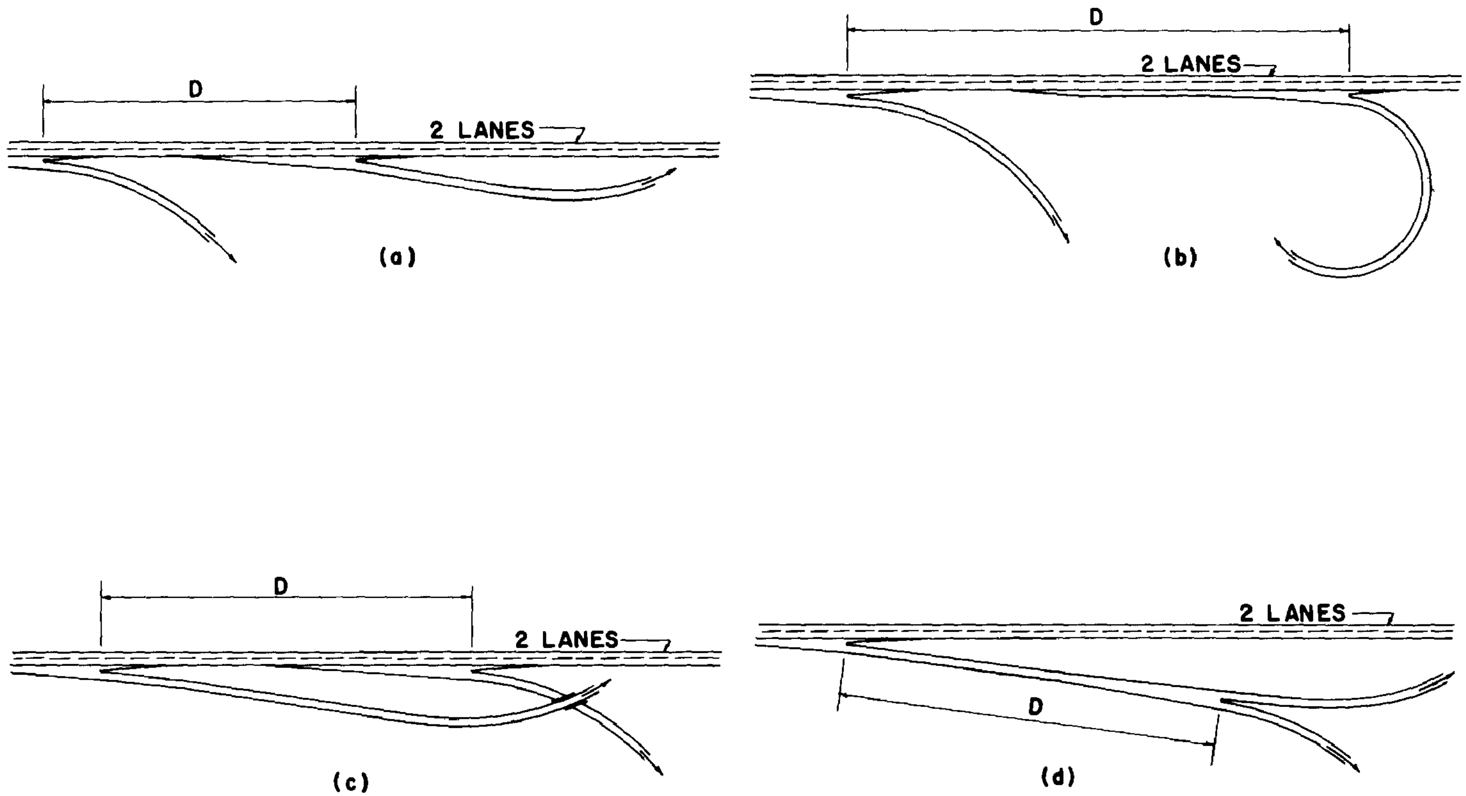


Figure 3-7. Exit Configurations

slower moving exit ramp. The single exit configuration reduces the number of signs the driver must read and process while in the high speed traffic stream. Figure 3-8 illustrates typical signing for single and double exits.

With the two-exit configuration, the order of the exits may be either right turn followed by the left turn, as in Figure 3-7(a) and (b), or left turn followed by right turn, as in Figure 3-7(c). The right turn exiting first is the most frequently used order, but the left turn exiting first has been used at various locations throughout the country with varying degrees of success.

The distance between successive ramp terminals, shown as dimension 'D' in Figure 3-7, is of critical importance for safe and smooth traffic operating characteristics. The AASHO Policy on Geometric Design of Rural Highways, 1965, recommends values for 'D' as shown in Table 3-4.

TABLE 3-4

DISTANCE BETWEEN SUCCESSIVE RAMP TERMINALS

Design Speed, mph	40-50	60-70	80
Distance D - feet			
Minimum	400	500	900
Desirable	700	900	1200

Several of the state design manuals base the minimum distance between successive exit ramps terminals on the minimum needed for adequate signing, sight distance, and capacity.

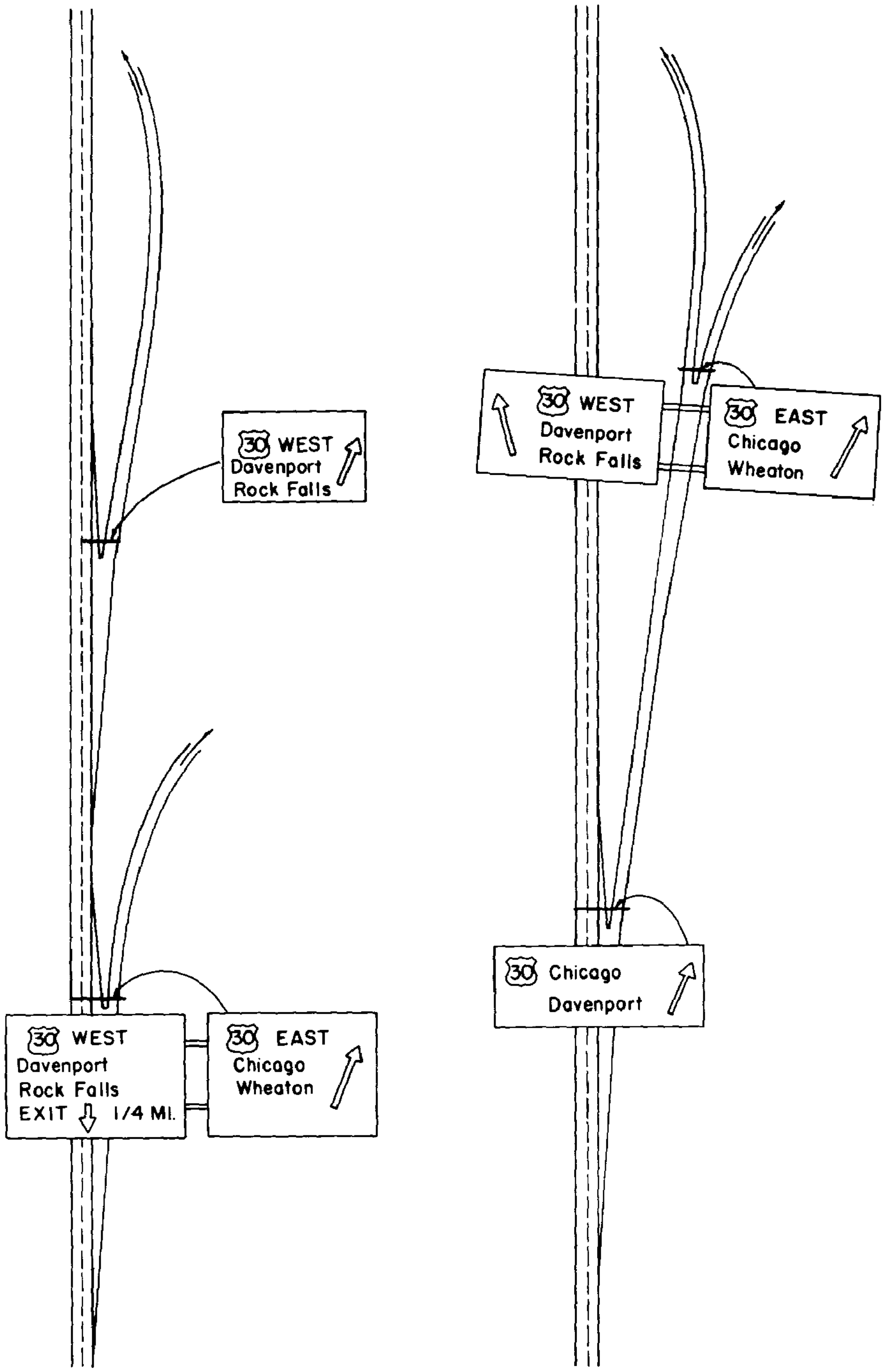


Figure 3-8. Signing: Single & Double Exit

Synthesis and Recommendations. Based on the literature review and workshop discussion, it is concluded that:

(1) The single exit design has some superior operating characteristics over the two exit design, even when a two-lane exit is required for capacity.

(2) Signing is greatly simplified with the single exit configuration, resulting in less driver confusion and hesitation.

(3) Applied to the cloverleaf, the single exit design usually results in safer driving conditions, since the weaving section is removed from the mainline and located on a collection-distribution road.

(4) However, there is nothing inherently wrong with a properly designed two-exit configuration, particularly in high volume interchanges. Therefore, each individual site requires an investigation to determine the most desirable exit ramp configuration.

(5) On double exit configurations, the right turn exit should, as a general rule, precede the left-turn exit, since this arrangement conforms to the expectancy of the unfamiliar driver. With adequate directional signing and ample sight distance (so that the configuration of the off-ramps is readily visible to the driver), designs with the left-turn ramp first can operate satisfactorily.

(6) The minimum distance between successive ramp terminals should be approximately 800 feet, with distances in the order of 1200 feet being desirable.

Entrance Ramp Configurations

With left and right turning movements from an adjoining freeway, the entrance to the through roadway may be either two consecutive entrance ramps as shown in Figure 3-9(a) or the two turning movements

