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MAJOR INTERCHANGE DESIGN, OPERATION, AND TRAFFIC CONTROL

Vol. 2. Appendixes A – G

J. I. Taylor and others

July 1973
Final Report

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For table of contents of this report, see Vol. 1 (FHWA-RD-73-80).

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:

FHWA-RD-73-80. Vol. 1. Text of Report
FHWA-RD-73-82. Vol. 3. Appendixes H-M

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APPENDIX A

SUMMARIES OF INTERIM REPORTS

APPENDIX A: SUMMARIES OF INTERIM REPORTS

Introduction

In the course of the project, three interim reports were published under separate cover. The summaries of these three reports are included in this appendix to indicate the nature of the information available therein.

Interim Reports 2 and 3 are essentially independent, whereas much of the material contained in Interim Report 1 has been incorporated in this report.

Interim Report #1: Current Practices and Research Review

General Findings

The topic of major interchanges is rarely treated as a separate entity either by state highway officials or by researchers. More often than not, major interchanges are coupled with minor interchanges, both of which are considered under the general topic of grade-separated intersections. Indeed, it may be that it is this failure to recognize the special problems and unique circumstances associated with freeway-to-freeway connections that results in the less-than-ideal traffic flows which characterize some major interchanges.

In the matter of design procedures, each state has its own, and seems reluctant to consider adopting a nationwide or regional standard. State highway officials argue that their procedures match their own requirements and regional peculiarities and that a standard procedure for all states would be unwildly, insensitive to local concerns, and generally impractical.

The state organizations vary widely in their approach to design. Some states support a central core of engineers who are responsible for the design of all freeway projects in the state, while others maintain resident engineers in various principalities to assure that new designs serve the local needs. In some states, the design process flows through a series of different offices, or bureaus, each of which controls or contributes to some phase of a project; the total design procedure in other states, from initial planning, through evaluation and approval, to final design is handled by a few individuals. No single procedure could possibly span such a wide range of practices without being accompanied by significant changes in organizational structure, policies, and procedures in a number of states. A set of guidelines, and suggestions, however, would be of value so that gradual changes that do occur could tend toward an optimal "standard" procedure.

With regard to design selection criteria, it is probably not unfair to observe that the choice of interchange configuration is more a matter of state custom and predisposition than of analysis. The merits and demerits of this situation may be argued, but given that each state organization and procedure is largely the result of independent evolution, it is easy to understand how the situation has developed.

As a consequence, a curious paradox obtains from this situation. It is an often quoted principle that every interchange must be designed in such a fashion as to marry it to the unique features of its location. Hong (1966) comments on this notion in his paper on interchange design:

> "Even though a set of geometric design standards has been established by the Bureau of Public Roads and State Highway Departments, these standards serve merely as a guide and not as a rigid set of rules. It is also true that there are many common denominators for freeway design features in different locations of the United States. However, each of these freeways is unique by itself in that the

climatological and geographical features, and socio-economical characteristics may be considerably different from one region to another.

"Therefore, each freeway or expressway must be custom-made to meet the local demands and to conform to the local topography in order to establish not only a functional but also an aesthetic transportation system. It must be emphasized that the judgment and ingenuity of the design engineers are the main guides for judicious design of an efficient transportation system."

In practice, however, a unique solution to interchange designs is not always applied to each site; rather, a general solution (based on past experience and/or predisposition) is applied and then modified to meet the particular requirements of the site. As a result, the state of interchange design is considerably less dynamic than might be supposed. Further, radical (but potentially useful) approaches to interchange design, such as the application of set-theoretic methods proposed by Alexander and Manheim (1965), have seldom found their way into the state procedures.

This is not to say that interchange design is purely a matter of chance or bias. On the contrary, the reason that a particular design finds favor with a state highway department is that in the past such designs have met with successful operations in that state, sometimes even in the face of contrary logic. Thus, in Texas there are interchanges that provide for left turns in advance of right turns -- a feature that is highly questionable to many design engineers. In response to criticism, Texas officials note that the people who drive the interchanges seem to have no trouble with them. While Texas designers do not promote such configurations as standard, they do not hesitate to utilize them when they feel the circumstances warrant it.

There appears to be as much disparity as conformity between practice and research results. Both the design texts and the research literature,

for instance, stress the importance of factors such as visibility and uniformity of features for the convenience and comfort of the driver. State design personnel, however, seem to consider uniformity as being of secondary importance. Likewise, while some researchers identify weaving areas as major problems and recommend their elimination from freeways, the design texts and practioners view them as necessary, if unpleasant, features.

Conversely, while most officials steer a clear path away from the use of left exits, research fails to provide incontestable evidence that left exits are, when properly designed and appropriate, more dangerous than right exits. Some studies which purport to show the hazard of left exits have been criticized for neglecting to correct left exit accident occurrences for exposure.

The following findings are also noted:

1. Accident data are not sufficiently sensitive to the effects of geometrics to be used as evaluative measures unless only gross geometrics are of interest and the data are corrected for exposure.

2. Two elements of entrance ramp terminals that are discussed at length by a number of researchers are visibility (including delineation) and standardization of length, taper type, and taper rate. Many authors suggest these factors may be more important than geometrics *per se*, providing that certain minimums are met.

3. The AASHO guidelines for shape and taper rate of speed change lanes are generally supported by the literature, although differences exist between the states as to preferred shape.

4. Both the research literature and the state design manuals are in accord with the AASHO guidelines for entrance ramp convergence angle.

5. The research data on acceleration lane length do not support the guidelines issued by AASHO. The use of volumes or speed differential between entering and through traffic as controls for length of merging areas are specifically criticized on the basis of logic and operational studies.

6. From the standpoint of driver comfort and traffic flow, the circular loop appears to be superior to the elongated loop. The AASHO publications fail to differentiate between the two.

7. State highway officials observe that in recent years, final designs are based less on optimal features and more on the least objectionable features. They are particularly troubled by the fact that local socio-political groups, who possess meagre information about an experience with roadway design can force changes in interchanges that seriously impair their adequacy.

8. Feedback from operations analysts to designers is at best poor and at worst nonexistent except on those interchanges which are almost hopelessly inadequate from their opening.

9. Speed-change lanes should be designed and signed or striped in such a fashion as to encourage use of their full lengths. Operational problems on such sections are often the result of drivers using only a portion of the section provided and making an abrupt entry or exit, thereby producing turbulence in the freeway lane.

10. Consistency in lane drop techniques is urged. Unfortunately, conclusive evidence as to optimal techniques has not yet been established. As a result, the states employ different lane drop configurations which are confusing to drivers unfamiliar with the particular roadway.

11. Ideas on and experiences with some promising new design features and interchange configurations fail to be promulgated outside the state of origin because the highway designers are too busy with daily operations to publish their ideas in the appropriate journals. A digest of some of these features and configurations will be compiled within this research project. It is anticipated this digest will be one of the major interest items in the Final Report.

12. Many of the less-than-optimal designs and features found on the older interchanges may have resulted from compromises for cost reductions. Today, on the other hand, cost factors seem to be almost entirely absent from the procedures utilized in the selection and evaluation of alternative component configurations and in the development of design details. Until data are available clearly relating operational and safety benefits to costs of the various design features, this tendency is not likely to change.

Major Problem Areas

Review of the literature, current standards, and research, and discussions of design practices with engineers in the state highway departments reveals a number of major problem areas in the design, operation, and traffic control of major interchanges. These problems, listed below and discussed in more detail in Chapter IV, fall into three categories:

Policy Problem Areas

Changing Priorities

Environmental, Aesthetic, Ecological Considerations

Involvement with Local Agencies

Local Access

Partial Interchanges

Exclusive Bus Lanes

Design Procedure Problems

Uniqueness

Design Project Management

Selection of Basic Configuration

Driver Needs

Adaptability and Flexibility

Trade-Off Analyses

Cost Effectiveness

Checklists

Design Experience of Reviewers

Component Design Problems

Left-Hand Ramps

Entrance Ramp Capacity

Two-Lane Ramps

Hidden Ramps

Collector-Distributor Roadways

Consecutive Ramp Arrangements and Weaving Areas

Lane Drops

Grades

Signing

Non-Conformance of Travel Paths and Construction Joints

Freeway Traffic Control

Nomenclature

Traffic Forecasts

Three-Dimensional Models

It was discovered both from the workshop and from the questionnaires that only occasionally are interchange models built, and that those models which are built are used principally for presentation purposes rather than as design tools.

As a group, the models take longer to fabricate and cost more than the questionnaire developers anticipated. This result obtains from the fact that design models (which, in comparison with presentation models, are primitive and lack cosmetic trim) are used much less frequently than was expected.

It may be observed that engineers are generally not unfavorably disposed toward the use of models: on the contrary, a majority held that better designs resulted from their use. Their infrequent utilization stems rather from their low order of priority in the hierarchy of events which forms the design process.

Checklists

To the workshop participants, checklists are anathema. They are regarded as impediments to imaginative thinking and as substitutes for professional judgment. In view of this antipathy, it was surprising to discover that the questionnaire respondents generally have a favorable attitude toward checklists, while at the same time noting that their use in the design process is rare. The disparity between the two groups may be attributable to experience. The workshops attendees were, by and large, seasoned veterans. It is suspected that the questionnaires were completed by more junior engineers, who might be more likely to depend on design aids.

Computer Graphics

The general reaction of the workshop and the questionnaire populations to computer graphics as a design aid is one of interest without enthusiasm. In both groups there prevails a posture of "wait-and-see" or "need more information." In part, this attitude seems due to a skepticism regarding the costs and the efficacy of computer graphics; but further, there appears to be a reluctance to permit highway design to stray too far from human influence toward impersonal dictation.

Interim Report #3: Innovative Designs Digest

This Digest 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. The designs, pictorial and literally defined, include:

- Turbine interchange
- Arch-supported interchange
- Double diamond interchange
- Directional interchange with left turn first
- Major fork configuration for direct left connections
- Anti-weave designs
- Local access diamonds
- Multinode interchange complex.

APPENDIX B

PRE-WORKSHOP QUESTIONNAIRE AND RESPONSES

APPENDIX B: PRE-WORKSHOP QUESTIONNAIRE AND RESPONSES

Introduction

The responses to the Pre-Workshop Questionnaire from 12 state highway design engineers, 4 consulting engineers, 1 highway research engineer, and 4 FHWA engineers are tabulated in this appendix. The respondees are identified in Table B-1. The questionnaire consists largely of questions regarding different design practices and personal opinions regarding various design elements. Most of the questions are of the multiple choice type, and all invite additional comments.

The instructions accompanying the questionnaire pointed out that opinion questions have no "right" or "wrong" answers, that the responses would be used to formulate recommendations for updating design guidelines, and that specific individuals would not be identified with specific responses.

A few of the questions are of the "essay type," and the responses are rather detailed. In large part, the answers to these questions are listed in summary form, deleting the repetitions; it is felt significant information would be lost if the answers are collapsed to tabular form.

As some of the questions do not apply to some of the respondents, the numbers of responses cited in some of the tables do not total to 21. Some of the tables give both the number of respondees selecting a particular answer, and the appropriate percentage.

Major points derived from a synthesis of the questionnaire responses are discussed briefly in the next sub-section. These are organized by major topical areas, in line with the organization of the Workshop Sessions. (See Appendix C for the Workshop Agenda.)

Synthesis of Responses by Workshop Topics

Standardization (Questions: 8, 9, 17)

The respondents felt that standardizing some aspects of the geometric design of major interchanges is possible, but generally not practical. Of those who do not feel that it is even possible in urban areas, most remarked that the unique conditions at each site are the major deterrents to standardization. For rural areas, topography appears to be a dominant factor in design decisions. Restrictions of cost, space availability, and special traffic conditions are the dominant arguments against the practicality of standardized urban designs.

Design Process (Questions: 1, 2, 3, 46)

It appears that the weighting assigned to the various factors which design engineers use to determine geometrics of a major interchange is variable. Two respondents state that the factors cannot be listed in any particular order. The design volumes and level-of-service appear to be the more important factors. The respondents also feel that the major differences between major interchanges and others are provision for continuous movement and basically a higher level of service. Finally, more than two-thirds of the respondents feel that it is desirable to have a separate design procedure for major interchanges.

Configuration Evolution (Questions: 1, 2, 27)

Question 27 is the only one dealing specifically with overall configurations, although Questions 1 and 2 are relevant. Lower cost and acceptability with lower turning volumes are cited as factors favoring the cloverleaf.

Cost-Effectiveness, Trade-Offs (Question: 19)

The number of engineers who will accept a left-side ramp increases as the relative cost savings of the left side ramp over one on the right increases from $100-500 thousand. One respondent indicated the savings would have to exceed $1 million before he would seriously consider using a left-side ramp in a major interchange.

Visibility, Design Aids (None)

Design aids are discussed in Interim Report 2.

Exits (Questions: 10-24, 34, 37, 38)

The respondents generally feel that the single exit for both turning movements is the most desirable, even for exits requiring two lanes. The two-exit configuration, with the right turn taken off first, is the next choice. Left-hand exits are almost never used.

The opinions on the desirable lengths between exit gores varied, and no one value dominated with a high percentage of selection.

Tapered deceleration lanes are preferred and used in most situations. The parallel type is used for situations where ramp and mainline speed differences are too great, sight distance is restricted, the mainline is on curve, or there is a possibility of back-up onto the through lanes.