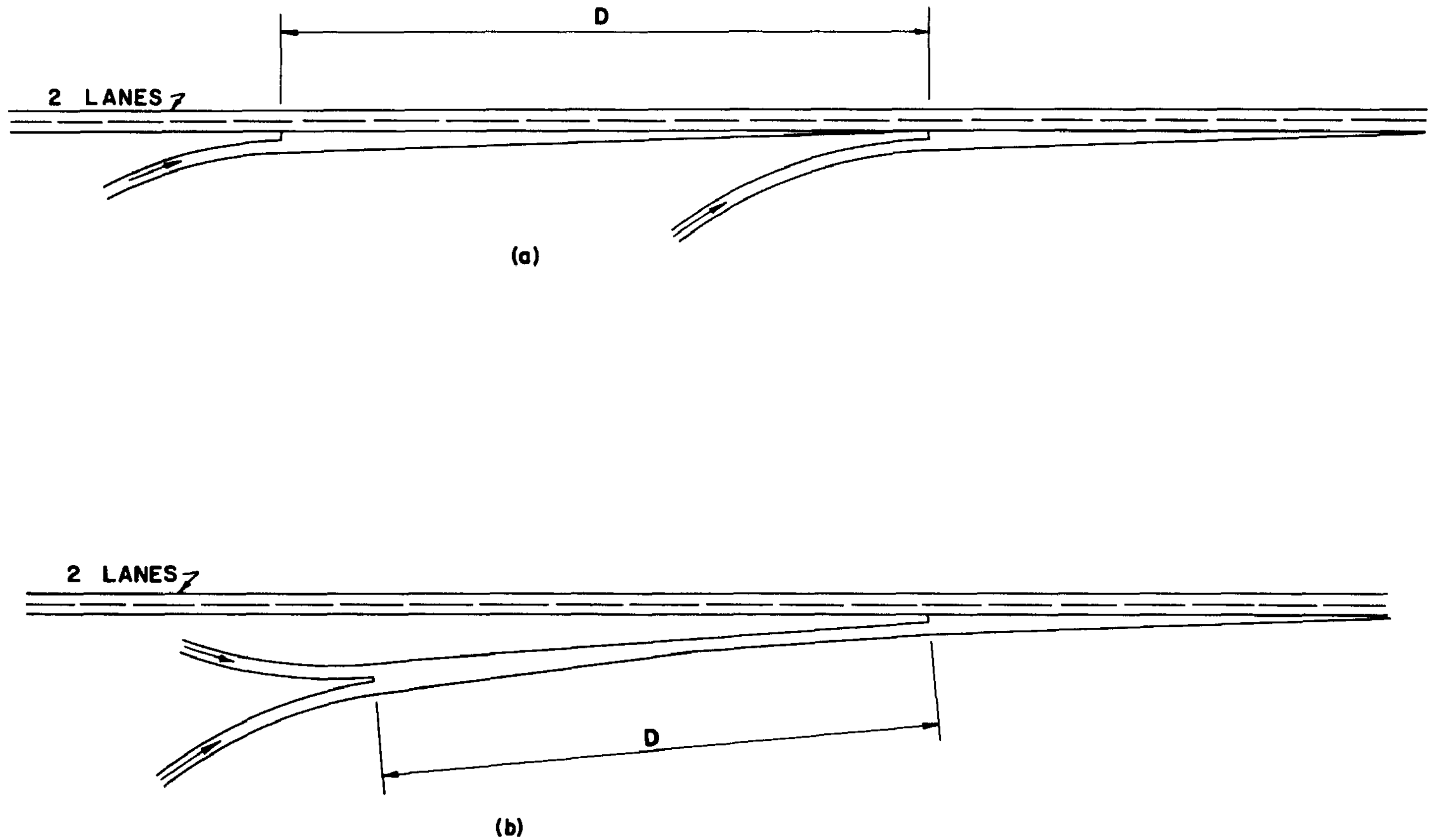


Figure 3-9. Entrance Configurations

may be merged into a single ramp with one entrance as indicated in Figure 3-9(b). A left-hand entrance for the left turning movement is a type of a two entrance configuration and has been previously discussed.

With the double entrance configuration, the order in which the two entrances occur is immaterial since the driver is only concerned with continuing ahead on the freeway.

Under light traffic conditions on the freeway and low turning volumes on both the turning roadways, the drivers generally encounters no difficulty in negotiating the merging maneuvers with either of the entrance ramp configurations illustrated in Figure 3-9. When the freeway and ramp traffic increase, the capacity problems associated with entrance ramps become a factor in the design. If no auxiliary or additional lane is added to the freeway downstream from an entrance ramp, the facility with which the entering traffic can merge into the traffic stream is a function of the entering volume and the lane-1 volume. With the two entrance ramp configuration, the lane-1 volume upstream of the second entrance may be too great to permit the entering vehicles to properly merge unless there is sufficient distance between the two entrances to permit vehicles entering at the first entrance to move out of lane-1 before reaching the second entrance. In order to determine the feasibility of providing two adjacent entrances and the distance between the successive ramp terminals, a capacity analysis of each individual case is necessary.

The distance 'D' in Figure 3-9, between successive ramp terminals, recommended by AASHO, is the same as for exit ramps, shown in Table 3-4. Most state design manuals specify the distance required for adequate capacity. Pennsylvania uses a minimum value of D = 1000 feet for the configuration in Figure 3-9(a) and D = 600 feet for the configuration in Figure 3-9(b), only if these distances exceed that required for capacity.

Synthesis and Recommendations. The principal conclusions, based upon the workshop discussions and questionnaire responses include the following:

(1) Whenever construction is feasible, a single entrance configuration is more desirable than a double entrance design, provided the single lane entrance has adequate capacity for the total turning traffic.

(2) When the combined turning traffic volumes require a two-lane entrance, a single entrance configuration is superior to a double entrance design only when another lane is added to the freeway.

(3) Properly designed with adequate distance between the successive entrances, the double entrance configuration will operate in a satisfactory manner.

(4) The criteria that determine whether a single or double entrance configuration will be used are generally the traffic volumes, available space, and construction cost.

(5) The distance between successive entrances in the double entrance configuration should be based upon a capacity analysis of the entering and lane-1 volumes for each entrance, but the distance should never be less than 1100 feet and preferably should approach 1800 feet.

Weaving Sections

The current procedure for the design of weaving sections used by the majority of states is outlined in the Highway Capacity Manual (1965) and the AASHO "Blue Book" (1965) for analyzing ramp capacity and weaving sections requirements. Research on the operational characteristics of weaving sections being conducted by Pignataro et al. at the Polytechnic Institute of Brooklyn indicates that these procedures are not producing reliable results in Levels of Service when applied to basic weaves and ramp weaves. Because of the complex dimensions of the weaving problem, further refinement of the design process is required in the future.

The weaving process is one of the least comprehended driving operations by the average driver and when confined to a restricted distance, generally results in congestion and delay except under very low traffic volumes. When a weaving section is introduced into a major interchange, it adds another distraction to the driver already concerned with merging and diverging maneuvers, points of decision, and directional signing. Several states restrict the use of weaving sections in major interchanges to reduce traffic conflicts and to avoid impairment to interchange flow. The Illinois Design Manual states: "While weaving sections simplify design, their use should be restricted to minor interchanges with relatively small weaving volumes, since turbulent effect of weaving operations can result in reduced operating speeds and capacity for through traffic." In Pennsylvania, weaves are permitted if a sufficiently high level of service can be provided; otherwise, weaving is eliminated from the through roadway by use of collector-distributor roads or by alternative ramp configurations.

Synthesis and Recommendations. The major conclusions, based upon the literature review and the workshop discussion, include:

(1) Weaving sections in major interchanges add one more distraction to the driver, already concerned with directional signs, entering and exiting maneuvers, and vehicles changing lanes.

(2) Weaving sections contiguous to the mainline roadway should not be permitted in major interchanges except in the case of very low volume rural interchanges and where the mainline also has a peak hour traffic volume of less than 1,000 vehicles per hour.

(3) Where the mainline peak hour volume exceeds 1,000 vehicles per hour, the weaving sections in cloverleaf interchanges should always be on collector-distributor roads.

(4) Mainline weaving sections created by adjacent local upstream entrances and downstream exits should be carefully investigated and eliminated wherever possible.

(5) Major interchanges should be spaced a minimum of 2 miles apart to reduce the mainline weaving section between them, particularly on high volume freeways. Offset T interchanges, as shown in Figure 3-10, should be avoided.

Lane Balance and Lane Drops

The number of lanes to be provided in each direction on a freeway is based upon the capacity of a lane and the anticipated peak hour traffic volume. In freeway construction, a minimum of two lanes are provided in each direction and it is generally not considered desirable to utilize more than four lanes directional except where an auxiliary lane is added for a short distance. Where a major change in the traffic volume occurs at a freeway entrance or exit, it may

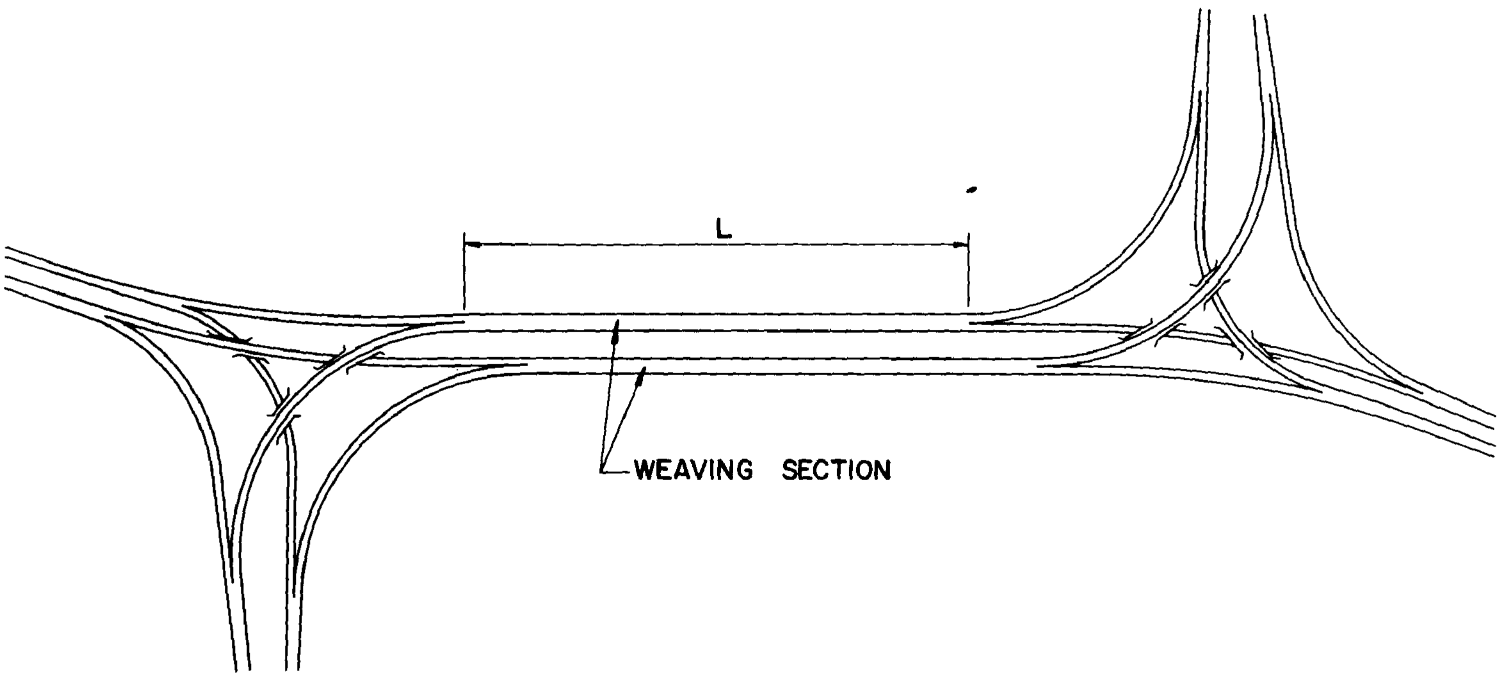


Figure 3-10. Offset T Interchange

be desirable to add or drop a freeway lane for lane balance.

There is general agreement among design and operations engineers with the four basic principles for lane balance outlined in the AASHO "Red Book" (1957):

(1) Highway lanes should not be reduced by more than one lane at a time.

(2) The highway beyond a two-lane ramp entrance should be at least one lane wider than its approach to the entrance.

(3) The number of lanes beyond the merging of two traffic streams should not be less than the sum of all lanes on the merging highway minus one.

(4) The number of through lanes beyond the ramps of a two-lane exit should be one less than the number of lanes approaching the ramp.

Although these principles are generally accepted, there is no agreement in the design community as to the details of how they should be implemented. This disagreement concerns whether a lane should be dropped within an interchange or some distance beyond the interchange; which lane should be terminated; and the geometrics of the taper where the lane is dropped.

The California Planning Manual of Instructions states that lane reductions are not permitted within local interchanges except at multiple lane exits where more than half of the traffic turns. Where traffic volumes decrease sufficiently to warrant a lane drop, the recommended location for the drop is beyond the influence of the interchange and preferably at least one-half mile from the nearest exit or entrance. Further, it is preferred that lane drops be located on tangent alignment with a straight or sag profile for maximum visibility of the merge markings.

The pitfalls of poorly designed lane drops are noted in several studies. Hong (1966) observes that ". . . freeway egress and ingress points must be designed so as to eliminate any supicious pocket, trap area, surprise element, or system discontinuity. . . . Congestion at a bottleneck can cause low over-all speed, high frequency of speed changes, time loss, driver discomfort, and, above all, drastic reduction in capacity."

A paper by Jenkins (1969) indicates that lane drops should be situated at major diverging forks in directional interchanges. Jenkins also observes that at directional interchanges where turning movements are heavy and separate exits for right and left turns are provided ahead of entrance roadways, it may be satisfactory to drop a lane within the interchange, preferably at the second exit. Jenkins further states that under normal circumstances, when a lane is to be dropped in the vicinity of a non-directional interchange, the lane should be carried beyond the interchange and then terminated. With regard to the choice of which lane to drop -- left or right -- Jenkins indicates there is no conclusive evidence available to support one or the other. When Mr. Jenkins' paper was discussed at the workshop, some conferees indicated the location of each lane drop should be resolved on the basis of its own unique merits.

Synthesis and Recommendations

The major conclusions, based upon the literature review and workshop discussions, include:

(1) When the downstream traffic volume justifies a reduction in the number of through traffic lanes at a major interchange, the preferred location for the lane drop is beyond the influence of the

interchange. The alternative of dropping a lane immediately beyond the major exit terminal may be utilized when economic considerations dictate.

(2) There should be no reduction in the number of lanes through the interchange, regardless of the through and turning traffic volumes.

(3) When a lane is to be dropped at a major exit to the right, the right through lane should always be dropped.

(4) When a lane is to be dropped beyond the influence of the interchange, the right lane is the preferred lane to be dropped; but the left lane may be dropped, particularly where a future continuation of the left lane is contemplated. An interior lane should never be dropped.

(5) The most important considerations in designing lane drops are to provide adequate visibility of the lane drop configuration and inform the driver of the impending situation. Therefore, lane drops should be on tangent alignment, preferably on sag vertical curves, and ample advanced signing should be provided.

(6) The taper at lane drops should be designed as acceleration lanes with a minimum taper ratio of 50:1.

Route Continuity

Route continuity is defined by Jack E. Leisch as "the provision of a directional path along and throughout the length of a designated route." The designation pertains to a route number or to a name of a freeway. One of the principal advocates of route continuity, Mr. Leisch has lectured on the topic in the "Dynamic Design for Safety" seminars held throughout the country in 1971-1972.

The basic premise of route continuity is to keep the unfamiliar driver "on line" by designing the interchange geometric configuration to favor the "through" route rather than the heavy traffic movement. It is urged that all entrances and exits from the through route be to the right regardless of traffic volume, so that the median lane always follows the designated route without interruption. Thus, the only time a left exit is recommended is when the through numbered route turns left. Figures 3-11(a), (b), and (c) illustrate three hypothetical interchanges where route continuity has been provided while Figures 3-11(d), (e) and (f) indicate comparable interchanges where continuity has been disrupted. In each case the through route has been designated Interstate Route 2.

The important feature of route continuity is that the through drivers, particularly those unfamiliar with the area, always take the left roadway at every point of decision in order to remain on the through route. This reduces the number of lane changes and hazardous maneuvers, easing the driving task. It is argued that route continuity permits the driver to operate with greater confidence and reduces the elements of surprise and indecision.

An opposing theory is that of volume continuity, in which the heaviest traffic volume is given the preference in the design of an interchange. For example, if the heaviest traffic volume in Figures 3-11(a) and (d) is northbound from Interstate 2 to U.S. 11, (d) would be the preferable scheme using volume continuity. It is Mr. Leisch's contention that since the heavy volume will most likely be the commuter drivers who are familiar with the end, the preference should be given to the stranger trying to follow a designated route.

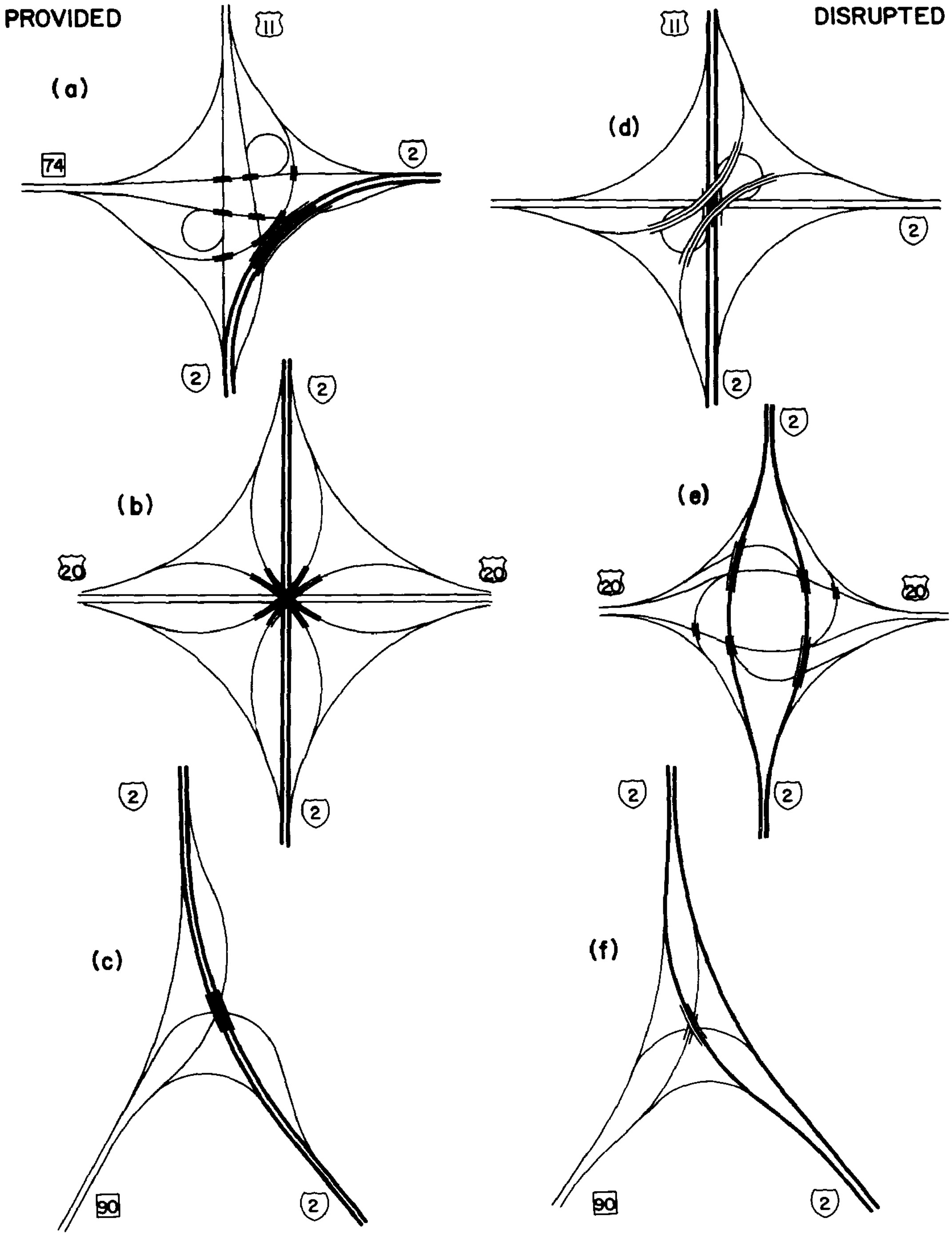


Figure 3-11. Route Continuity

Synthesis and Recommendations

The major conclusions on route continuity, based upon the literature review and workshop discussion, include:

(1) Route continuity has little relevancy to the repeat or commuter type driver who is familiar with the roadway. These drivers, knowing the geometric configuration of the approaching interchange, position themselves in the proper lane to make their desired maneuver.

(2) Unfamiliar or route oriented drivers must rely upon signing and the visible aspects of an interchange to select their proper maneuver. If route continuity were universally observed, these drivers would know to keep left for the through route or right to change routes.

(3) Although the percentage of drivers in the traffic stream who are unfamiliar with local conditions may be relatively small (5 percent or less), designing interchanges using the route continuity concept will improve operating characteristics and reduce accidents. Route continuity provides for simplification and uniformity of interchange signing which results in less confusion for the driver.

(4) The existing route numbering system in use in the country is not always conducive to good route continuity design in that the designated routes are not necessarily the most advantageous route for the through driver.

(5) A strict adherence to the route continuity principle precludes any subsequent change in route designations. A change in route numbers voids the route continuity concept.

(6) When the turning traffic volume is large (i.e., greater than 70 percent of the total traffic), it may be desirable to design for the heavy traffic movement rather than the designated route if in so doing there is an improvement in the interchange geometrics.

Exit Terminals

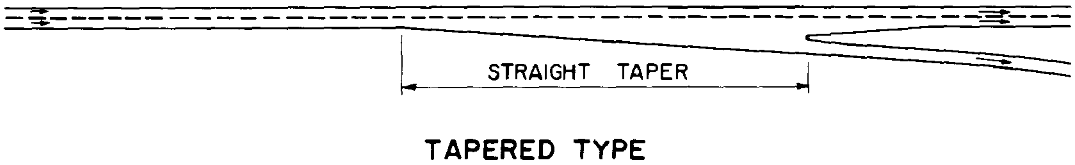
In order for a driver to change from one freeway to another it is necessary to exit from the mainline traffic flow to a ramp or turning roadway. In most cases the design speed on the ramp is lower than on the mainline. Thus, the exiting maneuver requires a decision by the driver to leave the mainline, a deceleration to reduce travel speed, and a turning of the vehicle to change direction.

The geometric design of an exit terminal is comprised of two parts: a deceleration lane, and a nose offset from the edge of the through pavement. The factors to be considered in the design of exit terminals include speeds, traffic volumes, capacities, curvature, grades, sight distance, and psychological factors such as driver expectancy. Depending upon the traffic volume on the turning roadway, the exit terminal may be one, two, or even three lanes in width.

Deceleration Lanes - Single Lane Exits

There are two general types of deceleration lanes in use today for single lane exits, the taper type and the parallel type (Figure 3-12). Each of these types has its application and highway designers have diverse opinions as to their appropriate use.

Taper Type. The tapered type deceleration lane follows closely the path traversed by the majority of exiting drivers and does not encourage maneuvering through a reverse curve path. AASHO suggests that the tapered deceleration lane should make an angle of between 4° and 5° with the through pavement. Most states conform to these angles although a few use considerably flatter angles and therefore longer tapers. For low to



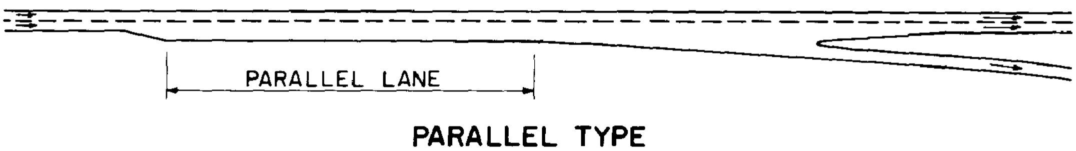


Figure 3-12. Deceleration Lane

medium traffic volumes, tapered deceleration lanes generally operate completely satisfactorily. The two major exceptions occur where: (1) the turning roadway exits to the right from a through roadway which curves to the left; and (2) when the exit occurs on or just beyond a crest vertical curve. In the first case, when visibility conditions are restricted there is a tendency for the through driver to be inadvertently led onto the ramp. This condition can be partially corrected by using contrasting pavement for the deceleration lane, or, heavy, dashed pavement markings along the entire length of the deceleration lane. When an exit occurs on a crest vertical curve, the tapered type deceleration lane does not provide sufficient advanced warning of the exit unless the vertical curve is designed for the anticipatory sight distance.