Entrances (Questions: 12-17, 25, 26, 28-33, 39-43)

Left side entrances are generally deemed permissible only when there is no space available for a right-side entrance, or cost of the alternatives is prohibitive, and the entrance lane(s) is added to the mainline. A single entrance (two ramps form one ramp prior to entrance terminal) to the mainline is almost unanimously deemed preferable over a

double entrance. Left entrances are rarely used, and the single entrance from the right shows a slight edge of use over the double entrance (both on the right). Again, if a lane is added, there is less objection to the left-side entrance.

Nearly half the experts would use the double entrance with an adequate separation distance rather than a single entrance requiring two lanes. Again, the values for that distance are almost evenly distributed in most cases.

The taper type acceleration lane for entrance ramps is generally preferred, but the parallel is used in some situations, such as restricted length, very heavy mainline flow, or poor sight distance. A 50:1 taper is felt to be the most desirable value for the single entrance. Fifty-seven percent of the experts felt that the right lane should be dropped for that situation.

On the mainline, the merge of the two entrance lanes shows dominance of use over the merge of the left entrance lane and the right mainline lane. Finally, the merge of the two right lanes showed clear dominance over merges of either the center or right lanes.

Route Continuity; Ramp Arrangements (Questions: 12-17, 19-24, 26, 28, 30-32, 44)

The collector-distributor road for two adjacent loop ramps is used more in urban areas than in rural areas, and more often than a design with no C-D road. The responses regarding the desirable distance between the loop ramp terminals do not make any one value the clear-cut choice.

About 80 percent of the respondents feel that weaving areas should always be avoided in major interchanges, yet 70 percent state that

weaving areas can be justified with adequate weaving length. Also, 75 percent feel that weaving sections are acceptable if they are off the through roadway.

Lane Drops; Lane Balance (Questions: 35, 36, 40-43)

Almost no one uses a left-lane drop adjacent to a right-side exit terminal. For the two-lane exit from a basic 8-lane freeway, the right side lane drop adjacent to the exit terminal shows clear dominance over the design where the lane is carried through the interchange.

The respondents clearly prefer the right side mainline lane drop. Also, the right lane is chosen as the lane to be dropped where a two-lane turning roadway is merged into one lane before the entrance terminal at the mainline. For the merge of two 2-lane roadways into one 3-lane roadway, the design where the two right lanes are merged is clearly preferred over the alternatives.

Local Access; Freeway Control (Questions: 4-7)

The respondents indicate that political pressure sometimes dictates the provision of local ramps in a major interchange, but that this is not usually the case. Two-stage public hearings have some impact on local ramp provision. Practically all the respondents feel that local access should not be permitted in a major interchange.

Opinion is divided on whether selective closure of ramps during peak periods is a practical solution to capacity-operational problems.

TABLE B-1. Respondents to Pre-Workshop Questionnaire

Design

Churchill, Robert R.
Deputy Design Engineer for Roadways
Florida Department of Transportation

Dayton, Edwin W.
Chief, Bureau of Surface Design
New Jersey Department of Transportation

Everhart, B. F.
Chief Design Engineer
Ohio Department of Highways

Foster, W. M.
Assistant Director of Highway Development
Washington State Highway Commission

Foy, Robert A.
Chief Engineer
Design Division
Wilbur Smith & Associates

Gazda, Andrew J.
Engineer of Geometric Design
Illinois Department of Transportation

Hall, Parker L.
Assistant Engineer for Design
California Division of Highways

Hibbs, John O.
Regional Design Engineer, Region 3
Federal Highway Administration

Hofmann, Frederick J.
Senior Highway Engineer
Edwards and Kelcey, Inc.

Housworth, Jack L.
Supervising Design Engineer
Texas Highway Department

Kenyon, Alan D.
Associate Civil Engineer
New York State Department of Transportation

Lins, William F.
Chief, Bureau of Highway Design
Maryland Department of Transportation

Loutzenheiser, Donald W.
Chief Highway Engineer
Federal Highway Administration

McCoy, William D.
Assistant State Highway Urban Engineer
Georgia Department of Transportation

Mueser, Robert R.
Deputy Chief Highway Engineer
Pennsylvania Department of Transportation

Pennington, Gordon R.
Sverdrup & Parcel Associates, Inc.

Randich, G. M.
Vice President
Deleuw, Cather & Company

Sigal, Andre H.
Associate Civil Engineer
New York State Department of Transportation
Region 10 Office

Operations

Taragin, A.
Traffic Performance and Analysis Division
Office of Traffic Operations
Federal Highway Administration

Academic - Research

Glennon, John C.
Midwest Research Institute

Pilkington, George
Federal Highway Administration
Environmental Design and Control Division

1. Please list in order of importance the most critical factors used in determining the basic geometrics of a major interchange.

Factors:

a. Traffic Mainline Volumes (Design Hour)
b. Site conditions - environment, topography
c. Standards and capacity of crossing freeways (Level of Service; use of direct vs. loop, left vs. right on-off)
d. Economics
e. Project Objectives
f. Weaving distances, grades, acceleration-deceleration lanes
g. Simplicity of design
h. Urban vs. rural
i. Right-of-way
j. Route and lane continuity and lane balance (freeway turns)
k. Design speed
l. Sight distance
m. Spacing between adjacent interchanges
n. Safety
o. Operational characteristics
p. Ability to obtain community approval (political constraints)
q. Signing
r. Effect abutting properties (socio-economic effects)
s. Appearance
t. Interchanging traffic volumes
u. Ramp design speeds
v. Lane drops on right
w. Land uses and development
x. Angle of intersection
y. Construction controls (funding, schedules, maintenance of traffic)
z. Alignment of approaches to points of divergence.
za. Percent of heavy trucks (composition of traffic).

Note: Factors were provided by experts; therefore number of factors varied for each individual.

Number of Experts (out of 21) Ranking Factors

FACTORS	RANK 1	2	3	4	5	6	7	8	9	10	11	12
a	6	4	4									
b	1	2	4	1	1	1						
c	5	3	1	2	1							
d			3	3			1					
e			1									
f				1								
g				1								
h	1	1										
i	2	1	2	2			1					
j	1	2		1								
k	1	3		1								
l		1										
m					2							
n	2	1	1									
o			2			1						
p						1						
q			1	1		1						
r				1		2						
s								1				
t	1	1			1							
u		1										
v			1									
w		1										
x	1											
y					1							
z						1						
za			1	1								

Note: While some categories might be combined, the actual wording of the answers suggest significant or at the least, discernible differences between most of the categories.

2. One of the principal goals of this major interchange study is to determine the differences in the design approach and design procedures for freeway-to-freeway interchanges compared to other interchanges (i.e., freeway-to-arterial, expressway-to-local highway, freeway-to city street, etc.) Please state your opinion regarding the major differences between design of a major interchange and other (non-major) interchanges. Please list the differences in order of importance.

A.

1. Major difference is that left turns (intersections) are eliminated at freeway-to-freeway interchanges.
2. Major interchanges must accommodate higher percentage of unfamiliar drivers - greater driver expectation for higher standards.
3. Major interchange design should allow more flexibility for change due to traffic projection inaccuracies.
4. In urban areas, major interchanges should be in balance with the entire freeway network - must look at total picture.
5. Major interchange design is more complex, more costly, has greater impact on the community and the freeway system than does a local interchange. A greater effort is necessary to overcome greater problems.

INDIVIDUALS

B.

1. Traffic volumes
2. Design speed
3. Level of service
4. Safety features, i.e., width of clear area, rate of slopes
5. Impact attenuators

C.

1. This, as above, cannot be given a particular number of importance. A chain is no stronger than its weakest link; therefore, if a major interchange on a particular facility is the ultimate in design features and the freeway-to-city streets is poor design; the facility's general operation is poor due to the freeway-to-city street "bottleneck." Every I/C on a facility or a system should be designed to insure that the overall system is sufficient.

D.

1. Higher ramp speeds must be maintained.
2. Initial consideration should be for minimum two lane direct connections.
3. Interchange locations, must give consideration of additional right-of-way requirements.

E.

1. Major interchange must provide continuous movements for all strands of traffic.
2. All turning roadways of a major interchange must provide minimum reduction in design speed from that of mainline roadways.
3. Directional design of major interchanges may result in left side terminals. There should be none in other interchanges.
4. There may be a reduction in number of mainline lanes through a major interchange. There should be no lane drops through other interchanges.
5. Major interchanges often require C-D roads or auxiliary lanes to facilitate weaving and turning movements.

F.

1. Higher level of service
2. Higher speed design
3. Operational characteristics can be less strict for a non-major interchange (merging, weaving, direction of movement, signing).
4. Local service to existing streets should be goal of minor interchange.
5. Different configuration of major interchanges and minor interchanges depending upon type of traffic (i.e., thru or local traffic).

G.

1. Major interchanges usually require direct connections to provide a high level of service. Minimum reduction in freeway design speed desirable.
2. Minor connections should be eliminated, such as service to local roadways and U-turn facilities.
3. Weaving areas - undesirable.
4. Major volumes should be designed as a through traffic move preferably with the minor moves using right hand off.

M.

1. Level of service - lower levels of service are tolerated on non-major interchanges before higher type facilities (directional ramps, C-D roads) are added.
2. Minimum interchange type - with (4-leg) major interchanges, a cloverleaf is the assumed minimum type before item (1) steps (a) to (g) are applied to determine maximum needs. For all other cases, a diamond type is used as starting point.

   Note: item (1) etc. refers to answer for Question 1.

N.

1. Characteristics of the traffic. That is, a major interchange handles traffic from 2 high speed freeway type facilities. Lesser interchanges handle traffic entering or leaving a roadway where traffic speed may be lower, interruptions to traffic may be frequent, at grade intersections exist, etc.

2. With a major interchange, free-flowing traffic through the interchange ramp is desirable. With lesser interchanges, ramp terminals at the crossroads may be other than free-flow.

3. Major interchanges direct attention to through traffic. Lesser interchanges should consider local traffic demands to a far greater extent.

O.

1. Higher interchanging volumes.

2. All ramp terminals must be of directional type (no traffic conflicts can be tolerated).

3. Design speeds are higher on both mainlines and therefore ramp design speeds should be higher.

4. Major interchanges generally involve multiple structures and/or multi-level structures.

P.

1. All maneuvers are free-flowing on major interchange, not usually required on lesser interchanges.

2. Higher speeds are normally accommodated in major interchanges and the operational integrity of both freeway facilities is of major importance whereas on less significant interchanges, nominal sacrifice on the lesser roadway may be justified.

3. Lane balance and continuity are of extreme importance on major interchanges.

4. No surprises -- this is especially true with higher operating speed - major interchanges.

5. Excellent signing required -- the more complex the interchange and higher the operating speeds the more critical this becomes.

6. Compound weaving must be avoided in major interchanges. This is undesirable on all interchanges, however, it may be tolerated on less significant interchanges with lower volumes and lower operating speeds.

R.

1. Design approach -- same for all interchanges.

Research

A. No response

M.

1. Higher turning volumes.
2. Desire to maintain level of service.

FHWA

A. No response

B.

1. Basic design is not really different or the procedures used.
2. Major has higher speed, free-flowing ramps as opposed to diamond ramps or loops. This entails greater lengths, added lanes, longer terminals, larger sight distance, etc. that tends to approach through traffic operational layouts. May be C-D roads to separate weaving and frequent access.
3. Spatially large and a greater effect on the site area. Layout results in sizeable open areas, extracted from valuable development potential land which must be made into attractive, fitting open space and efforts made to include appropriate joint development or other community aiding uses.
4. Freeways are alike so design for two does not have to be compromised to fit the limitations and operational restraints of the normal crossroad, often existing, of a lower type.

C.

1. Major necessitates a free-flow type ramp, therefore many interchange types are eliminated. The choice is basically a full cloverleaf, a directional, or combination thereof.
2. Signing is more critical for major interchange designs because more or most drivers must make decisions at major forks.
3. Alignment or view of road approaching major decision points is a prime item for consideration in design of major forks (no sign is a substitute for being able to see the roadway pavement ahead).
4. Traffic - necessary to determine the type ramps and number of lanes. Less weight should be given to traffic as compared to achieving good "lane balance" at ramp terminals and "lane continuity."

Consultants

A.

1. For major interchanges, all traffic should be considered as through moving traffic regardless of direction of movements within interchange.
2. All traffic should be considered as operating under high speed conditions without interruption.
3. No local access or ramps should be provided.
4. Greater emphasis must be placed on traffic flow continuity and less on sheer capacity.
5. Weaving, merging and stop conditions should be avoided.

B.

1. High speed geometrics.
2. Greater safety demands dictated by the higher speeds.
3. Direct ramps (not loops).
4. No local access to streets/roads.

M.

1. Design speed required on turning roadways and terminals.
2. Larger volumes of traffic to accommodate (2-lane ramps, perhaps) (direct connections).
3. Maintenance of existing traffic.
4. Lighting, signing.

N.

1. Recognition route continuity characteristics.
2. Design for higher speeds and operating conditions.
3. Complexity.
4. Adaptability of design to accept signing.
5. Spatial requirements.
6. Dollars invested.

P.

1. Provision of directional connectors.
2. Elimination of "surprise" elements. Design should conform to driver expectancy.
3. Design speed on ramps should be within 20 mph of the approach speed.

R.

1. Facility should be capable of operating at or close to the operating speeds of the through roadways.
2. Facility should be capable of handling the volume and composition of traffic both in the through and turning lanes.
3. Facility should "blend" with the area; i.e., it should not present the driver with any conditions he would not expect to find in the area.
4. Facility should maximize safety demands.
5. Should demand more "fore-thought" and sensitivity to non-highway demands than has been exercised in the past.