Parallel Type. The parallel deceleration lane provides a full lane adjacent to the through roadway in advance of the exit. When the full lane begins abruptly, the initial part of the lane is not intended for traffic use, but serves as a target for the driver, advising him that an additional lane has been added. Frequently, a short taper is provided at the beginning of the parallel lane; this in effect extends the dead lane length with very little additional pavement. The parallel deceleration lane permits the driver to move out of the through roadway before slowing down for the turning roadway and also advises him that he is approaching an exit. When fully utilized, a parallel deceleration lane requires a reverse curve travel path which many drivers find objectionable. The lengths of deceleration lanes are a function of the change in design speed between the mainline and the turning roadway and are shown in Table VII-10 in the AASHO Blue Book.

Deceleration Lanes - Multi-Lane Exits

When the volume of traffic on a turning roadway requires more than a single lane, lane 1 (the outside lane of the through roadway) may not have sufficient capacity for all the turning traffic. Therefore, some turning vehicles will be in lane 2 as they approach the deceleration lane and some through vehicles, primarily slower trucks, may remain in lane 1. Under these conditions it is essential to use a parallel type deceleration lane of sufficient length to permit the necessary lane changes to take place well in advance of the point of divergence. Figure 3-13 indicates a typical multi-lane exit deceleration lane. On high volume freeways requiring more than two lanes directional and where more than 35 percent of the mainline traffic is turning traffic, the exit terminal becomes a major fork.

Nose and Gore Area Design

The geometric design of the exit terminal nose and the treatment of the gore area between the ramp and through traffic lanes is intended to accomplish three principal goals: (1) to provide a clear indication of the point of divergence of the ramp; (2) to provide an escape zone for the errant vehicle; and (3) to eliminate or reduce the potential danger of fixed obstructions in the path of the errant vehicle. The current design practice of providing flush, high-type, paved shoulders, both left and right of all mainline roadways and ramps has influenced the design standards for exit terminal nose and gore area design throughout the country. Figure 3-14 illustrates four designs currently in use in different states. In the top two illustrations the gore area is paved with the same material as the ramp and mainline pavements with or without

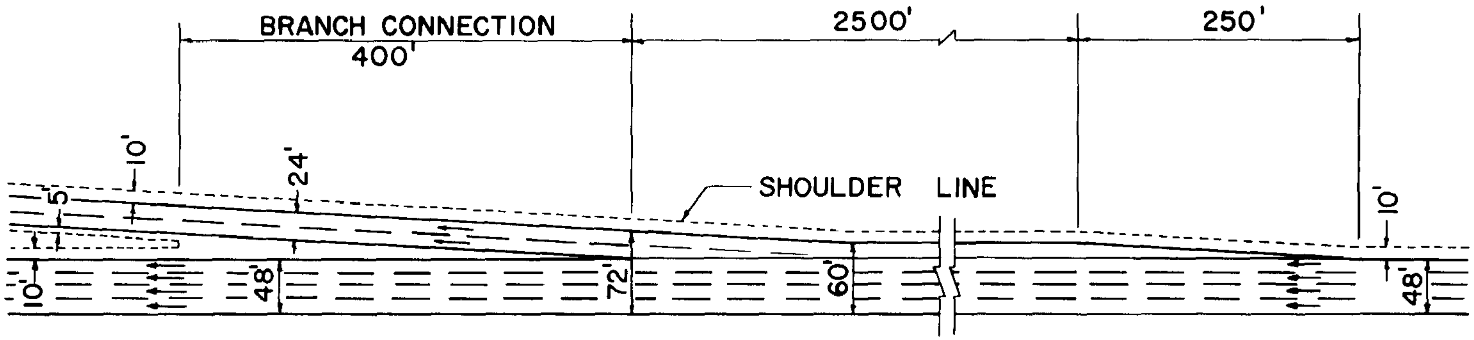


Figure 3-13. Parallel Deceleration Lane: Two-Lane Exit

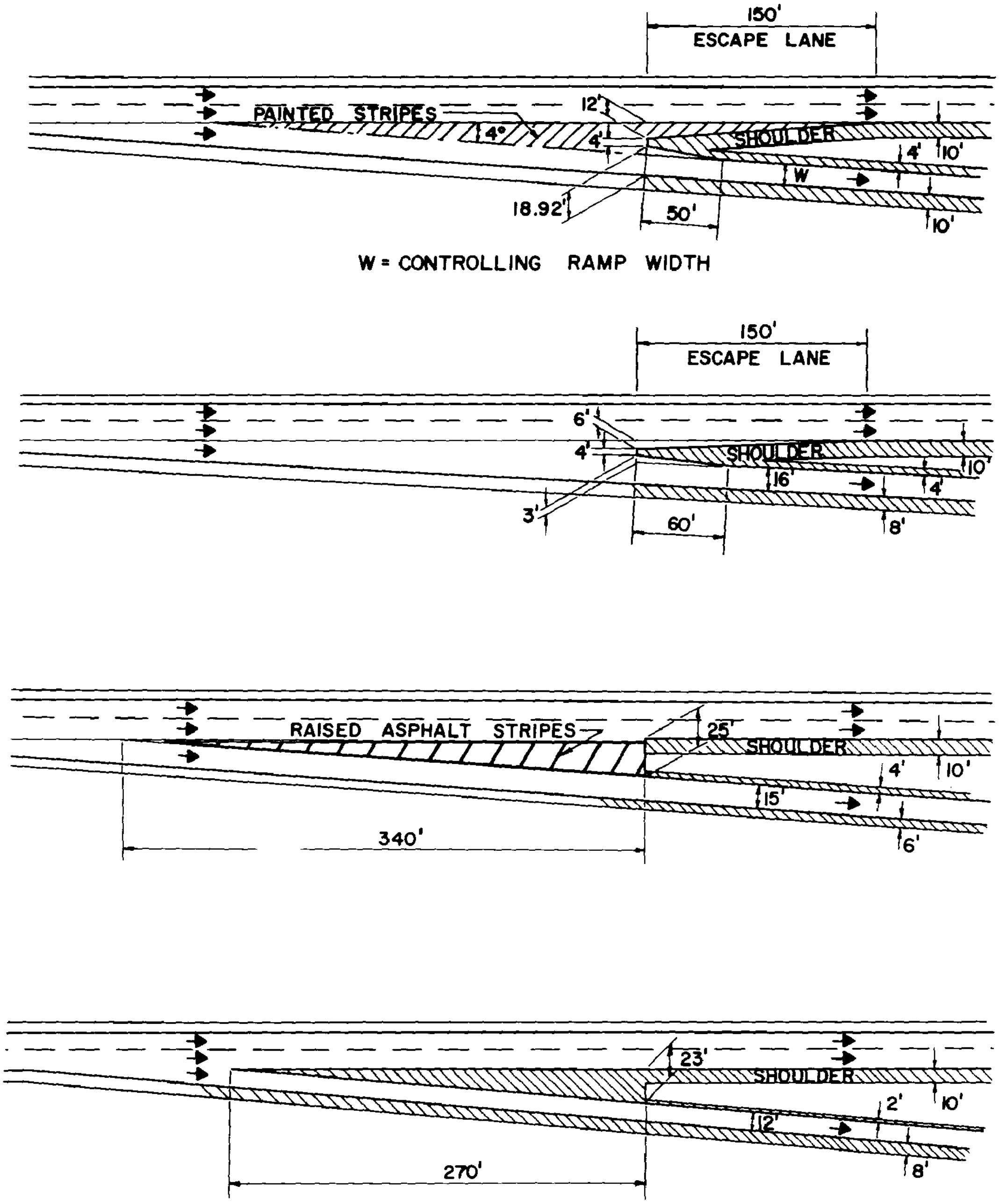


Figure 3-14. Exit Terminal Nose and Gore Area

painted cross hatching. An escape lane is provided beyond the nose and the nose is offset from the edge of the ramp.

The bottom two illustrations show the gore area paved with the same material as the paved shoulders and no specific treatment of the nose. In these latter two designs, the shoulder area is used by the errant driver to return to the mainline. The graded area beyond the nose, in all cases, should be maintained relatively flat and free from obstructions, other than break-away signs, to reduce the hazard to those drivers who fail to properly negotiate the exiting maneuver.

A special condition exists where it is necessary for the exit terminal to be located on a structure. Because the structure parapets present a major fixed object, the design of the nose should provide adequate area for the installation of an energy absorption device to reduce damage to vehicles colliding with the nose. Figure 3-15 shows a typical gore treatment on structures.

Synthesis and Recommendations

Major conclusions, based on the research literature and the workshop discussions, include:

(1) There is no one standard design for exit terminals which will satisfy all conditions and locations and meet all the criteria for safety, operational characteristics, and cost-effectiveness.

(2) The tapered deceleration lane provides the ideal exit terminal for a single lane ramp when the geometrics of the exit ramp are clearly visible to the approaching driver and he recognizes he can maintain a relatively high speed on the ramp. The natural path of most drivers at exit ramps under free-flowing, high-speed traffic conditions where ramp speeds can be maintained at or above 40 mph is easily accommodated by the tapered deceleration lane. Under the above conditions, the added

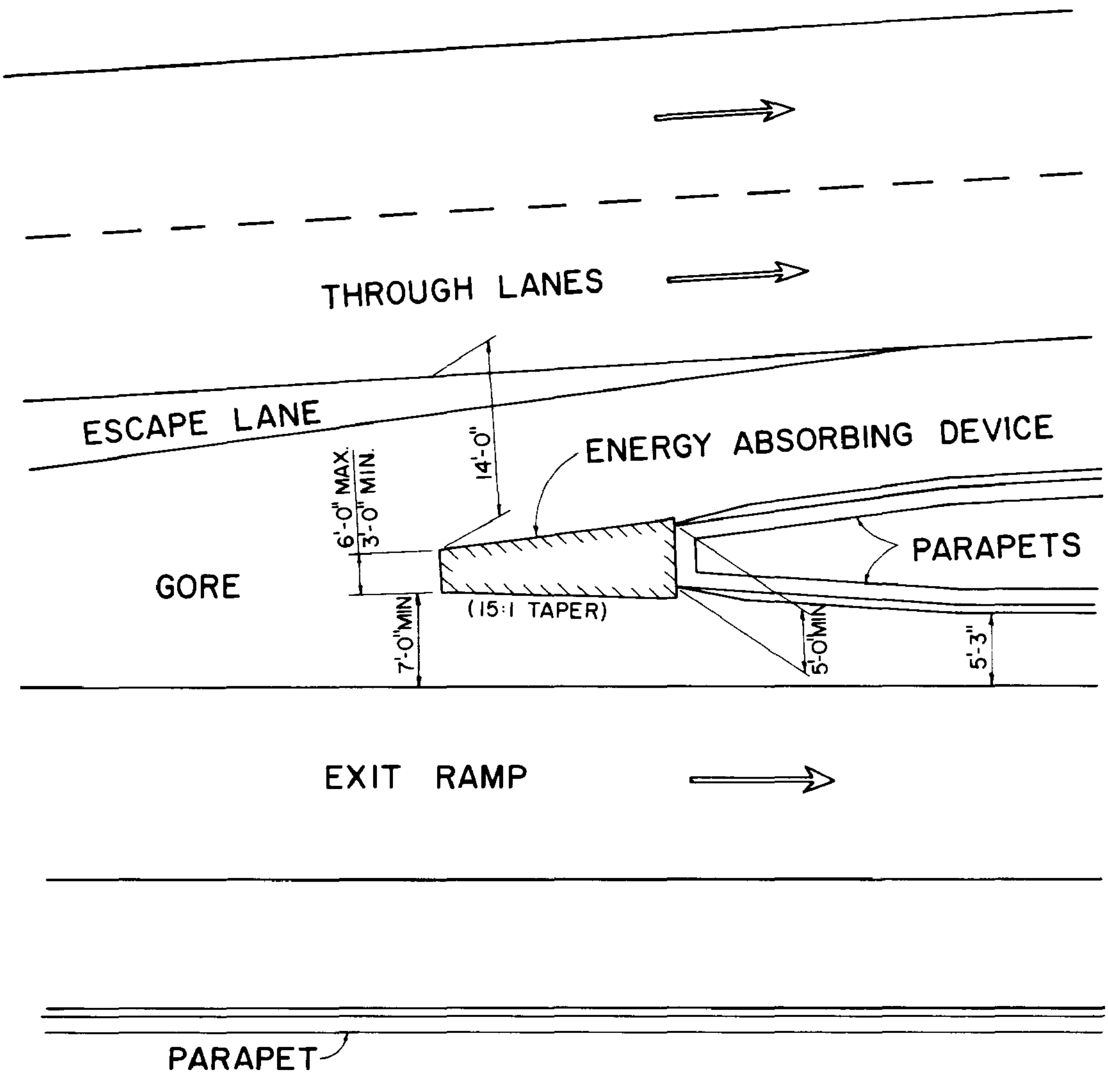


Figure 3-15. Gore Treatment on Structures

pavement in a parallel deceleration lane would receive little useage and the additional cost cannot be economically justified.

(3) Conversely, when the exit geometry is not readily visible to the driver well in advance (e.g., over a crest vertical curve or on the outside of a horizontal curve), or when a major reduction in design speed occurs on the ramp, a parallel deceleration lane aids the driver in identifying the proper exiting and through pathways. It also provides an opportunity for the turning driver to move out of the main traffic stream. This is particularly desirable when the through roadway is operating at low levels of service. An initial width of 4 to 6 feet, followed by a taper to full width, could be used to provide desired target value.

(4) In the case of multi-lane exits, a parallel deceleration lane should be utilized to permit at least one lane of turning traffic to move out of the through traffic lanes well in advance of the gore area.

There is divided opinion on the geometrics required to provide good operational characteristics at the ramp nose and gore area. The essential elements to be incorporated into the design are: (1) to provide a smooth transition from the deceleration lane to the ramp, (2) to provide a feasible route for the driver who inadvertently enters the deceleration lane to return to the through roadway, (3) to provide a nose area which will be "forgiving" to the driver who fails to negotiate the exiting maneuver properly.

The above considerations dictate the following principles in the design of ramp noses and gore areas:

(1) The exit ramp itself, whether preceded by a tapered or parallel type deceleration lane, should diverge from the mainline at an angle of 2° to 5°.

Acceleration Lanes - Single Lane Entrance

The two principal types of acceleration lanes for single lane entrances are the long flat taper and the parallel lane followed by a short escape taper as shown in Figure 3-16. Each of these types have their application.

Taper Type. The tapered type acceleration lane is the standard in most of the state design manuals since the taper is the natural vehicle path under light traffic conditions. When a heavy traffic volume in the right freeway lane does not provide adequate gaps to permit the merging maneuver, the tapered acceleration lane requires the driver to reduce speed or even stop at the beginning of the acceleration lane. When an acceptable gap appears, the driver still has the full length of the acceleration to perform the merge to the freeway lane.

For freeway conditions, AASHO recommends a taper ratio of 50:1 for acceleration lanes which provides a convergence angle of one degree ten minutes. Most of the state design manuals specify a 50:1 taper for freeway acceleration lanes although a few states utilize a one degree convergence angle corresponding to a 57.3:1 taper.

The length of the tapered type acceleration is dependent upon several factors other than the taper ratio: (1) the offset between the outside edge of the freeway lane and the inside edge of the ramp at the entrance nose; (2) whether curvature is provided on the acceleration lane beyond the nose; and (3) the geometry of the downstream end of the taper. Figure 3-17 illustrates several typical designs used by various states. For entrances to freeways, most states provide a minimum of 900 feet of acceleration lane for high speed turning roadways and greater lengths, conforming to the AASHO recommendations, for lower speed ramps. It should be noted that the total length of the accelera-

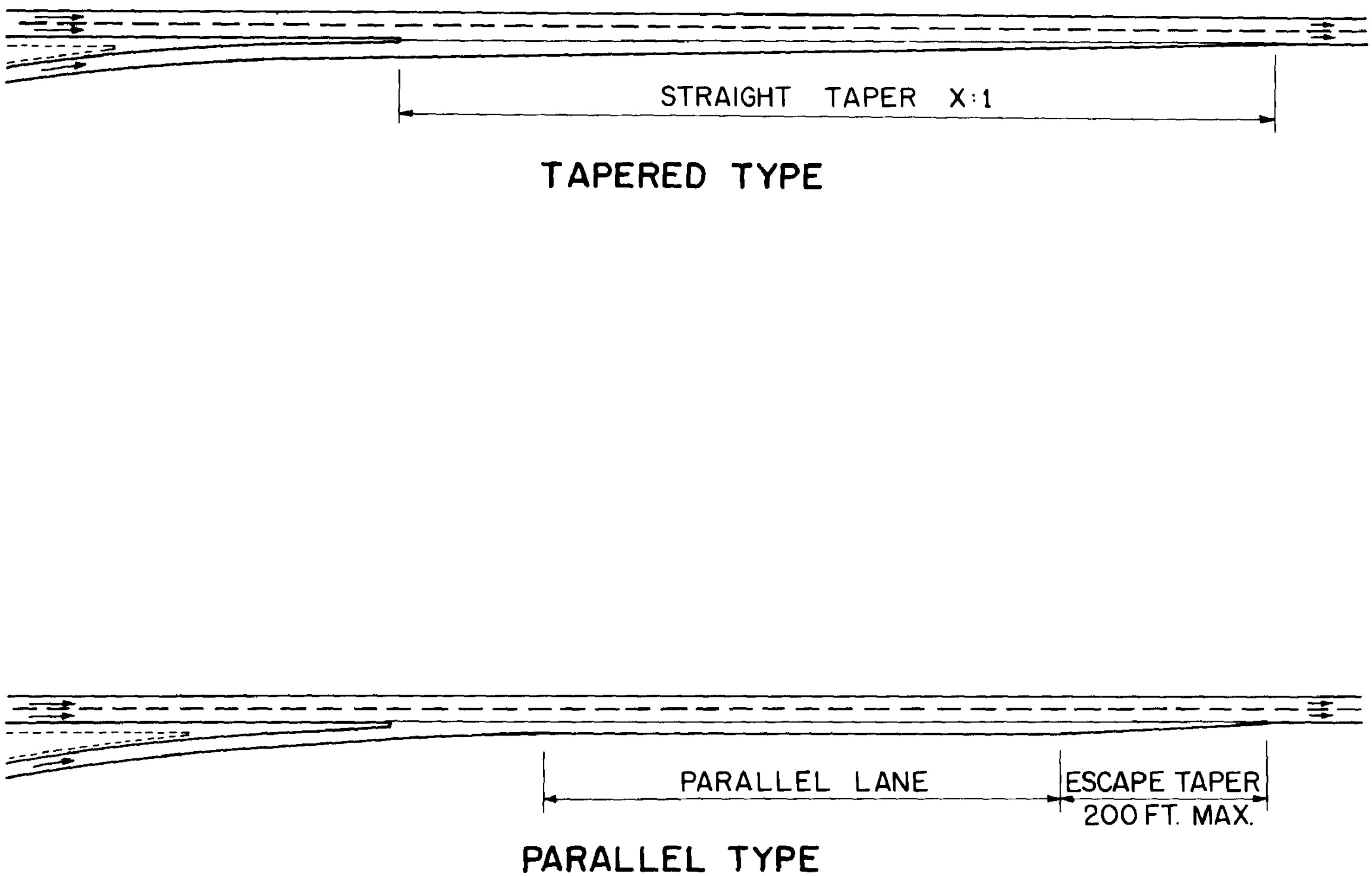
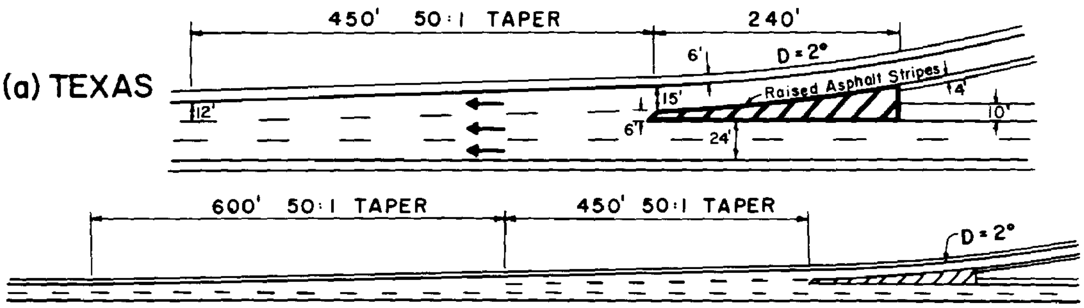
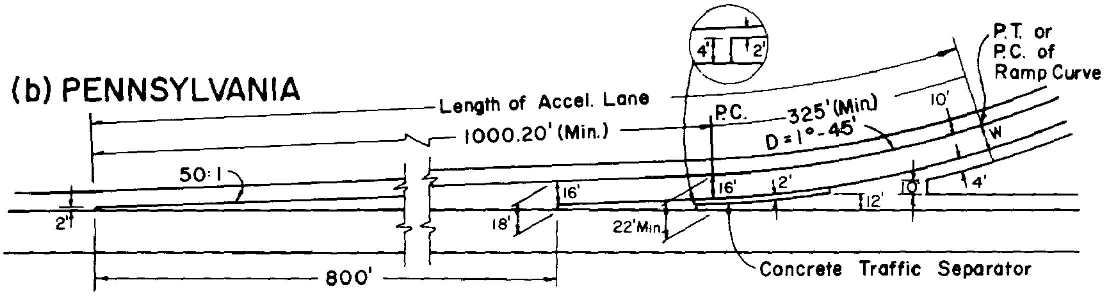


Figure 3-16. Acceleration Lanes

(a) TEXAS

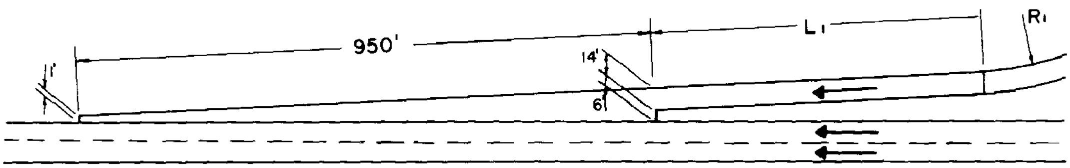


(b) PENNSYLVANIA



(c) ILLINOIS

Design Speed-70		Design Speed-60		Design Speed-50	
R_1	L_1	R_1	L_1	R_1	L_1
760'	100'	600'	100'	465'	100'
600'	200'	465'	200'	350'	200'
465'	300'	350'	300'	250'	300'



(d) CALIFORNIA

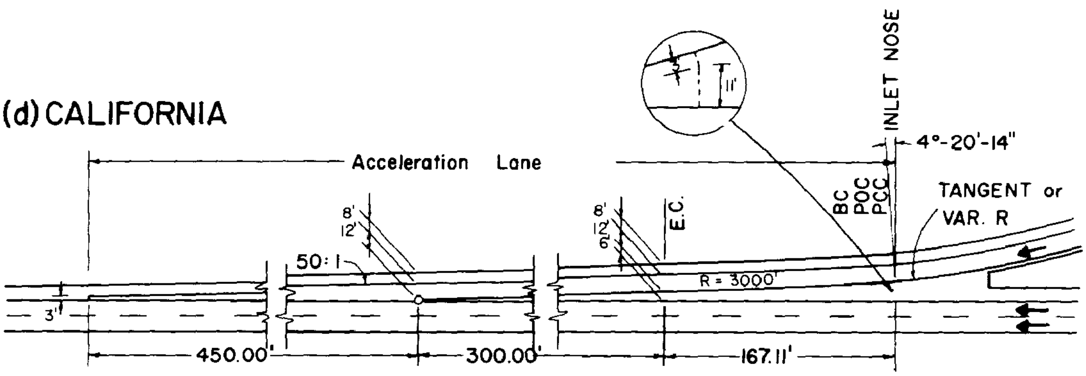


Figure 3-17. Entrances

tion lane may extend back of the point where a vehicle may physically merge with the freeway traffic.