3. Using as a reference the Intersection Design Procedure as summarized in the AASHO "Blue Book" (pages 603-4), do you feel the design of a major interchange is sufficiently different from the design of other (non-major) freeway interchanges to make a separate design procedure for major interchanges either necessary or desirable?

Choices	Number of Participants Selecting Given Answer	%
a. There is a definite necessity to have a separate procedure for the design of major interchanges	5	23.8
b. The same procedure can be used for both types of interchanges but it would be desirable to have a separate procedure for major interchanges	7	33.3
c. The same procedure should be used for major and non-major interchanges	9	42.9
Total Number Responding	21	100

Comments:

Major interchanges should continue to be "custom" designed while the use of uniform designs (more diamonds) should be the objective for non-majors.

The same procedure should be used with different standards.

The same procedure should be used although the study of major interchanges must be in greater depth. Some steps may be eliminated or curtailed for non-major interchanges.

The same procedure should be used but the extent and details differ.

4. In a state funded project, how often do political or public pressures in your state dictate that local ramps be designed into a major interchange?

	Almost never (0-5%)	Sometimes (6-35%)	Often (36-65%)	Usually (66-95%)	Almost always (96-100%)	Total Number Responding
Number Responding	4	11	1	1	1	18
Percent	22.1	61.1	5.6	5.6	5.6	100

Comments:

I think that practically all designers know better, it is just that we do not know how to resist the pressures.

5. In a federal/state funded project requiring the two stage public hearings, how often do such hearings result in the requirement to design local ramps into a major interchange?

	Almost never (0-5%)	Sometimes (6-35%)	Often (36-65%)	Usually (66-95%)	Almost always (96-100%)	Total Number Responding
Number Responding	5	11	2	0	0	18
Percent	27.8	61.1	11.1	0	0	100

Comments:

Our public hearings requirements are the same for 100% state as 50-50 projects on Interstate.

Most pressures are brought before the public hearing. The hearings are often a formality.

I have not experienced such requirements because of limited public hearing involvement regarding major interchanges.

6. From a design and operations standpoint, how often do you think local access ramps should be permitted in a major interchange?

	Almost never (0-5%)	Sometimes (6-35%)	Often (36-65%)	Usually (66-95%)	Almost always (96-100%)	Total Number Responding
Number Responding	16	3	0	0	0	19
Percent	84.2	15.8	0	0	0	100

Comments:

This is dependent on location and area needs. If needs can be satisfied within reasonable distance, then no local connections should be permitted.

Confusion and erratic operations often result from local ramps in major interchanges due to signing difficulties. Where proper signing can be provided a local ramp would be acceptable, but this is rarely possible.

Sometimes, but they must not reduce the effectiveness of interchange movements.

Local access should be planned for and gained from properly spaced interchange with local roads.

Costs and signing, etc. complications indicate we should avoid mixing freeway-to-freeway interchanges with local interchanges. This is not always reasonable in urban situations and there are acceptable way to combine the two types.

I think that practically all designers know better, it is that we do not know how to resist the pressures.

This should be based on traffic desire flow and not a "fixed" figure. You may recall that the old BPR had a fixed number of ramps and distance spacing between interchanges which created "bottlenecks" at the few ramps provided and created a "Chinese Wall" through urban areas.

Occasionally special geometric conditions may prevail where operations would not be hampered by the introduction of a local ramp connection. A local ramp connection to a low volume interchange would be a case in point

Local access ramps always create operational and signing problems when incorporated into major interchanges. They cause driver confusion and therefore safety problems. Local ramps should thus rarely be incorporated into a major interchange and should be avoided within the influence zone (1 mile ±) of a major interchange.

Problems have arisen when such is permitted. European practice bears this out.

Local access should only be permitted in major interchanges where there is no other way to feasibly allow access, and projected local access traffic volumes would not force the facility to operate below level of service "A".

7. In some instances local ramps located within a major interchange are selectively closed during peak periods. In your opinion, is selective closure of ramps during peak periods a practical solution to capacity-related operational problems which may exist?

	Almost never (0-5%)	Sometimes (6-35%)	Often (36-65%)	Usually (66-95%)	Almost always (96-100%)	Total Number Responding
Number Responding	5	4	6	3	2	20
Percent	25.0	20.0	30.0	15.0	10.0	100

Comments:

This is not recommended -- more satisfying and permanent solutions should be sought.

This depends on alternate connections or facilities within the corridor and whether traffic could be adequately redirected.

If such is anticipated prior to construction, the ramps should not be built.

Selective closure of ramps is practical only if acceptable alternate routes are available during the period of closure. The community must be ready to accept this type of restraint.

Selective closure of ramps during peak periods is an effective solution for existing conditions, but generally should not be designed for that condition on new freeway designs.

Since it assumed the problem exists (probably from poor planning), selective closure often is a practical remedy in large metropolitan areas.

Almost never -- metering should be considered.

We should use this experience to resist building more local service ramps too near major interchanges.

A better solution would be to eliminate the ramp entirely and require some vehicles to travel greater distances on local roads. Local opposition sometimes makes this solution impractical.

Local ramps are not closed during peak periods in any interchanges in the state of Maryland. There are only two local off-ramps in Maryland. No need to close in peak hour.

The ramp should be removed permanently if practical, without severe damage to local businesses.

This will help keep the traffic flow of the mainline at an acceptable level. Local traffic conversely will be more congested.

If a ramp is not usable during peak hours it has questionable value in the system.

Basic objective of a major interchange is to effect a freeway-to-freeway movement. This function should be safeguarded particularly the main roadway is severely congested and local ramps are hampering traffic operations.

Once a ramp is constructed and opened to traffic, it should not be intermittently closed. This type of closure causes driver confusion and related safety problems. This is especially true in the case of local exit ramps, more latitude is possible with entrances.

This is only a stop-gap solution.

A better method would be to close them permanently.

8. In reviewing the literature relevant to major interchange design and operations, it was found that a number of authors proposed some standardization of geometric design so that the driver is faced with a more constant exiting/entry situation. In your opinion is the standardization of geometric design of major interchanges feasible and practical for the situations listed below?

Urban Interchange

	Feasible?		Practical?	
	No. Responding	Percent	No. Responding	Percent
Yes	12	63.2	8	38.1
No	7	36.8	13	61.9
Total	19	100.0	21	100.0

Rural Interchange

	Feasible?		Practical?	
	No. Responding	Percent	No. Responding	Percent
Yes	15	78.9	13	61.9
No	4	21.1	8	38.1
Total	19	100.0	21	100.0

Comments:

Standardization, yes; uniformity, no (all "no" responses above).

It is related to the siting situation (all "yes" responses).

I assume we are talking about terminal configurations and not the actual shape of the ramps (all "yes" responses).

The word some is underlined. Standardization or uniformity should rate high in the priority of design considerations, but a "cookbook" approach is not feasible, especially for major interchanges.

9. If you answered "no" to any part of question #8, please list, in order of importance, those factors which argue against the feasibility or practicality of standardizing the geometric design of major interchanges. In listing the factors, please indicate the relevant situation as follows: UF = Urban Feasibility; UP = Urban Practicality; RF = Rural Feasibility; RP = Rural Practicality.

Urban Feasibility:

When thinking of standard design what comes to mind is the single exit, single entrance design connected to collector-distributor roads. This is a desirable thing to do but is not always feasible in heavily built up urban areas because of the added right-of-way required.

Usually a limited area is available and interchanges are tailor-made to fit existing conditions. The economical aspect is probably the biggest deterrent in standardization.

The proximity of exit and entrance terminals in large metropolitan areas often makes it infeasible to shift certain terminals just for uniformity. Also the need for more multi-lane directional ramps (major forks) in these areas and ramp splits reduces the feasibility in these areas.

In urban areas factors other than a standardized configuration assume such magnitude that it is necessary to design each interchange to fit the particular circumstances.

Proper signing negates the need for standardization since strangers depend entirely on signing, not geometrics. There are also terrain, topographical, and right-of-way restrictions.

Right-of-way attainability
Environmental and social effects
Influence of adjacent interchanges
Time lapse in system development.

Urban Practicality:

Right-of-way restrictions
Economics

Due to restrictions of development and right-of-way costs and variations in traffic needs and street patterns, the complete standardization of urban interchanges is not practical.

Many times in urban areas the land is simply not available to standardize design.

Usually a limited area is available and interchanges are tailor-made to fit existing conditions. The economical aspect is probably the biggest deterrent in standardization.

Our studies (Ill.) have shown that alternative designs to accomplish uniform exits and entrance have all involved additional C-D roads, greater structure costs, more right-of-way, etc., to the point where it was not considered the optimum design.

In urban areas factors other than a standardized configuration assume such magnitude that it is necessary to design each interchange to fit the particular circumstances.

Right-of-way costs involved in the more desirable concept are sometimes so excessive that compromise is necessary. Special traffic conditions may on selected occasions indicate the desirability to construct non-standard configurations. Such situations are isolated in occurrence (certain non-standard configurations may eliminate or minimize weaving, etc. -- depending upon the proximity of other entrances and exits).

Economics, due to dense development in urban areas, makes standardization unrealistic.

Proper signing negates the need for standardization since strangers depend entirely on signing, not geometrics. Standardization would result in higher right-of-way and higher construction costs.

Right-of-way constraints
Physical restrictions
Cost

Right-of-way attainability
Environmental and social effects
Influence of adjacent interchanges
Time lapse in system development

Rural Feasibility:

If all interchanges required the same type of connections and were at a similar crossing angle, it might be possible. But topographical features sometimes dictate changes in design from a more desirable type interchange.

Proper signing negates the need for standardization since strangers depend on signing, not geometrics. There are also terrain, topographical, and right-of-way restrictions.

Right-of-way attainability
Environmental and social effects
Influence of adjacent interchanges
Time lapse in system development.

Rural Practicality:

Right-of-way restrictions
Economics

If all interchanges required the same type of connections and were at a similar crossing angle, it might be possible. But topographical features sometimes dictate changes in design from a more desirable type interchange.

Our studies (Ill.) have shown that alternative designs to accomplish uniform exits and entrances have all involved additional C-D roads, greater structure costs, more right-of-way, etc., to the point where it was not considered the optimum design.

While it may be feasible to standardize geometric design for rural interchanges because of fewer restrictions due to existing land use, it is not practical. A better design, and often a more economical one, will result from tailoring the design to the circumstances.

Proper signing negates the need for standardization since strangers depend entirely on signing, not geometrics. Standardization would result in higher right-of-way costs and higher construction costs.

Right-of-way attainability
Environmental and social effects
Influence of adjacent interchanges
Time lapse in system development

General Comments:

Parts of any freeway-to-freeway interchange can and should be standardized (exiting and entering, etc.). I doubt benefit to any "standardization" of the interchange as a whole -- no two situations are ever the same. A good line must be drawn between standardization for consistency which is desirable and standardization that will stifle "thinking."

It should be noted that while entire interchanges should not be standardized, the various components that make up an interchange are normally standardized, thus maintaining consistency in design.

10. In your organization how frequently are loop ramps used in new (i.e., recently designed or constructed) freeway-to-freeway interchanges for major turning movements?

	Almost always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost never (0-5%)	Total Number Responding
Number Responding	1	0	0	14	3	18
Percent	5.5	0	0	77.8	16.7	100

Comments:

Almost always where single lane capacity is not exceeded and the route does not turn.

11. In your organization how frequently are loop ramps used in new freeway-to-freeway interchanges for minor turning movements?

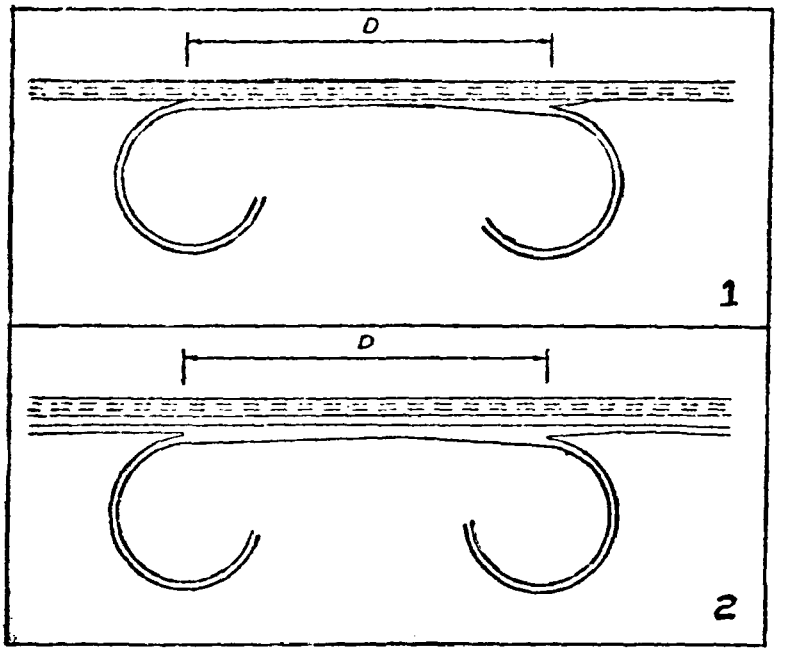
	Almost always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost never (0-5%)	Total Number Responding
Number Responding	1	6	6	4	1	18
Percent	5.6	33.3	33.3	22.2	5.6	100

Comments:

Almost always where single lane capacity is not exceeded and the route does not turn.