Parallel Type. The major attribute of the parallel type acceleration lane is that the driver is not forced to merge. If what appears to be an acceptable gap disappears, the parallel lane permits an escape for the driver. However, with the vehicle stopped at the far end of the acceleration lane, when a suitable gap occurs the driver must utilize the freeway lane to accelerate to freeway speed. With the current practice of constructing full width paved shoulder to the right of both freeway mainline roadways and ramps, the driver always has an escape area even with the tapered type acceleration lanes. Colorado is the only state investigated which specifies parallel type acceleration lanes in their design criteria. Advocates of the parallel acceleration lane emphasize that it permits better utilization of the rear view mirror for locating acceptable gaps.

Ramp Approach and Terminal Nose

The geometric configuration and treatment of the terminal nose and gore area of entrance terminals varies considerably among the states. Figure 3-17 indicates four typical examples: Texas places raised bituminous stripes in the gore area; Pennsylvania uses a rippled concrete traffic separator; while California and Illinois use paved shoulder material. In each case the purpose is to align the entering vehicle with the acceleration lane and discourage rapid entry to the through traffic lane, thereby reducing turbulence and possible front-to-rear type accidents at the merging nose. All the state design manuals emphasize the necessity of providing adequate visibility between the entering ramp and the through traffic lanes, in back of the entrance nose. Some states specify a sight distance equal to the stopping sight distance for the freeway design speed.

Previous Studies

Since the entrance ramp with no added freeway lane downstream is the one collision course remaining in freeway operation, researchers have investigated the operational characteristics of entrance terminals. Many of the older entrance terminals studied had short acceleration lanes which did not provide good operating qualities, particularly under heavy traffic conditions. Fukutome and Moskowitz note that when the length of acceleration lanes is based upon the difference in the design speed of the ramp and through roadway, unimportant ramps with sharp radii have longer acceleration lanes than important ramps with larger radii. Several investigators indicate that uniformity in the design of entrance terminals, independent of ramp design speed, would simplify design and improve operating characteristics. The most frequently mentioned desirable lengths for acceleration lanes are between 900 and 1,000 feet.

Many researchers contend that the tapered type acceleration provides superior operating patterns since it follows the natural driving path and the pavement edge serves as a guide for the intended maneuver. Also, when properly delineated, the taper shape indicates to the mainline driver that he is in a merging area.

Drew et al. (1967), in a nationwide study of entrance ramps, determined that entrance ramps on a downgrade have better operational characteristics than comparable ramps on ascending grades because the improved sight distance allows the driver to select an acceptable gap several hundred feet in advance of the entrance nose. This permits the driver to accelerate to meet the gap and rapidly merge with the through traffic.

Synthesis and Recommendations

Based upon the research literature review and the workshop discussion, the major conclusions include:

(1) The tapered type acceleration lane conforms to the natural driving path and is preferred by most designers to the parallel type, except where the mainline is on a steep ascending grade and the entrance has a high volume of trucks.

(2) The principal advantage of the parallel type acceleration lane is that it provides an escape area if the gap in the mainline traffic closes before the merging maneuver can be accomplished.

(3) With full width, flush type, paved shoulders adjacent to tapered type acceleration lanes, the escape characteristics of the parallel type are still available to the driver using the tapered type lane.

(4) A taper ratio of at least 50:1 should be used to provide a desirable entrance angle.

(5) The length of straight taper should be a minimum of 900 feet, regardless of the design speed of the turning roadway, in order to afford desirable merging operation characteristics.

(6) In order to properly orient the driver to make full use of the acceleration lane and reduce premature entrance into the mainline traffic, the tangent extension of the tapered acceleration lane back of the physical gore nose should be at least 100 feet, and preferably 200 feet, long.

(7) One of the most important factors in designing entrance terminals is the provision of ample sight distance between the entering roadway and mainline several hundred feet in advance of the physical nose in order that acceptable gaps may be determined as early as possible.

Multi-Lane Exits

In major freeway-to-freeway interchanges, it frequently occurs that the traffic volume turning from the designated through route requires more than a single lane turning roadway. This generally takes place when the mainline, upstream from the interchange, contains three or more lanes in each direction, although it may occur on a two-lane approach roadway of a Y- or T-type interchange when the turning volumes vary materially during different times of the day. The generally accepted criteria for multi-lane turning roadways is a design hourly volume in excess of 1,200 vph. Multi-lane turning roadways usually have design speeds of at least 40 miles per hour since the driving characteristics on the curvature associated with lower design speeds are not conducive to successful multi-lane operation.

At multi-lane exits, the deceleration lane geometrics vary considerably from single-lane exits. It is generally considered essential to add at least one lane to the through roadway as a parallel type deceleration lane for a distance up to 2,500 feet in advance of the exit nose. This provides an opportunity for at least a portion of the turning traffic to shift out of the through traffic lanes and permits the remainder of the turning traffic to move into the right through lane.

When the turning volume represents a large percentage of the total traffic (greater than 40 percent), the exit is generally referred to as a major fork. Except in cases where a high volume entrance occurs immediately downstream from the exit, the number of through traffic lanes is commonly reduced by one lane beyond the high volume exit for lane balance.

Until recently, with the completion of a major part of the interstate system, there have been relatively few multi-lane exits throughout the country. Consequently, there is very little published data on the operational characteristics of multi-lane exits. The principal source of data on multi-lane exits is the current highway design manuals from the few states with experience in this type of design. Figure 3-18 shows the California Division of Highways standard for multi-lane exits from a four-lane directional roadway.

Synthesis and Recommendations

On the basis of the literature review and the workshop discussion, the major conclusions regarding multi-lane exits include:

(1) Multi-lane exits should be provided when the turning volume exceeds 1,200 vph. The minimum design speed for multi-lane turning roadways should be 40 mph and preferably 50 mph.

(2) For satisfactory operating characteristics, a multi-lane exit should be preceded by at least one parallel auxiliary lane between 1,500 and 2,500 feet in length. With heavy turning volumes, a single auxiliary lane 1,500 feet long followed by a double auxiliary lane 2,500 feet long in advance of the multi-lane exit will provide the most desirable operations.

(3) In general, the lane striping and longitudinal construction joints should follow the preference route with extra lanes added for the turning movements.

(4) When the turning volume requires a multi-lane exit and exceeds 40 percent of the approach volume, the multi-lane exit becomes a major fork. Under these conditions, usually one lane is dropped from the preference route beyond the exit.

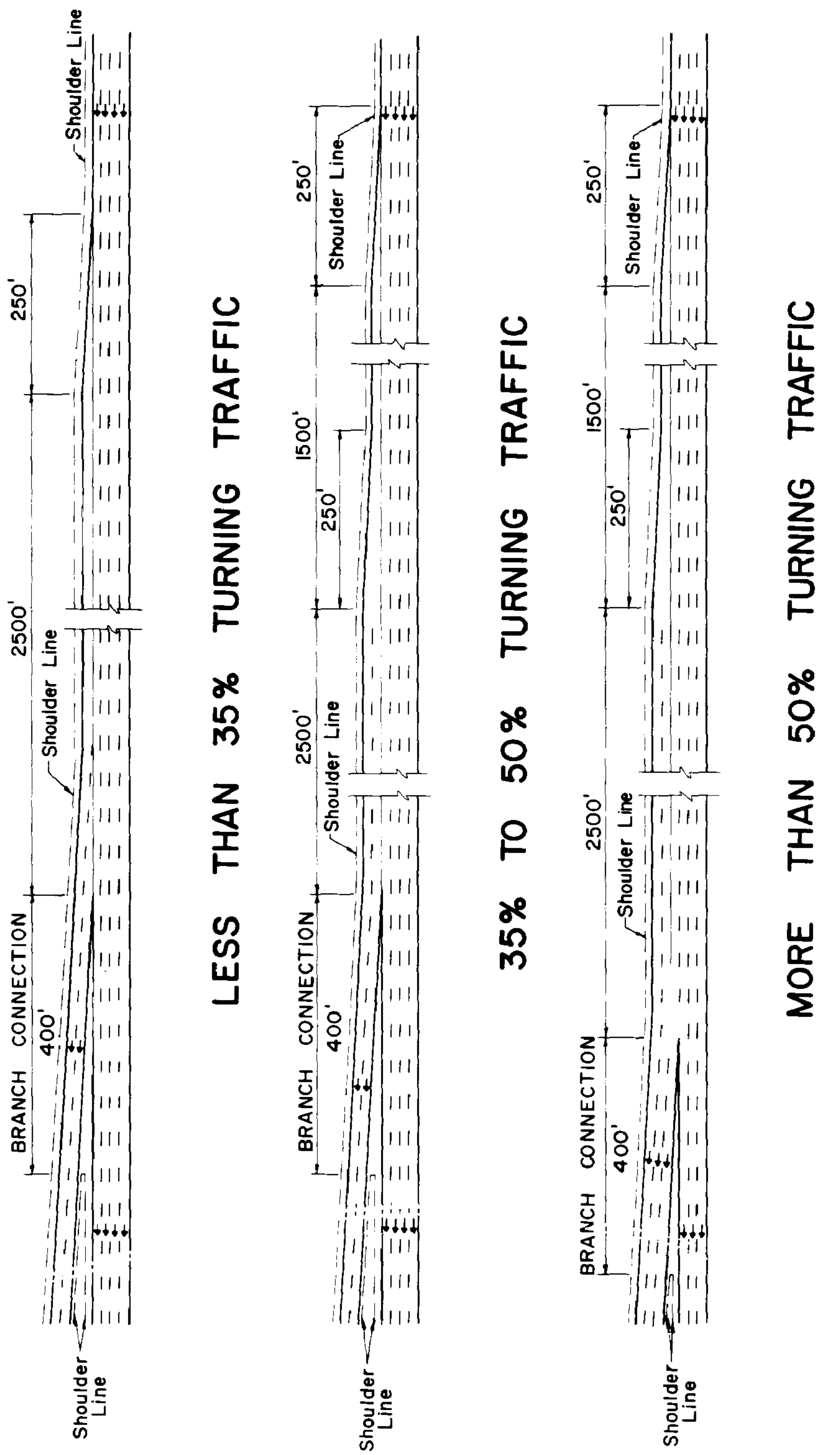


Figure 3-18. Multi-Lane Exits

(5) A four-lane approach roadway should not be split into two two-lane roadways (two lanes dropped from the preference route) except where the fourth lane is obviously an auxiliary lane, less than a mile long, from an upstream entrance.

(6) At major forks, the total number of lanes, including auxiliary lanes, in advance of the split should equal the number of lanes on both roadways beyond the split to eliminate an optional middle lane.

(7) At multi-lane exits, overhead lane control signs are essential for desirable operating characteristics.

Multi-Lane Entrances

When the turning traffic volume in a major interchange exceeds approximately 1,200 vph, it is generally essential to provide multi-lane turning roadways with a relatively high design speed to accommodate these volumes. Where these multi-lane turning roadways join the through freeway lanes, the geometric configuration of the multi-lane entrance and the total traffic volume on both approaches greatly effect the operational characteristics of the merging traffic. In major interchanges where two through freeways cross, the turning traffic volume seldom requires more than two-lane turning roadways. However, if two freeways merge into a single freeway or one freeway terminates at another, as at Y and T type interchanges, more than two lanes may be required at the entrance. Since the principles associated with two-lane entrances are equally applicable to entrances with more than two lanes, the following discussion is concerned primarily with two-lane entrances.

A major factor in the design of multi-lane entrances is the number of through freeway lanes upstream of the entrance and their relative traffic densities. If the upstream freeway approach is immediately preceded by a high volume exit ramp and no freeway lanes are dropped, the traffic density on the through freeway lanes will be relatively low at the multi-lane entrance, permitting easy merging maneuvers for the entering traffic. However, if the through freeway lanes have a high traffic density owing to an upstream entrance or a lane drop at an upstream exit, the merging operation will be considerably more difficult. Figure 3-19 illustrates three alternate arrangements of two-lane entrances: (a) with no lanes added downstream; (b) with one lane added; and (c) with two lanes added. The selection of the proper design will be dependent on the traffic volumes, lane densities, and lane balance. Figure 3-19(a) would be used only where the approach densities are low and the three lanes downstream have adequate capacity. Figure 3-19(c) is seldom utilized except where all approach lanes have high densities or where the fifth lane is to be dropped at a subsequent exit. The most common arrangement for two-lane entrances is the addition of one downstream freeway lane as shown in Figure 3-19(b).

Except in the case where two downstream lanes are added, at least two lanes must be merged into one at two-lane entrances. Three configurations for the arrangement where one lane is added downstream are illustrated in Figure 3-20. Each of these configurations has its advantages and disadvantages -- there is divided opinion on the most desirable configuration.

There is relatively little published information on the design and operating characteristics of multi-lane entrances since, prior to a substantial completion of the Interstate system, there were few in existance.

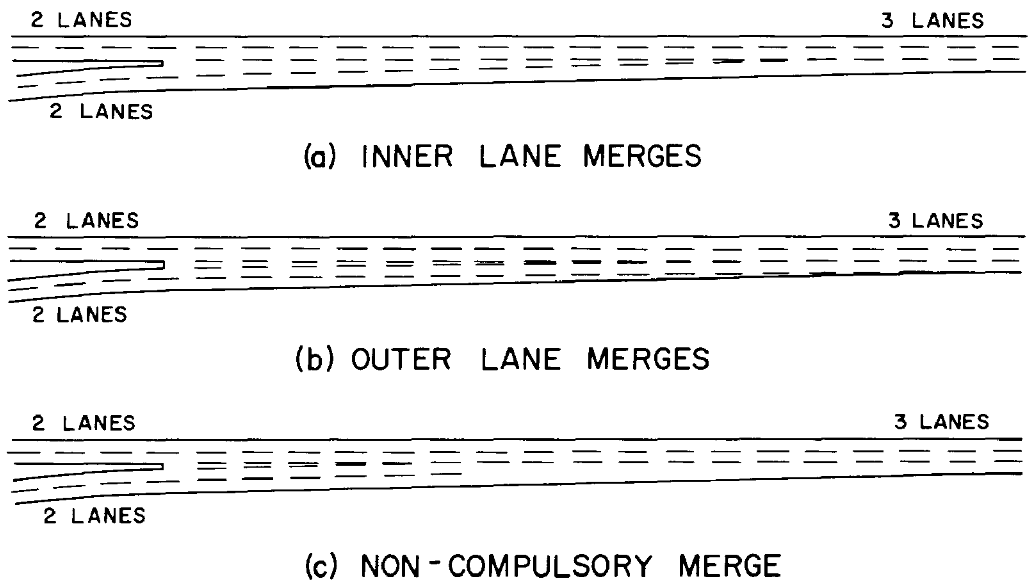


Figure 3-19. Two-Lane Entrance Arrangements

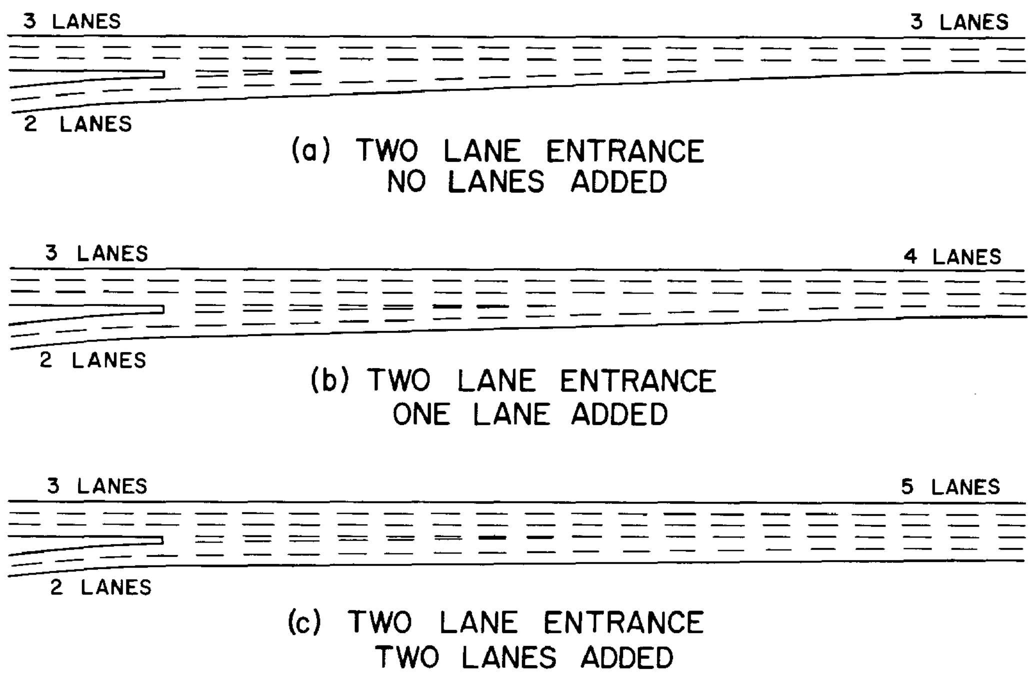


Figure 3-20. Merging Lane Configurations

An Institute of Traffic Engineers Report on "Two-Lane Entrance Ramps" outlined the basic principles and problems involved but drew no major conclusions other than that additional research on the operational characteristics of two-lane entrances was required.

Synthesis and Recommendations

The principal conclusions regarding multi-lane entrances, based upon the literature review and the workshop discussions are:

(1) Since relatively little research on the operating characteristics of multi-lane entrances has been conducted, this is a field in need of further investigation.

(2) When the turning traffic volume warrants a multi-lane entrance, it is generally essential to add at least one freeway lane downstream from the entrance.

(3) Where two entrance lanes must be merged, the merging of the outer lane provides the best operating characteristics, since the adjacent paved shoulder provides an escape route when no gaps are available for merging. With an inner lane merge, there is no escape route.

(4) If no additional freeway lane is to be provided downstream from a two-lane entrance, thus requiring the two lanes to merge into the freeway lanes, a parallel auxiliary lane at least 800 feet and preferably 1,200 feet long should be provided between each separate merge.

(5) A minimum merging taper ratio of 50:1 for all merges should be provided. On very high speed rural freeways (75-80 mph) taper ratios up to 100:1 may be desirable.

Chapter Four

CONCLUSIONS, RECOMMENDATIONS AND SUGGESTIONS FOR FUTURE EFFORTS

Specific conclusions, recommendations, and suggestions for future efforts as derived from the various study phases of this project are incorporated in the pertinent chapters and appendices. A number of the most significant of these are summarized and presented in the chapter. Additional general conclusions and suggestions, based on an overview of the total project, are also included here.

Design Procedure

1. Design Process Flow Chart. It is not feasible to construct a definitive flow chart of the major interchange design process. If sufficiently general to encompass the practices of all highway departments and consulting firms, the procedure is little more than the listing of the many factors and considerations involved. A more detailed description typically is not responsive to the varying design approaches dictated by variations in organizational structure among the design agencies. Further, the specific constraints on an individual project largely determine the nature of the final design, and these site-specific constraints cannot be meaningfully integrated in a generalized design procedure.

2. Continuing Improvement in Interchanges. That the design process is "non-chartable" at a meaningful level of detail is not a major problem. The end products, the interchanges themselves, are improving from the standpoints of both the user and non-user. The recommendations in this report for improving the design process are aimed at acclerating this product improvement trend.

3. Conventional Systems Approach. A "systems approach" for an individual interchange design may be feasible and desirable -- the magnitude of the costs involved justifies considerable effort in developing a project-specific design approach. However, a generalized systems approach applicable to all major interchanges will suffer from the same weaknesses mentioned in (1) above. These include the inability to formulate a universal procedure at a meaningful level of detail considering the constraints imposed by varying organizational structures.

4. Bayesian Decision-Theory Approach. A Bayesian decision-theory approach to evaluation of alternative designs is introduced in Chapter Two and described more fully in Appendix E. This approach has considerable merit in that it is flexible enough to be applied to many situations -- permitting wide variations in goal setting and systematic incorporation of "engineering judgment" in the design process. Additional research is needed to develop techniques for extracting goal and value judgments from design experts and from the public in a manner compatible with the procedures proposed. Field testing will also be required to validate the applicability of this methodology.

5. Trade-off Analyses. A trade-off analysis technique is also presented in Appendix E. This technique can be used to select the desirable level of investment for the various interchange elements (e.g., length of acceleration lane, width of shoulder at bridge structures) and the expenditure justified to obtain "desirable" configuration features (e.g., right vs. left ramps, single vs. double exits). For this technique to become useful, further effort in deriving consensus "expert judgment" on the relative worth of variations in design elements is required.

6. Standardization. While it may be desirable to standardize elements and features of interchanges, there is little hope of developing a small number of "standard interchanges" which may be installed at any site. Even for the more common elements and features, there is little convincing information available for recommending "minimal," "better," or "optimal" values.

7. Policy Manuals. National policy manuals on traffic control devices and geometric design have led to a reasonable degree of consistency in interchange design. It is obvious that user convenience and safety are improving with time through the application of evolutionary design criteria. Revisions of the manuals are not made on any regular basis, however, and they frequently do not reflect the latest thinking of the design community. Although Policy and Procedure Memorandums are issued by the Federal Highway Administration and by the various state agencies as interim measures, it is felt permanent groups should be established at the national level and charged with updating and coordination of the various manuals. They should also see that new research results are adequately reflected in the manuals, and could be charged with defining the research efforts needed to remedy policy deficiencies.

8. Public Inputs. As public opinion and goals are now major factors in the interchange design decision, better methods for assessing these are required. In particular, techniques for quantification are needed.

The purposes to be served by the public hearing process need to be clarified. At present, the hearings may serve as information gathering devices and/or as presentation mechanisms. Public relations specialists should conduct the hearings, with the design engineers as resource personnel. Techniques for effective presentation of design impacts and methods for gathering public opinion should be developed through future research efforts.

9. Feedback. The designer must learn more from his previous designs than he currently does. Operational and non-user impact feedback channels to the engineer must be established in a more formal way to encourage

learning by experience, both within a state and across the nation. Monitoring procedures should be established to assess operating efficiency, and public opinion surveys should be conducted to assess non-user impacts.