Almost never would be a preference if given such a choice. Combinations of loops with diamond ramps in some quandrants are even a lower choice, except for possibly the Parclo "A", 4-quandrant design where left turn desire along the crossroad is so heavy that signalization is not possible.

12. Figure 1 in the "Figures Package" shows 2 adjacent loop ramps, without a collector-distributor road, and Figure 2 shows the same arrangement, with a collector-distributor road. Place an "X" under the use category, which characterizes the frequency with which your organization uses these designs for freeway-to-freeway interchanges, under the following conditions:



USE CATEGORY

	CONDITION		Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total Number Responding
a.	Fig. 1 Rural Areas	No. Responding	2	1	4	7	3	17
		%	11.8	5.9	23.5	41.1	17.7	100
b.	Fig. 1 Suburban Areas	No. Responding	0	3	3	4	7	17
		%	0	17.7	17.7	23.5	41.1	100
c.	Fig. 1 Urban Areas	No. Responding	1	0	3	3	9	16
		%	6.3	0	18.7	18.7	56.3	100
d.	Fig. 1 Max. Vol. -- each loop ramp < 100 vph	"	5	2	1	4	4	16
			31.3	12.5	6.2	25.0	25.0	100
e.	Fig. 1 Max. Vol. -- each loop ramp 100-300 vph	"	1	5	1	4	4	15
			6.7	33.2	6.7	26.7	26.7	100
f.	Fig. 1 Max. Vol. -- each loop ramp 300-500 vph	"	1	2	4	3	7	17
			5.9	11.8	23.5	17.7	41.1	100
g.	Fig. 2 Rural Areas	"	2	0	4	7	4	17
			11.8	0	23.5	41.2	23.5	100

CONDITION		USE CATEGORY: Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total Number Responding
h. Fig. 2 Suburban Areas	No. Responses	1	3	3	9	1	17
	%	5.9	17.6	17.6	53.0	5.9	100
i. Fig. 2 Max. Vol. -- each loop ramp < 100 vph		1	3	7	2	3	16
	"	6.3	18.7	43.8	12.5	18.7	100
j. Fig. 2 Max. Vol. -- each loop ramp < 100 vph		1	0	0	5	10	16
	"	6.2	0	0	31.3	62.5	100
k. Fig. 2 Max. Vol. -- each loop 100-300 vph		0	2	1	8	4	15
	"	0	13.3	6.7	53.3	26.7	100
l. Fig. 2 Max. Vol. -- each loop ramp 300-500 vph		2	2	5	6	2	17
	"	11.8	11.8	29.3	35.3	11.8	100

Comments:

Our (Illinois) use of Figure 2 in rural areas would generally be for weaving capacity needs, whereas in urban areas it would generally result from a preplanned C-D road to collect closely spaced ramps. However, without preplanned C-D, it is also used for capacity more often in urban than rural areas.

In Maryland directional ramps are used in urban areas.

The above represents past practice in our organization (New York). It is assumed that, with publication of our new design manual, the use of configuration 1 will virtually disappear for freeway-to-freeway interchanges.

13. Indicate the minimum and desirable distance D between entrance and exit nose for figures 1 and 2 under the following conditions. The answer is to reflect your opinion as a designer and not necessarily the values presented in your state design manual, blue-book, red-book, etc.

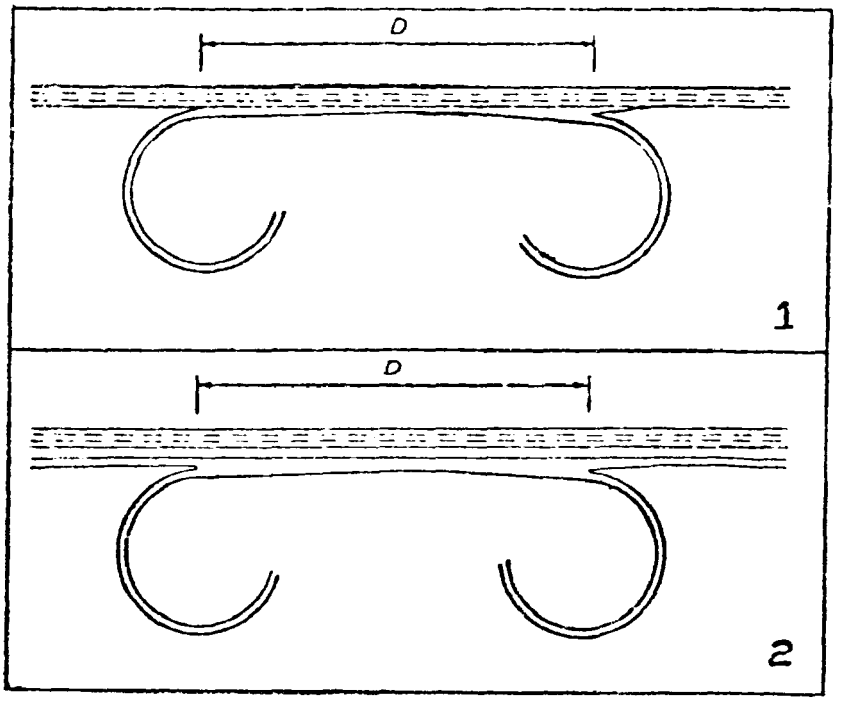


Fig. 1 Mainline Design Speed 50 mph

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
400	1	5.0	5.0
450	1	5.0	10.0
500	3	15.0	25.0
600	1	5.0	30.0
700	3	15.0	45.0
800	4	20.0	65.0
900	2	10.0	75.0
1000	2	10.0	85.0
1400	1	5.0	90.0
1500	1	5.0	95.0
1700	1	5.0	100.0
Total	20	100.0	

Comments:

A 4-quadrant cloverleaf is obviously not practical for these distances (1400 min., 2000 desirable). Today at 4-quadrant cloverleafs we would provide C-D roads.

700 ft., subject to weaving criteria (quality of flow II for fig. 1 and III rural or IV urban for fig. 2).

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
600	1	5.9	5.9
700	1	5.9	11.8
800	2	11.7	23.5
900	1	5.9	29.4
1000	5	29.4	58.8
1200	2	11.7	70.5
1300	1	5.9	76.4
1500	1	5.9	82.3
2000	3	17.7	100.0
Total	17	100.0	

Comments:

The distance is based on weaving volumes (no specific answer given).

A 4-quadrant cloverleaf is obviously not practical for these distances (1400 min., 2000 desirable). Today at 4-quandrant cloverleafs we would provide C-D roads.

Should not exist.

Fig. 1 Mainline Design Speed 70 mph

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
450	1	5.3	5.3
500	2	10.5	15.8
700	1	5.3	21.1
800	1	5.3	26.4
1000	4	21.0	47.4
1200	3	15.7	63.1
1300	2	10.5	73.6
1400	1	5.3	78.9
1800	1	5.3	84.2
2000	2	10.5	94.7
2300	1	5.3	100.0
Total	19	100.0	

Comments:

Distances greater than 800-1000 are beyond the realm of practical loop design.

A 4-quadrant cloverleaf is obviously not practical for these distances (1400 min., 2000 desirable). Today at 4-quadrant cloverleafs we would provide C-D roads.

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
900	1	5.9	5.9
1000	4	23.5	29.4
1200	1	5.9	35.3
1400	1	5.9	41.2
1500	3	17.6	58.8
1600	1	5.9	64.7
1700	1	5.9	70.6
2000	3	17.6	88.2
2600	1	5.9	94.1
3000	1	5.9	100.0
Total	17	100.0	

Comments:

1000 ft. would require spreading the crossroad lanes or very large radii on the loops.

The distance is based on weaving volumes (no specific answer given).

A 4-quadrant cloverleaf is obviously not practical for these distances (1400 min., 2000 desirable). Today at 4-quadrant cloverleafs we would provide C-D roads.

Distances greater than 800-1000 are beyond the realm of practical loop design.

Should not exist.

Fig. 2 C-D Road Design Speed 35 mph

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
225	1	5.0	5.0
250	1	5.0	10.0
300	1	5.0	15.0
400	3	15.0	30.0
450	1	5.0	35.0
500	3	15.0	50.0
600	5	25.0	75.0
700	4	20.0	95.0
1000	1	5.0	100.0
Total	20	100.0	

Comments:

700 ft., subject to weaving criteria (quality of flow II for fig. 1 and III rural or IV urban for fig. 2).

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
400	1	5.6	5.6
450	1	5.6	11.2
500	2	11.1	22.3
600	3	16.6	38.9
700	2	11.1	50.0
800	4	22.2	72.2
900	2	11.1	83.3
1000	2	11.1	94.4
2000	1	5.6	100.0
Total	18	100.0	

Comments:

The distance is based on weaving volumes (no specific answer given).

Fig. 2 C-D Road Design Speed 50 mph

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
400	1	5.0	5.0
450	2	10.0	15.0
500	3	15.0	30.0
600	4	20.0	50.0
700	2	10.0	60.0
800	5	25.0	85.0
900	1	5.0	90.0
1000	2	10.0	100.0
Total	20	100.0	

Comments:

500+. If you are referring to long C-D roads that go through several local interchanges, then the distances should approach 1400 ft. min. and 2000 ft. desirable.

Distances greater than 800-1000 are beyond the realm of practical loop design.

700 ft., subject to weaving criteria (quality of flow II for fig. 1 and III rural or IV urban for Fig. 2).

Desirable Distance D (ft.)	No. Responding	Percent	Cumulative %
500	1	5.6	5.6
600	2	11.1	16.7
750	1	5.6	22.3
800	1	5.6	27.9
900	2	11.1	39.0
1000	5	27.7	66.7
1200	4	22.1	88.8
1300	1	5.6	94.4
2000	1	5.6	100.0
Total	18	100.0	

Comments:

The distance is based on weaving volumes (no specific answer given).

500+. If you are referring to long C-D roads that go through several local interchanges, then the distances should approach 1400 ft. min. and 2000 ft. desirable.

Distances greater than 800-1000 are beyond the realm of practical loop design.

General Comments:

Weaving and volumes will determine length.

An arbitrary minimum can be chosen but the desirable should be equal to or greater than the length required for weaving.

14. Regarding the values used in answering #13, please indicate below whether they differ from values recommended in your state manual.

	Yes	No	Not Covered in Manual	Total Number Responding
Number Responding	5	3	10	18
Percent	27.8	16.6	55.6	100

Comments:

No State Manual (N.J.).

Yes - Manual does not cover fig. 1, but covers fig. 2.

Yes and not covered in manual (dual response).

No or not covered in manual (dual response).

Yes, slightly higher than the AASHO Blue Book.

15. What, in your opinion, would be the maximum desirable weaving volume in vehicles per hour for the arrangements shown in:

in Figure 1 _____vph, in Figure 2 _____vph.

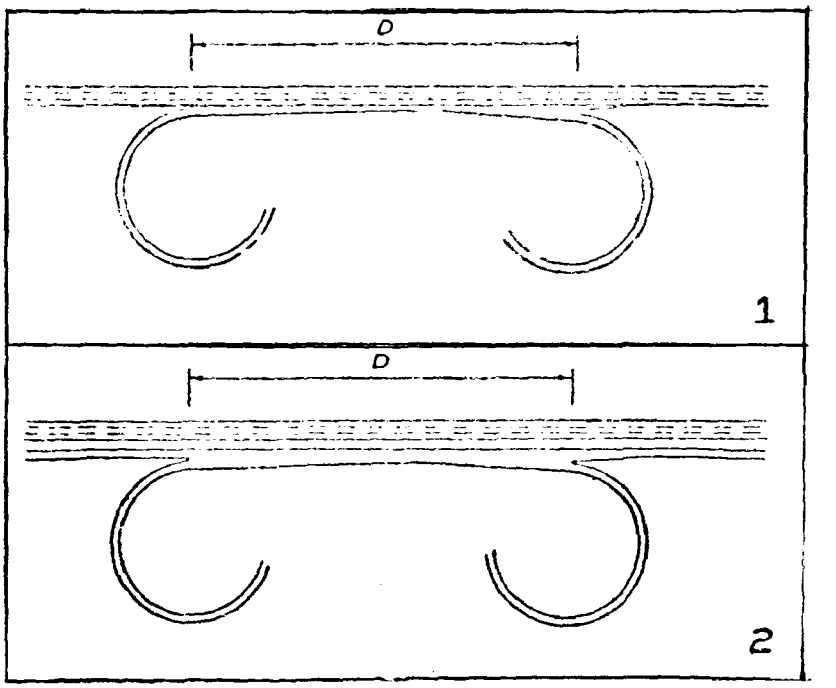


Figure 1 - Maximum Desirable Weaving Volume (vph)	Number Responding	Percent	Cumulative Percent
0	2	11.1	11.1
400	2	11.1	22.2
500	2	11.1	33.3
600	1	5.6	38.9
800	2	11.1	50.0
1000	5	27.6	77.6
1100	1	5.6	83.2
1400	1	5.6	88.8
1500	1	5.6	94.4
2000	1	5.6	100.0
Total	18	100.0	

Figure 2 - Maximum Desirable Weaving Volume (vph)	Number Responding	Percent	Cumulative Percent
600	1	5.9	5.9
800	1	5.9	11.8
1000	4	23.5	35.3
1500	6	35.2	70.5
1600	3	17.7	88.2
2000	2	11.8	100.0
Total	17	100.0	

Comments:

These figures (1000, 2000) could vary depending on whether a heavier volume occurred entering or exiting.

Fig. 1 is not desirable at any volume on a freeway.

No answer. The maximum weaving volume would depend on the resultant. length. As a rule of thumb, I like 1 vehicle per 1 foot of length for non-C-D and 1.5 vehicles per foot of length for C-D roads.

The maximum volume of a loop is 800 vph. This is the primary reason loop ramps are not used in Texas. Our new design manual states that all loop ramps will have C-D roads. I do not believe that any loop pair can actually handle 1600 vph. with or without C-D roads.

800 and 2000 assuming level grades and a large urban area.

No answer - depends on length and layout.

500 and 800 given as answers. D should be known to estimate weaving volume. From 0 to 1000 vph. when D = 0 and D = 1000 respectively. Also speed should be known.

800 and 1000 given as answers. Influence of main roadway weaves or "late weaves" would have a volume reduction effect.

Weaving on freeway mainlines should be avoided (fig. 1 = 0 vph). On C-D roads, maximum weaving volumes will depend on the length of weaving section and must be analyzed on a case-by-case basis.

Volumes will depend on value of D and average speed (500 and 1000 vph for 50 mph operating speed).

16. Regarding the values used in answering question #15, please indicate below whether they differ from values recommended in your state manual.

	Yes	No	Not Covered in Manual	Total Number Responding
Number Responding	2	1	13	16
Percent	12.5	6.2	81.3	100

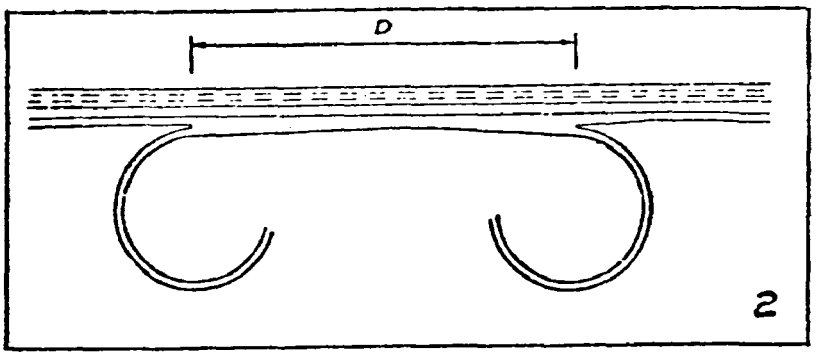
Comments:

No State Manual (N.J.).

Yes - Figure 1 not covered in manual. Figure 2 agrees with manual.

No and C-D road not covered in manual (dual response).

17. In designing major interchanges, is uniformity sufficient justification for using the arrangement in Figure 2 at all locations with adjacent loop ramps?



	Number Responding	Percent
Yes	10	47.6
No	11	52.4
Total Number Responding	21	100.

Comments for "yes" responses:

Figure 1 is unacceptable.

The maximum volume of a loop is 800 vph. This is the primary reason why loops are not used in Texas. Our new design manual states that all loop ramps will have C-D roads. I do not believe that any loop pair can actually handle 1600 vph. with or without C-D roads.

When thinking of standard design, what comes to my mind primarily is single exit, single entrance design connected to collector-distributor roads. This is a desirable thing to do but is not always feasible in heavily built-up urban areas because of the added right-of-way required.

If costs are not excessive.

If loop type ramps must be employed, Figure 2 should be minimum acceptable design.

It is desirable to use single exit designs for all interchanges. This can be accomplished through the use of C-D roads at cloverleafs.

Only if uniformity is being sought to conform with "driver expectancy," not uniformity for the sake of uniformity.

Comments for "no" responses:

I do not think uniformity is an exclusive criteria upon which to make this decision. Admittedly, uniformity offers some very significant benefits.

This condition would usually occur at low volume ramps at a major interchange.

Uniformity of both geometrics and signing are important considerations. However, each interchange must be designed on its own merit, and at times Figure 1 may be appropriate.

Our studies have led us (Illinois) to conclude that it is not practical to do so just for uniformity. There are other priorities which in our (my) opinion are more cost effective.

Where volumes are extremely low and the loop (free flow) design is provided only because of a freeway-to-freeway condition, the C-D road should be omitted. The C-D roads and related overhead signs can cause confusion to the mainline driver, not to mention the cost where not warranted for weaving.

Economics are still involved.

Loops with weaving should not be used in a major interchange.

Uniformity is desirable, but with lesser traffic the collector road is not required.

18. In your opinion what is the most appropriate minimum design speed for turning roadways (ramps) on major interchanges? Please provide an answer for both major and minor traffic movement.

Major movement _____mph

Minor movement _____mph

Minimum Design Speed (mph) Major Movement	Number Responding	Percent	Cumulative Percent
35	1	5.3	5.3
40	1	5.3	10.6
45	3	15.8	26.4
50	12	63.1	89.5
60	2	10.5	100.0
Total	19	100.0	

Minimum Design Speed (mph) Minor Movement	Number Responding	Percent	Cumulative Percent
25	2	10.5	10.5
30	7	36.9	47.4
35	4	21.1	68.5
40	2	10.5	79.0
45	2	10.5	89.5
50	2	10.5	100.0
Total	19	100.0	

Comments:

50,30 mph -- At major interchange higher speeds may be desirable in both categories assuming other factors such as right-of-way cost, adverse travel distance, additional construction cost, etc. do not out-weigh the benefits.

50,50 mph -- for 70 mph on the mainline, the desirable minimum design speed for turning roadways should be 60 mph. Obviously, the use of loop ramps is ruled out.

35,20 mph -- If directional ramps are used, minimums should be 40 mph.

No answer -- Urban multi-lane directional ramp = 60 mph.
Urban multi-lane semi-directional ramp = 40 mph
Rural multi-lane directional ramp = 70 mph
Rural multi-lane semi-directional ramp = 70 mph
In rural areas, I believe it is highly desirable to design all multi-lane directional ramps to eliminate the need for speed-zoning.

50,25 mph -- Major - 850 ft. radius or 50 mph is considered a good fit considering safety, capacity, cost, etc. Minor - 150 ft. radius or 25 mph for loops (150 ft. radius would not be advocated on a connection other than a loop).

50 and 40 mph would be for directional-type ramps, not loops. Loops should be maintained at 225 ft. to 300 ft. radii in the interest of economy and uniformity. Providing a 45-50 mph design speed would not be economical and would invite drivers to expect the same operating conditions at the many 150-225 ft. radii loops in existence.

No answer - The minimum design speed should be at least 0.7 of the main lane design speed. We generally try to obtain higher design speeds than minimum. The minimum length of an exit ramp should be based on the minimum stopping distance of the main lane design speed.

50,35 mph - The major movement should be as near the through movement as practical.

50,35 mph - It would be desirable to have the same design speed as the mainline on major traffic movements.

45 to 60 mph for both major and minor movements (45 used in tabulations above). For freeway-to-freeway movements, a basic philosophy should be to maintain speeds as near to mainline speeds as practical. A 10 mph reduction in speed, in my view, could be acceptable. This matter should be related to operating speeds in an urban or rural situation. For example, some urban areas restrict mainline speeds to 55 mph or less. These comments exclude consideration of loop-type ramps.

45,30 mph - These are minimums based on a freeway design speed of 70 mph; higher speeds are desirable.

50,40 mph - The design speed for turning movements should be related to the design speed of the through movement. A maximum of 20 mph difference in speed should be provided.

50,50 mph for 70 mph mainline. Use 40 mph for 60 mph mainline and 30 mph for 50 mph mainline.

45,30 mph - These speeds are or should be dependent upon the design speeds of the through lanes:

1) Speed of the major movement be no less than 10 mph lower than the fastest through lane, but not less than stated above (45 mph).

2) Speed of the minor movement should be no less than 25 mph lower than the fastest through lane, but not less than stated above (30 mph).

19. In your opinion, under which of the following conditions should a left turning movement be permitted from the left or high speed lane in a major interchange?

		Almost always	Sometimes	Almost never	Total number responding
a. Left turn volume, 10% of total volume	No. responding	1	1	19	21
	%	4.8	4.8	90.4	100
b. Left turn volume, 30% of total volume	No. responding	1	6	14	21
	%	4.7	28.6	66.7	100
c. Left turn volume, 50% of total volume	"	6	10	5	21
		28.6	47.6	23.8	100
d. Left turn volume, requires 2 lanes	"	5	12	4	21
		23.8	57.1	19.1	100
e. Through numbered route turns left	"	8	9	4	21
		38,1	42.9	19.0	100
f. Only alternative to a loop ramp	"	4	8	7	19
		21.0	42.1	36.9	100
g. Left turn from right lane cost $100,000 more	"	1	3	13	17
		5.8	17.7	76.5	100
h. Left turn from right lane cost $250,000 more	"	2	5	10	17
		11.8	29.4	58.8	100
i. Left turn from right lane cost $500,000 more	"	5	5	7	17
		29.4	29.4	41.2	100
j. Other (specify)	"	3	0	0	3
		100	0	0	100

j. Other:

Left turn from right lane in excess of $1,000,000

Only when a major fork design is used.

c. above, as the mainline road (thru route continues ahead)

Comments:

Almost always for c. above assuming that 50 percent would require a major fork design, therefore permitted. Almost always for e. above if two lanes (or more) are provided.

If this is a major fork, one must go to the left. But, if main route goes left, make this roadway generally straight thru at the fork with the other one breaking to the left.

Sometimes for d. above if the right turn is also two lanes.

New York's policy statement on left-hand exits and entrances is to not do it except for major forks, and then place the higher commercial volume on the right.

g through j are variable based on conditions.

Any major fork has one on the left.

Sometimes for c. above assuming the thru route continues ahead. Almost never for e. above assuming a ramp connection.

For g to j -- The consideration is mostly cost independent.

Cost should be an insignificant factor when safety is compromised; that is, a left turn ramp should be the <u>only</u> possible alternative.

20. Figures 3 thru 8 in the "Figures Package" alternate exit ramp arrangements on one approach of a major interchange. Place an x under the use category which characterizes the frequency with which your organization uses each arrangement for new, i.e., recent and current, designs.

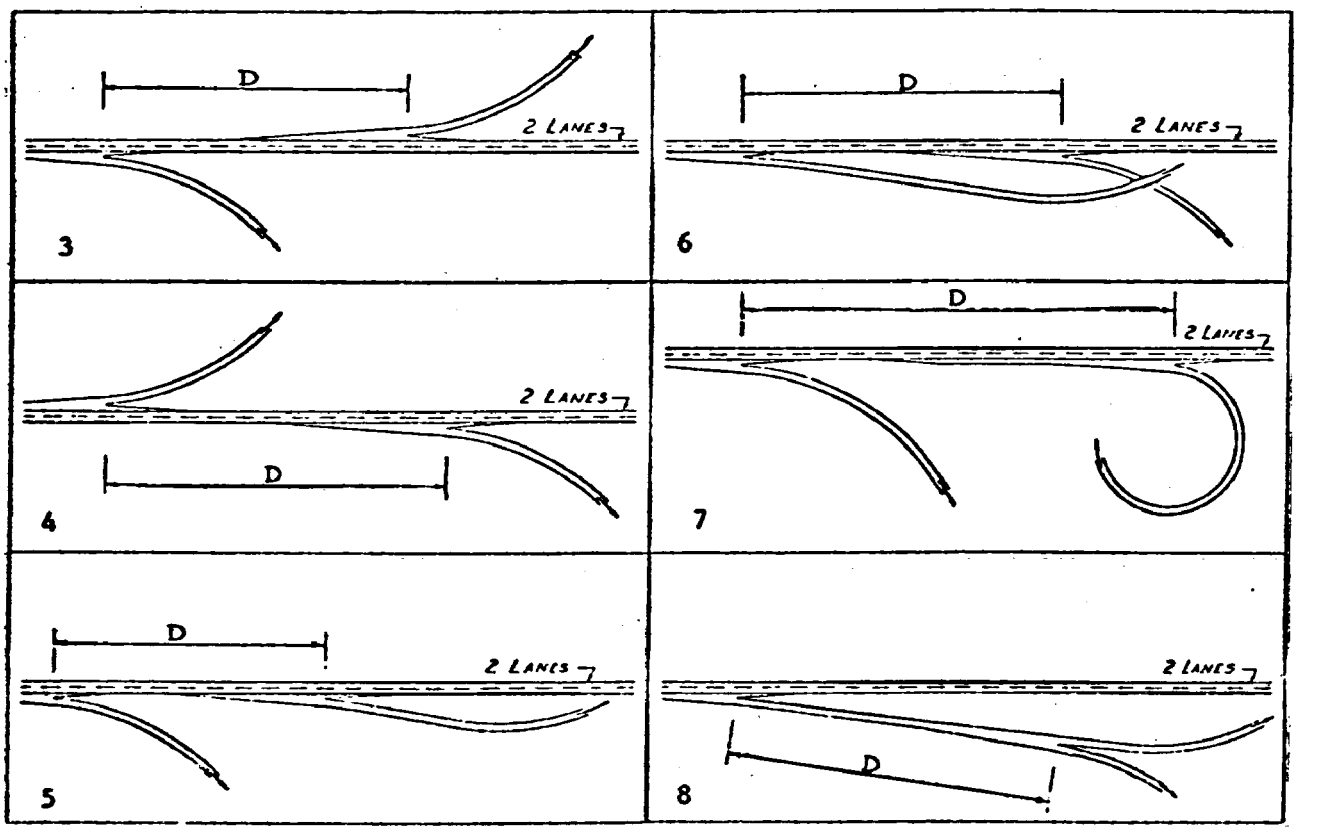


Figure No.		USE CATEGORY					
		Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total Number Responding
3	No. Responding	0	0	0	3	14	17
	%	0	0	0	17.6	82.4	100
4	No. Responding	0	0	0	2	15	17
	%	0	0	0	11.8	88.2	100
5	No. Responding	0	0	2	10	5	17
	%	0	0	11.8	58.8	29.4	100
6	No. Responding	0	1	1	11	4	17
	%	0	5.9	5.9	64.7	23.5	100
7	No. Responding	0	1	4	12	0	17
	%	0	5.9	23.5	70.6	0	100
8	No. Responding	1	5	9	2	0	17
	%	5.9	29.3	53.0	11.8	0	100

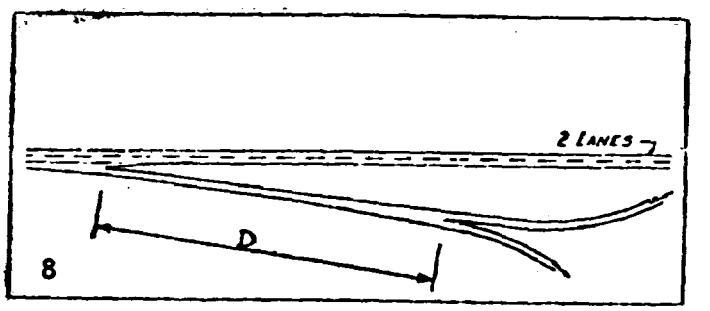
Comments:

Left-hand exits are generally avoided.