Interchange Elements -- Design Criteria and Guidelines

1. Visibility. The principal design elements of major interchanges were analyzed in detail in this study to determine the factors contributing to both good and bad operating characteristics. Except in those cases where traffic volumes exceeded design volumes, resulting in serious congestion, the erratic maneuvers indicative of poor operating characteristics are generally attributed to the unfamiliar drivers confronted with sudden or unexpected decisions. The best operating conditions are obtained under the following conditions: (a) when drivers have adequate advance information concerning the interchange; (b) when "anticipatory sight distance" is available in the vicinity of decision points; and (c) where "unexpected" design elements are avoided. In fact, the majority of the design engineers at the workshops expressed the belief that virtually any reasonable design configuration will provide satisfactory operating conditions if good signing is provided in conjunction with sufficient visibility of the design geometry.

2. Left Exits. Left exits are generally undesirable, but may be used where high left-turn volumes require two or more turning lanes. In general, this situation should be treated as a major fork.

3. Left Entrances. Left entrances are to be avoided unless an additional freeway lane is added on the left downstream from the entrance.

4. Cloverleaf Interchanges. Cloverleaf configurations are generally not appropriate for major interchanges in urban and suburban areas, but may be

acceptable for low volume rural freeways -- particularly when provided with collector-distributor roads.

5. Lane Drops. When decreased traffic volumes justify a reduction in the number of through lanes, the preferred location for the lane drop is beyond the influence of the interchange. Most of the design experts polled feel the right lane should be dropped.

6. Single Exit Designs. A single exit configuration is more desirable than two separate exits from signing and operating characteristics viewpoints. If a two-exit configuration is chosen, designers, in general, feel the right turn exit should precede the left turn exit. However, satisfactory exceptions to this were cited in Texas, perhaps indicating that factors such as signing rather than the design *per se* have greater influence on performance.

7. Acceleration and Deceleration Lanes. For single-lane entrances and exits, the tapered type acceleration and deceleration lanes are recommended. Where it is important to provide more "target value", the full-width deceleration lane should be used.

8. Multi-lane Entrances and Exits. The number of lanes on the main line should be increased by one downstream from two-lane entrances. An additional auxiliary lane at least 1,000 ft. long should be provided downstream from multi-lane entrances and upstream from multi-lane exits.

9. Route Continuity. Route continuity and map relatability are considered desirable by the design community, but are accorded low priorities in the determination of the final design configuration.

10. Research Priorities. Many of the design elements of major interchanges have counterparts in other types of interchanges, and research, operation, and design experts have studied these elements extensively in the past. However, multi-lane entrances and exits are almost unique to

(2) The horizontal and vertical alignment of the exit ramp immediately beyond the nose should be readily visible to provide a target area for the driver entering the ramp.

(3) Constructing contrasting paved shoulders on both the right and left of ramps also assists in delineating the intended travel path.

(4) The gore area should have a high-type pavement. It should be flush with the mainline and ramp pavements to permit the errant vehicle to cross it easily when necessary. The gore pavement may be the same as a travelled roadway, or a high-type paved shoulder material.

(5) The gore area may be zebra stripped with painted or plastic markings for additional delineation. Raised pavement markers are also helpful in areas where they can be used.

(6) The physical ramp nose should be offset at least a full-shoulder width from the mainline roadway and a minimum of 4 feet from the ramp pavement. It should be as free as possible from any curbs or obstructions, and the area beyond the nose should be graded for at least 100 feet.

(7) When directional signs must be located beyond the gore, they should be of the break-away type.

(8) If it is not possible to eliminate physical obstructions at the gore (e.g., bridge parapets), sufficient area for the installation of an energy absorption device should be provided.

Entrance Terminals

In a major interchange, where a ramp or turning roadway connects to a freeway mainline roadway, the number of freeway lanes downstream from the entrance may be the same as upstream from the entrance, or may be increased by one or more lanes, depending upon the mainline and entering traffic volumes. When the number of lanes added downstream is equal to the number of lanes on the turning roadway, there is no forced merge and the roadway beyond the entrance nose operates the same as any other section of multi-lane freeway except for the slower moving vehicles on the mainline moving to the right. One of the most frequently occurring entrance conditions is where a single lane ramp joins a mainline roadway and no additional traffic lane is provided downstream from the entrance. This generally occurs in low volume interchanges and where more lanes than necessary for the traffic volume are carried through the interchange to avoid lane drops.

Where multi-lane turning roadways join freeway mainlines and no lanes are added to the freeway or the number of lanes added is less than the number on the turning roadway, a complex merging problem exists. This condition will be discussed in a subsequent section on "Multi-Lane Entrances."

The design speeds of interchange ramps and turning roadways are generally 15 to 40 miles per hour slower than the through roadways, depending upon the location of the interchange, rural or urban. With single lane entrance ramps where no additional lane is provided downstream, the driver performing the entering maneuver is required to select a gap in the outside freeway lane traffic stream, accelerate to a comparable speed, and merge with the through traffic. To permit the driver to perform this maneuver with the least effort, the geometric design of entrance terminals must take into account the length and shape of the acceleration lane, the geometry of the entrance nose, and the approach to the acceleration lane. The factors to be considered in entrance terminal design include design speed, capacities, grades, curvature, traffic volumes, and sight distance.

freeway-to-freeway interchanges and additional research on the operational characteristics of these elements should have first priority.

11. Application. Two detailed case studies are presented in Appendices G and H to demonstrate the applicability of the design guidelines and criteria (Chapter Three) to the evaluation of existing or proposed interchange configurations.

Freeway Control and Operations

The question of the feasibility of including major interchanges into over-all control schemes must be considered from the standpoint of the types of control involved and cannot be answered as a general question. That is, it is necessary to make separate feasibility evaluations of ramp control (metering and closure), mainline speed and lane control, lane reversibility, and lane exclusivity as they apply to major interchanges.

1. Ramp Closure. Closure of a two-lane connecting roadway at a major interchange is obviously not feasible. The closure of single-lane local access ramps is possible, but in many cases results in "political" problems. Further, unless the closure device is automated there is the problem of sending personnel to each closure site on a daily basis. Physical closure is necessary because it has been shown that "signed" closures are violated by a large percentage of the drivers.

While ramp closure is not highly recommended, if it is used it should be used consistently (ideally from the outset of facility operation) so that "locals" can predict when the closure will occur and establish their travel patterns accordingly.

2. Ramp Metering. There is little experience with the metering of two-lane ramps. Currently, the display and software technology has not

been developed to the point where adaptive control can be effectively used on two-lane high volume merge situations. Fixed metering is not desirable because of the requirement for free-flow through major interchanges. The metering of local access (single lane) ramps within a major interchange can be expected to operate as effectively as it does on any comparable non-major facility.

Research is required to determine the geometric requirements, display placement requirements, sensor placement requirements, and software requirements for a merging control system designed for two-lane high volume ramps.

3. Main Line Speed and Lane Control. The use of main line control to improve merging and flow operations is technologically feasible. The major problem with such controls is that drivers have been shown to disregard the control signs and signals. It is, therefore, suggested that research be conducted to determine the types of information and display required to produce the desired behavior. If such controls are used, it is recommended that the required response (i.e., speed or lane change) be based upon real-time information so that the request has credibility. Additional criteria for the use of main line controls are presented in Appendix L.

4. Lane Reversibility. While the reversible lane is not a control method which has wide-spread applicability and should not be used as a remedial treatment, it has been used successfully where serious directional imbalance occurs during peak periods. While the feasibility of designing reverse operation into a major interchange is not in question (from the standpoint of geometrics), there may be significant problems associated with signing such a design.

special conditions to be observed, a description of the final design with an evaluation of the subsequent operations, the remedial actions initiated as a result of operational problems, and the lessons learned from the experience. Somewhat different headings may be necessary in different situations.

The principal problem will be eliciting appropriate materials from the design community. It may be that a special group at FHWA will be required to work with the various design agencies to develop the materials, or an outside agency might be contracted to develop the Fact Sheets in conjunction with the design agencies.

3. Innovative Designs Digest. An open-ended digest of innovative approaches to the design of entire interchanges or interchange elements was presented as Interim Report 3. This digest should be expanded through an intensive effort to assemble appropriate information already in existence in the files of the various design agencies. After that, a mechanism for continual updating should be established. The principal problem will be in obtaining submissions from the many sources.

4. Interchange Classification. A detailed classification scheme for reference to interchange configurations is not practical nor would it fulfill any real need of the working design groups.

5. Detailed Design Feedback. An inventory of interchanges, with detailed information on each segment, shows promise as a tool for evaluating the "successfulness" of specific design solutions. Such an inventory would provide feedback for improving design and for integrating all elements of an interchange. At present, specific design feedback is very scarce or non-existent.

6. Computer Graphics Inventory. A comprehensive inventory and history on all elements of interchanges would require a computer (for storage of

detail) and a graphics output unit. Since this computer graphics inventory system could be maintained at a reasonable cost by a central agency and be useful to designers, operational groups, planners, and researchers, it is recommended that such a system be studied for possible implementation to collect all known information on interchanges and make it readily available to all groups requiring it.

7. Photologs. Photologging is an existing roadway inventory technique, but it is limited in the kind of detailed feedback it can provide to designers. Operational groups have found photologs valuable for sign, maintenance, and road furniture studies, but accident rates, initial cost, maintenance frequency, public reaction, traffic flow, erratic driver behavior, and values for geometric features are not available.

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To the extent that there is interest in using reversible lanes as a control method, additional research will be required as to the best methods of signing and in the development of guidelines for sign placement.

5. Lane Exclusivity. The general feasibility of exclusive bus or carpool lanes has been established. However, there are many questions related to whether median or outside lanes should be used as the exclusive lanes and how the entrance/exit problems should be handled. The median lane requires that separate entrance and exit ramps be provided or the buses must weave across a number of traffic lanes, including the high speed lane. The use of the outside lane as the exclusive lane produces the problem of bus traffic interacting with other exiting traffic. One of the major questions is whether the more desirable separate entrances and exits are justified from the cost-benefit standpoint. Experience with exclusive lanes has indicated that they do not operate as intended unless the bus volumes are high or rigid enforcement is provided. Under low bus volumes, voluntary compliance by other drivers is low.

There is need for research on the cost-effectiveness of the exclusive lane concept so that some additional guidelines can be developed for use by designers in evaluating this as an alternative to other designs. Further, there is need for additional design analysis to determine the relative merits of median vs. outside lane designs under several exit/entrance configurations.

Design Information Systems

The Highway Research Board is highly effective in disseminating research findings relative to the design and operation of freeway interchanges. However, there is need for one or more forums for exchanging ideas and experiences in configuration selection and evaluation and specific element design decisions. A number of possible approaches to the dissemination of this information were investigated within the various phases of this project, and a number of these are mentioned in this section.

1. Workshops. The two workshops were valuable in providing information for the project itself -- but many of the participants remarked that they felt they received far more from the workshop than they had contributed. The format of prepared topical presentations, followed by open discussion by the participants and summary written questionnaires appears to be quite effective. High interest can be maintained over a three-day period.

It is suggested that more workshops be held on a regularly scheduled basis, and that the participants should be drawn from the working levels of highway departments and consulting engineering firms. The FHWA can organize and conduct these workshops, but the possibility exists that the participants will then consider these as training sessions to bring them more in line with FHWA policies and directives. A more open discussion can probably be obtained if the workshops are conducted by an independent agency, as was the case in this project.

2. Design Experience Fact Sheets. While the findings of research studies are usually published and distributed through governmental or institutional channels, actual design experiences of individual engineers often are not collected, organized, or made available to those who would find such material useful. Hence, a need exists for a method of gathering, indexing, and publishing information which will permit the freeway designer to compare and evaluate his design with other designs used in similar situations.

The idea for a documented case history "Book of Fact Sheets" was conceived at the workshops. As developed in this project, the Fact Sheet incorporates a description of the general situation and traffic characteristics,