Try to avoid Figures 3 and 4.

Preference to Figure 8 is initially explored in interchange layouts.

21. The single exit with a fork shown in Figure 8 has been advocated over configurations having two separate exits. Presented below are several statements regarding the single exit configuration. Please indicate the extent to which you agree or disagree with each statement by placing a circle around the appropriate word. Following your responses to the statements, please list or describe other advantages or disadvantages of the single exit.



		a. Strongly Agree	b. Agree	c. Disagree	d. Strongly Disagree	Total Number Responding
Statement 1.						
The single exit is always desirable for traffic flow advantages.	No. Responding	5	9	7	0	21
	%	23.8	42.9	33.3	0	100
Statement 2.						
The single exit is always desirable for safety	No. Responding	6	9	6	0	21
	%	28.6	42.8	28.6	0	100
Statement 3.						
The single exit can be used if the exit is one lane, but not if it is two lanes	No. Responding	0	1	13	6	20
	%	0	5.0	65.0	30.0	100

Other Advantages:

If volumes warrant only a one-lane takeoff, signing is simplified. If there is a two-lane takeoff required, then a major fork or bifurcation can be used.

My disagreement with statements 1 and 2 above is because of the word "always." If the single exit can be designed to accommodate traffic volumes, it is desirable.

When normal routing is involved, it can simplify overall signing for the freeway. If a uniform exit pattern is desired, it is compatible.

Generally prefer #8 due to signing advantages, etc. #5, 6, 7 are acceptable designs and may have advantages in the specific case.

The single exit greatly simplifies signing.

Better signing -- more time for driver decision.

As a general comment I do not believe that single lanes do or do not help traffic flow. Here again, it would depend on the traffic volumes to determine if a single exit or two separate exits should be used. We (Texas) have employed both types of design and are pleased with both. If a two-lane exit is employed, then one lane must be dropped. We have found that signing two separate ramps is less confusing to the driver than one exit with a split.

The design reduces driver confusion.

Driver expectancy and signing. Merge-diverge turbulence is off the main lanes.

Easier to properly sign the single exit.

Easier for user, only one choice at the exit. Second decision would be at a slower speed.

Simplified signing design, driver makes one decisive maneuver from the mainline, driver hesitation at ramp terminals is lessened.

Single exits reduce decisions to be made on the mainline to one question: to exit or not to exit. Other decisions are made on the lower volume, lower speed ramp or C-D road.

Other Disadvantages:

Signing of the single exit is critical.

The volume of traffic using the ramp and the maximum distance that can be obtained between the two ramp exits will bear on whether a single exit will function better than two exits.

Where complex routing is involved, it can overload signs. When a two-land exit is used, it creates a weaving section.

The problems to be weighed in the balance is that a multilane or major fork design is often necessary. This is more complicated than two single exits.

Capacity restraint.

Single exit increases construction and right-of-way costs.

Second fork problems cause backups, large spatial requirements.

Design, right-of-way, and topography may not be compatible with resulting high increase in cost.

With high volumes, the single exit may be a problem due to concentration of two ramp flows at one exit.

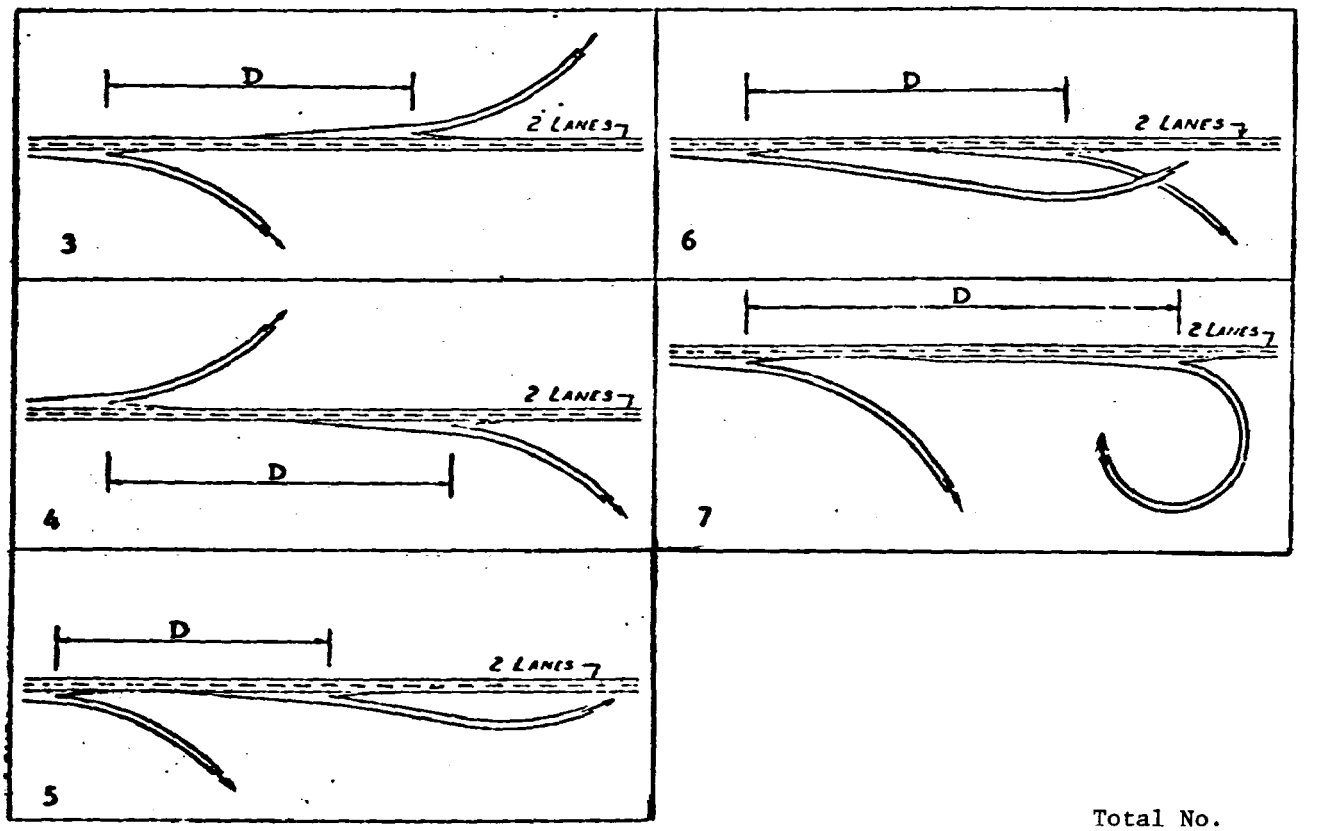
Cost, spatial requirements.

Single exit designs are usually more expensive due to the need for additional and/or wider structures.

22. If the turning volume in Figure 8 requires a two-lane exit, would one of the other arrangements be more desirable? If yes, circle the more desirable arrangements as depicted in Figures 3-7. If you circle more than one of the alternative arrangements, indicate the most desirable by putting an "a" below it, the next most desirable a "b", etc.

a. Yes b. No

Figures 3 4 5 6 7



	Yes	No	Total No. Responding
Number Responding	8	13	21
Percent	38.1	61.9	100.0

Figure	3			4			5			6			7		
Desirability	a	b	c	a	b	c	a	b	c	a	b	c	a	b	c
No. Responding	1	0	0	0	0	0	4	0	0	1	2	1	2	2	1

Comments for "no" responses:

We have found out that signing two separate ramps is less confusing to the driver than one exit with a split. The answer depends a great deal on whether you have adequate D distance to provide the Figure 5 arrangement. If you do not, then use the arrangement in Figure 8.

Figures 6 and 7 are sometimes more desirable.

23. For each of the exit arrangements shown in Figures 3-8, indicate the minimum and desirable distance D between exit noses. Your answer is to reflect your opinion and not necessarily the current practice of your organization. For the purpose of answering you should assume a flat grade and the existence of adequate signing.

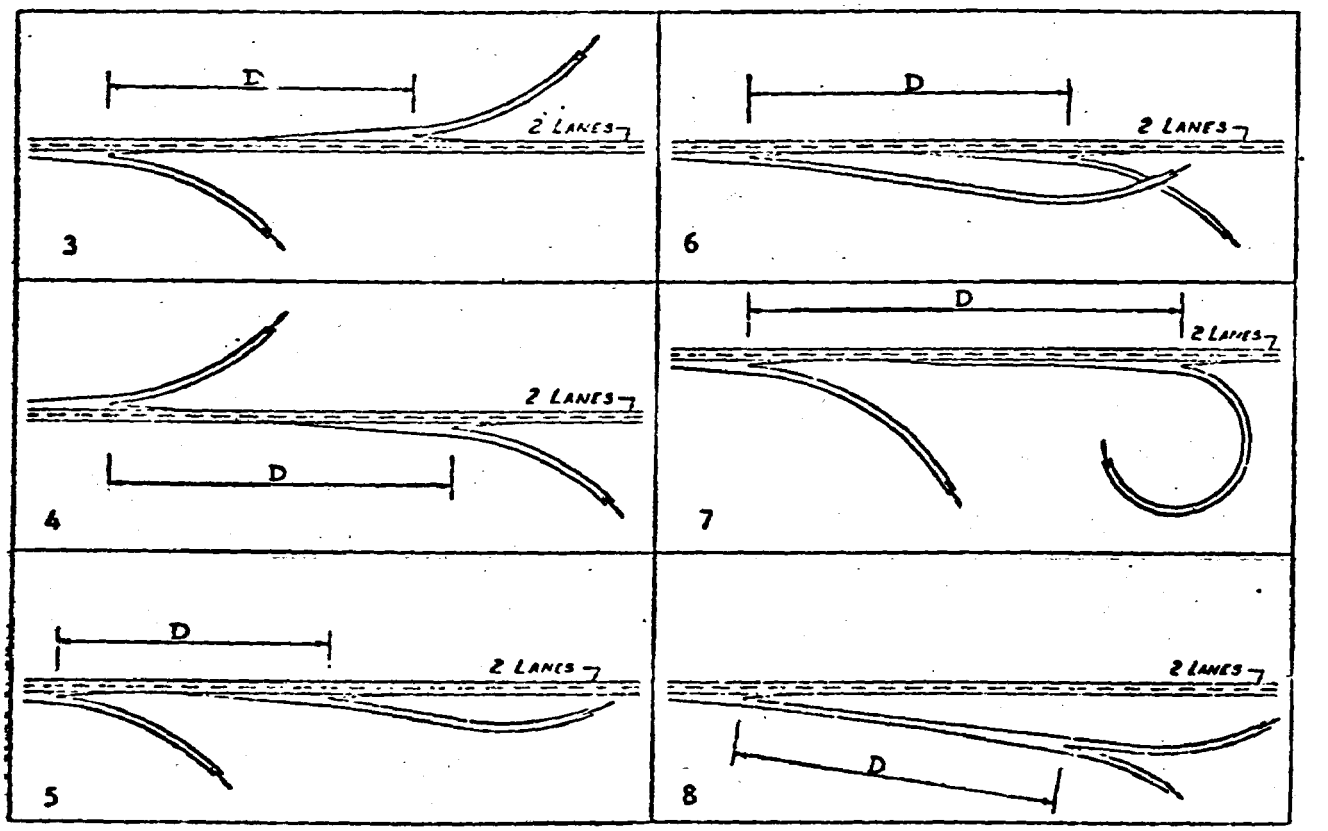


FIGURE 3

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
350	1	6.7	6.7
500	2	13.3	20.0
600	5	33.3	53.3
800	2	13.3	66.6
900	1	6.7	73.3
1000	1	6.7	80.0
1200	1	6.7	86.7
1500	2	13.3	100.0
Total	15	100.0	

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
600	2	12.5	12.5
1000	6	37.5	50.0
1200	3	18.8	68.8
1400	1	6.2	75.0
2000	4	25.0	100.0
Total	16	100.0	

FIGURE 4

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
350	1	6.7	6.7
500	2	13.3	20.0
600	6	40.0	60.0
800	2	13.3	73.3
1000	1	6.7	80.0
1200	2	13.3	93.3
1500	1	6.7	100.0
Total	15	100.0	

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
600	2	12.5	12.5
900	1	6.3	18.8
1000	6	374.	56.2
1200	2	12.5	68.7
1400	1	6.3	75.0
2000	4	25.0	100.00
Total	16	100.0	

FIGURE 5

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
350	1	4.8	4.8
400	1	4.8	9.6
500	1	4.8	14.4
600	4	19.0	33.4
670	1	4.8	38.2
800	5	23.7	61.9
1000	7	33.3	95.2
1400	1	4.8	100.0
Total	21	100.0	

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
600	1	4.8	4.8
700	1	4.8	9.6
900	3	14.3	23.9
1000	5	23.8	47.7
1200	2	9.5	57.2
1500	4	19.0	76.2
1600	2	9.5	85.7
2000	2	9.5	95.2
2300	1	4.8	100.0
Total	21	100.0	

FIGURE 6

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
350	1	4.8	4.8
400	1	4.8	9.6
500	1	4.8	14.4
600	4	19.0	33.4
670	1	4.8	38.2
800	5	23.7	61.9
1000	7	33.3	95.2
1400	1	4.8	100.0
Total	21	100.0	

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
600	1	4.8	4.8
700	1	4.8	9.6
900	3	14.3	23.9
1000	5	23.8	47.7
1200	2	9.5	57.2
1500	4	19.0	76.2
1600	2	9.5	85.7
2000	2	9.5	95.2
2300	1	4.8	100.0
Total	21	100.0	

FIGURE 7

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
400	1	4.8	4.8
500	1	4.8	9.6
600	3	14.3	23.9
670	1	4.8	28.7
800	5	23.7	52.4
900	2	9.5	61.9
1000	4	19.0	80.9
1200	2	9.5	90.4
1500	1	4.8	95.2
2300	1	4.8	100.0
Total	21	100.0	

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
700	1	5.0	5.0
900	3	15.0	20.0
1000	5	25.0	45.0
1200	2	10.0	55.0
1500	4	20.0	75.0
1600	2	10.0	85.0
2000	2	10.0	95.0
2500	1	5.0	100.0
Total	20	100.0	

FIGURE 8

Minimum Distance D (ft.)	Number Responding	Percent	Cumulative %
400	1	4.9	4.9
500	2	9.5	14.4
600	7	33.3	47.7
800	7	33.3	81.0
900	2	9.5	90.5
1000	2	9.5	100.0
Total	21	100.0	

Desirable Distance D (ft.)	Number Responding	Percent	Cumulative %
700	1	4.8	4.8
800	3	14.3	19.1
900	2	9.5	28.6
1000	5	23.7	52.3
1200	7	33.3	85.6
1400	1	4.8	90.4
1500	1	4.8	95.2
1600	1	4.8	100.0
Total	21	100.0	

Comments:

The distances are general. Physical conditions and economics could play an important part in determining length. They would not vary too much from above in most instances.

Figures 3 and 4 are not recommended.

It is assumed that these values are desirable minimums since generally the greater the distance the better the overall situation (for all desirable distances).

Existence of adequate signing should be the main factor in determining the minimum distance. No answer for Figures 3 and 4; left-hand ramps should not be built on future projects.

No answer for Figures 3 and 4; they should not be considered regardless of "D" distance.

Distances are not given for Figures 3 and 4 because, in my opinion, they should never be used.

Fig. 3-7, min. D = 800-1000 ft. (800 ft. used in tabulations)
Fig. 3-7, desirable D = 1000-1500 ft. (1000 ft. used in tabulations)

Adequate signing is a key feature.

Left-hand ramp exit situations should recognize lane changes and indecisiveness of drivers.

Fig. 5-7, min. D = 670 ft., length of standard exit ramp taper. Since I believe that left-hand exit ramps should never be used, I have not shown distances for Figures 3 and 4. Distance for Figure 8 (600 and 900) is for signing.

Distances are given assuming a mainline design speed of 70 mph and based on the assumption that no more than one decision be made within 10 sec.

Fig. 7, desirable D; should not exist.

24. Does your opinion, as expressed in answering question #23, differ from the standard practices used in your organization?

	Number Responding	Percent
Yes	6	37.5
No	10	62.5
Total	16	100.0

Comments for "yes" responses:

Left-hand ramps should not be built on future projects. We have not been 100% successful in avoiding left-hand ramps, but this is our objective.

We are generally not able to get the distances shown in #23.

Comments for "no" responses:

The distances in #23 are general. Physical conditions and economics could play an important part in determining length. They would not vary too much from those given in #23 in most instances.

We have recommended only one value and recognize it is a desirable minimum distance.

General Comments:

Since each situation is evaluated separately, we have no standard practice for this situation.

25. Please state your opinion as to the conditions under which entrances from the left should be permitted.

Note: Answers with "Major fork," probably should be branch connections.

Conditons:

a. High relative traffic volumes (when nearly equal or greater left turning volume).

b. High speed merge - auxiliary lane, etc.

c. Improves downstream weaving situation.

d. Economical (cost is prohibitive).

e. No merging areas, all entrance lane(s) is/are added to mainline.

f. At least one entrance lane is added.

g. Horizontal and vertical geometry must be conducive to proper merge.

h. Parallel auxiliary lane is at least 2000 feet.

i. When the traffic volume warrants the addition of another through lane.

j. If there is no other way to construct a ramp and sufficient warrants for the movement (over 250 vph).

k. At all major forks. (Related to 'a'.)

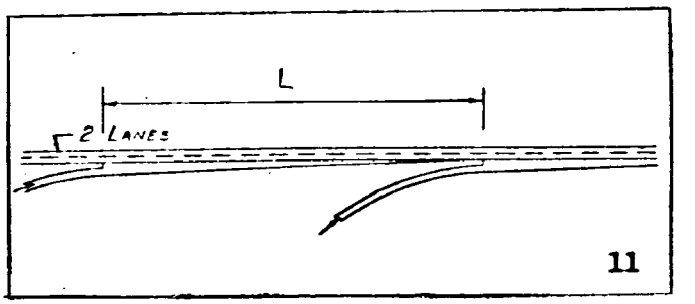
l. No downstream exits on the right near enough to cause weaving.

m. Where necessary number of entrance lanes cannot be provided on right. (Related to 'j')

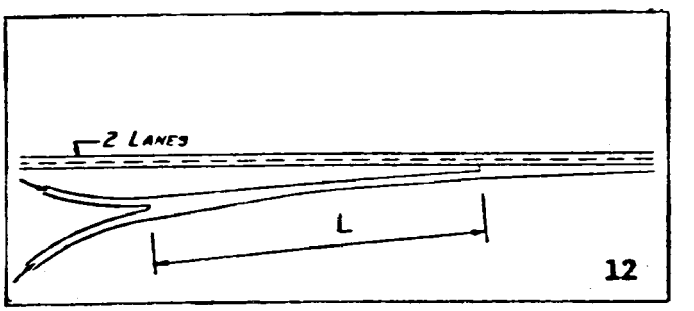
n. Not compounded by right hand entrance in same proximity.

o. Under no conditions because of safety hazard, e.g., "blind spots" in view of the through traffic.

26. Shown in Figures 11 and 12 are two arrangements for handling the situation where two ramps enter the main roadway from the right. Please put an'"x" beside the arrangement you feel is the most desirable from the standpoint of safety and operations.



Two successive entrances to the through roadway



Merging of two ramps to provide a single entrance to the main line.

Figure	# Responding	Percent
11	1	5.2
12	18	94.8
Total	19	

Comments:

Cannot answer

Answer depends on traffic volumes, weaves, etc.

Both are good arrangements and Fig. 11 would be OK if aux. was added. The addition of aux. lane would of course depend on the length of L. Fig. 12 would be safer from a "min. point of access" criteria, however, we have used both and the criteria that determines which to use was traffic volumes and available space.

Either can serve.

Depending on the entering volume.

27. Considering only current (or recently completed) design efforts, please describe briefly the primary conditions and/or situations which lead to the use of the cloverleaf configuration for major interchanges.

Conditions:

a. Cloverleaf is cheapest, therefore, it is the obvious choice in rural, low volume situations. We upgrade when needs call for it. (Calif.)

b. When cost too high for direct or semi-direct ramps. ('a')

c. None, cloverleaf not used in Texas for new major interchanges. (Georgia, New Jersey

d. Moderate turning movements requiring only single lane ramps. (Ohio 800 dhv or less)

e. Right-of-way (No space restrictions)

f. Topography

g. Light weaving volumes.

h. When neither route turns.

i. Stage construction is easily accomplished with cloverleaf.

j. Highly directional flow so adjacent loop ramps never carry maximum volumes simultaneously.

k. Use of collector-distributor roads to eliminate weaves.

l. Right-angle of intersection.

28. Figures 9 thru 12 indicate four alternate entrance ramp arrangements on one leg of a major interchange. Place an x under the use category which characterizes the frequency with which your organization uses each arrangement in current (or recent past) designs.

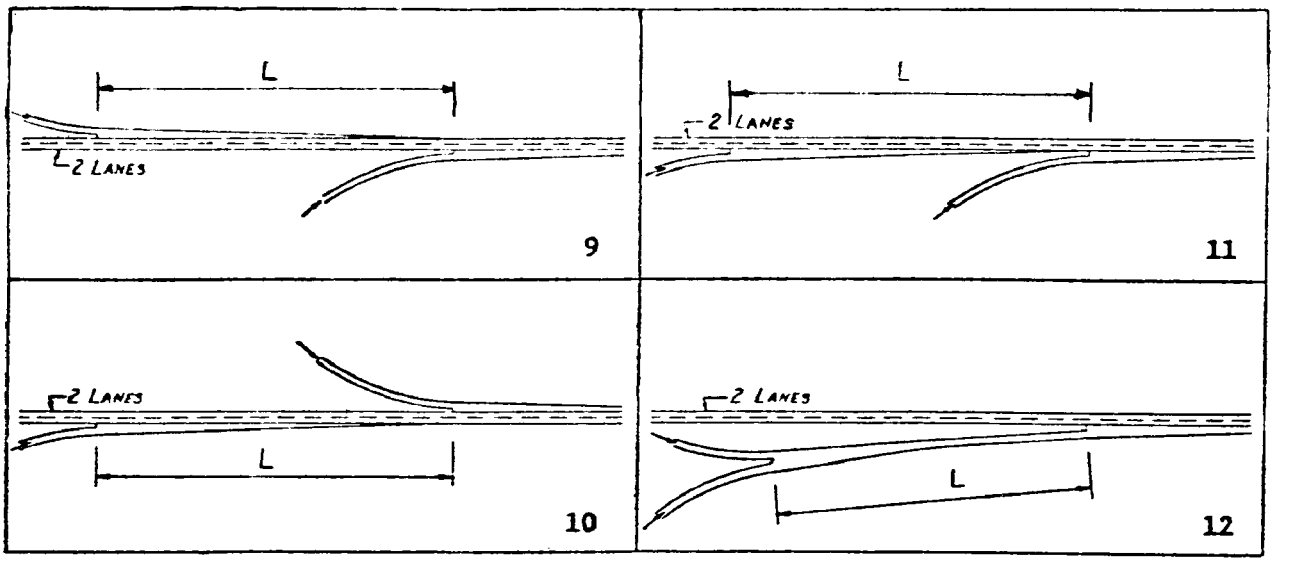


Figure Number		USE CATEGORY Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total Number Responding
9	No. Responding	0	0	0	0	17	17
	%	0	0	0	0	100	100
10	No. Responding	0	0	0	0	17	17
	%	0	0	0	0	100	100
11	No. Responding	0	1	8	6	2	17
	%	0	5.9	47.0	35.3	11.8	100
12	No. Responding	1	6	8	2	0	17
	%	5.9	35.3	47.0	11.8	0	100

Comments:

Try to avoid Figures 9 and 10.

29. If an extra lane is added to the through roadway as a continuation of the entrance ramp from the left would your answer to question #28 change?

	Yes	No	Total Number Responding
Number Responding	13	4	17
Percent	76.5	23.5	100

Comments for "yes" responses:

With a continuous free lane the left-hand entrance becomes more acceptable. Multiple right and left entrances in the same vicinity would adversely affect operations. Ramp entrances and exits which create compound weaving situations would have to be avoided.

When no merge is involved the left entrance would be acceptable.

It is still more desirable to have the ramp on the right.

The continuous extra lane for the left entrance ramp would allow the left entrance to be more acceptable and used more often by reducing or eliminating traffic in the left lane from merging right in a short distance.

Usage of Figures 9 and 10 would probably increase since we would not be as hesitant to bring a ramp in on the left if no merge is required, and the overall cost of the interchange would be notably reduced.

No objection to a left on-ramp if a lane is added.

It would eliminate part of the merging problem.

Continuation of the left entrance ramp as a through lane prevents the hazardous merging from left to right into the high-speed lane.

An extra lane is required if the ramp is on the left.

Yes; however, entrance ramps from the left should not be a matter of design practice.

Comments for "no" responses:

No left ramps should be provided.

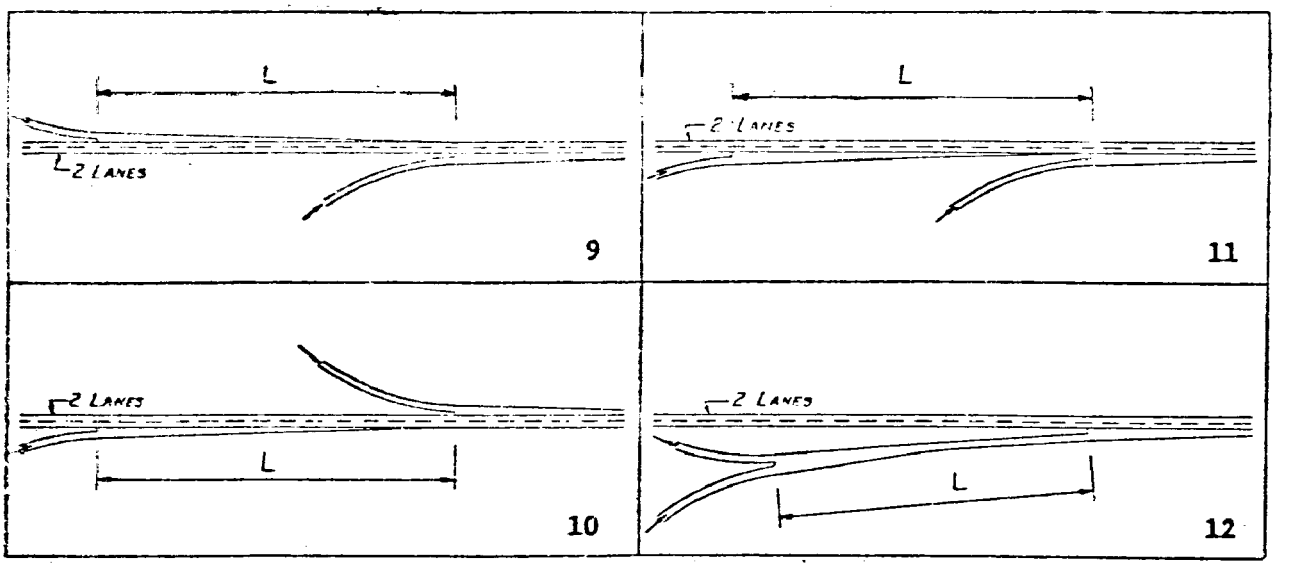
Left entrances should never be provided unless the two legs of the connection are of equal importance and have an equal number of lanes.

Left-hand entrances are always hazardous, especially for trucks, since the following exit will generally be on the right, and thus a merge to the side of reduced visibility must eventually be made.

30. If the turning volume in Figure 12 requires a two-lane entrance, would one of the other arrangements be more desirable? (Circle one) If yes, circle the more desirable arrangements as depicted in Figures 9, 10, and 11.

a. Yes b. no

Figures 9 10 11



	Yes	No	Total Number Responding
Number Responding	9	12	21
Percent	42.9	57.1	100

Figure No.

	9	10	11
Number Responding	0	0	9
Percent	0	0	100

Comments for "yes" responses:

Each situation requires individual analysis.

The answer is "no" if an added lane or auxiliary lane is available. The answer is "yes" if no additional lane is provided.

Figure 11 if adequate distance for L is available. If not, use Figure 12 and add two lanes.

Figure 11, if L were at least 2,500 ft.

Comments for "no" responses:

Either 11 or 12 can be designed suitably.

An extra lane would be picked up.

31. For each of the four entrance arrangements shown in Figure 9-12 indicate your opinion as to the minimum and desirable distance L between entrance noses.

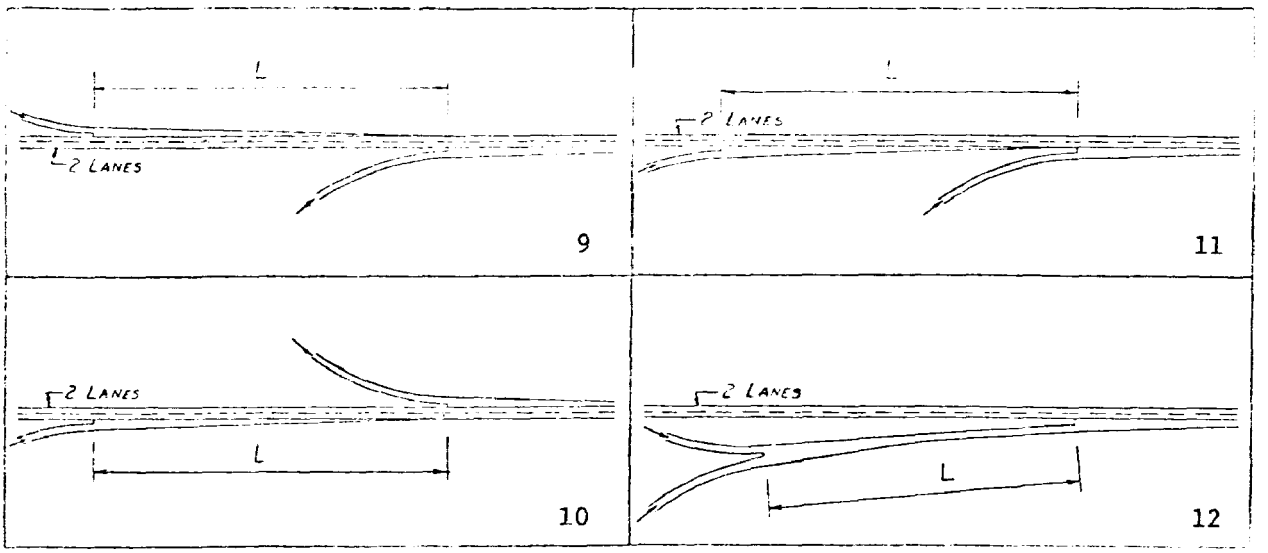


Fig. 9 Minimum Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
500	1	5.6	5.6
575	1	5.6	11.2
900	2	11.1	22.3
1000	8	44.5	66.8
1200	2	11.1	77.9
1400	1	5.6	83.5
1500	3	16.7	100.0
Total	18		

Fig. 9 Desirable Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
900	1	5.9	5.9
1200	2	11.8	17.7
1400	1	5.9	23.6
1500	2	11.8	35.4
1600	1	5.9	41.3
1800	1	5.9	47.2
2000	8	47.1	94.3
2400	1	5.9	100.0
Total	17		

Fig. 10 Minimum Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
500	1	5.6	5.6
575	1	5.6	11.2
900	2	11.1	22.3
1000	9	50.0	72.3
1200	2	11.1	83.4
1500	3	16.7	100.0
Total	18		

Fig. 10 Desirable Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
900	1	5.9	5.9
1200	2	11.8	17.7
1400	1	5.9	23.6
1500	2	11.8	35.4
1600	1	5.9	41.3
1800	1	5.9	47.2
2000	9	53.0	100.0
Total	17		

Fig. 11 Minimum Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
500	1	4.8	4.8
575	1	4.8	9.6
900	3	14.3	23.9
1000	5	23.8	47.7
1150	1	4.8	52.5
1200	3	14.3	66.8
1400	3	14.3	81.1
1500	2	9.5	90.6
1600	1	4.8	95.4
2300	1	4.8	100.0
Total	21		

Fig. 11 Desirable Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
900	1	5.3	5.3
1000	2	10.5	15.8
1150	1	5.3	21.1
1200	2	10.5	31.6
1500	3	15.8	47.4
1800	1	5.3	52.7
2000	7	36.9	89.6
2400	1	5.3	94.9
2500	1	5.3	100.0
Total	19		

Fig. 12 Minimum Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
400	3	14.3	14.3
500	3	14.3	28.6
600	3	14.3	42.9
750	1	4.8	47.7
800	5	23.8	71.5
900	2	9.5	81.0
1000	1	4.8	85.8
1200	1	4.8	90.6
1500	1	4.8	95.4
1600	1	4.8	100.0
Total	21		

Fig. 12 Desirable Length (L)

L (in feet)	No. Responding	Percent	Cumulative %
600	1	5.0	5.0
700	1	5.0	10.0
800	2	10.0	20.0
900	3	15.0	35.0
1000	1	5.0	40.0
1200	7	35.0	75.0
1400	1	5.0	80.0
1500	1	5.0	85.0
2000	3	15.0	100.0
Total	20		

Comments:

Fig. 9-12 (min. length) are ±.

Fig. 9-12 (desirable length) - cannot answer - depends on other factors - merging distance #12 depends upon approach speeds, etc.

Fig. 9 and 10 (Min. & desirable lengths) - not applicable

Fig. 9 and 10 -- undesirable condition

Fig. 11 (min. & desirable) -- length of standard entrance taper (New York).

1000 ft. min. length for all figures.

2000 ft. desirable length for all figures

Fig. 9 and 10 --

Fig. 11, desirable length -- "should not exist"

32. Does your opinion, as expressed in answering question #31, differ from the values recommended in your state manual?

Response	No. Responding	Percent
Yes	1	7.2
No	2	14.3
Not Covered in State Manual	11	78.6
Total	14	

Comments for "no" responses:

Only Fig. 11 and 12 covered in state manual (Ohio).

33. In designing entrance ramps you may prefer the taper type acceleration lane for one situation or condition and the parallel type for another. Please state below the situations or conditions under which you prefer each type of geometry.

The tapered type is preferred unless other considerations indicate the desirability of a more sophisticated treatment. A parallel type is recommended where merging distance is believed necessary, especially where anticipated through volumes show minimum merging opportunity or where other adverse conditions indicate the desirability of this treatment.

Tapered type is preferred.

Tapered type is always preferred except where weaving is involved.

Prefer the tapered type unless ramp is 2 lanes on the right with an added lane, or unless the ramp is on the left.

Tapered type entrances are desirable under most conditions where sight distance, grades, and alignment are acceptable. Parallel type entrances may often be used where longer acceleration distances are desired, and where continuous lanes are to be used.

Tapered type preferred for all conditions but especially on horizontal tangents.

We generally provide a standard tapered acceleration lane (L = 1000 ft. ±). Additional parallel lanes are added as traffic requires.

The tapered type is most always preferred simply for the sake of uniformity and as recommended by AASHO since the Special Freeway Committee Report of 1960 (Red Booklet). Parallel type used when situations warrant a climbing lane as a part of an entrance, but a 50:1 taper would still be the optimum desired at the terminal.

The tapered type is usually used except on an upgrade or a two-lane off-ramp.

We employ the taper type of ramp in most cases because our (Texas) experience has shown that most drivers drive a parallel acceleration ramp as if it was a taper ramp.

A tapered type is used for most cases. When an upgrade exists beyond the entrance, a parallel acceleration lane may be needed for commercial vehicles to attain a reasonable operating speed before entering the mainline.

We do not use a tapered type in present design. A parallel type is used all the time.

Either alternative can be designed to work well.

Tapered type used in all cases except where grade sight distance and capacity would require a length longer than practical for a taper.

Tapered type is favored for all conditions since the merge is gradual.

The taper-type is adequate if there is plenty of length (1000 ft. +). Parallel may be used if lengths is restricted and/or if there is very heavy mainline traffic.

Tapered type is generally preferred in urban high volume situations principally to encourage the driver to be aggressive. Parallel type is generally preferred on high speed-medium volume facilities where interchanges are spaced at or greater than two-mile intervals, such as toll roads or rural remote interstate routes. It is also preferred for two-lane entrance situations and trucks entering on an upgrade.

Tapered type used in all cases except:

1. At an entrance carrying a large truck volume in combination with a steep upgrade. The lane should then be designed as a climbing lane.
2. For lane balance, when it is desired to maintain the basic number of through freeway lanes at a two-lane entrance.
3. On highways curbed at the edge of the travel lane.

I prefer the tapered type where the ramp merges at a flat angle and sight distance is good. Parallel used when the reverse is true.

In rural situations where the mainline traffic volumes are low, a taper is used. In <u>most</u> situations a parallel type is used because the parallel type speed-change lane will allow both the mainline driver and the entering driver to adjust to the final entering maneuver at the prevailing mainline operating speed.

34. In designing exit ramps you may prefer the taper type deceleration lane for one situation or condition and the parallel type for another. Please state below the situations or conditions under which you prefer each type of geometry.

The tapered type is normally used. A parallel-type is used under very high exiting volumes, or where geometry indicates desirability in order to overcome deficiencies in target value or the operational characteristics of a normal taper.

The tapered type is preferred. Where there is the possibility of a back up on the mainline a parallel type is used.

Tapered type used for a major fork or a major terminal where little or no slowing is required. The parallel type is used exclusively where deceleration is required. The beginning is abrupt and well-defined, thus alerting the motorist that the nose is ahead.

Tapered type is preferred in most situations. A parallel lane is preferred if the off-ramp is 2-lane or if the ramp terminal is such that deceleration or storage back onto the freeway is needed. Also used if stopping sight distance is restricted.

Tapered type exits are preferred where high-speed exits are used. Parallel type exits are preferred where slower ramp speeds are necessary.

Tapered type used on all tangents, parallel type on all curves.

A standard taper is used for freeway-to-freeway exits. Approach auxiliary lanes are provided as traffic requires.

Tapered type is desired at 3°-5° or 15:1 angles of exit to fit the path that most drivers take and to show a definite point of take-off from the mainline. Longer tapers cause problems, for through drivers make unintentional exits, especially during poor visibility conditions at night. The parallel type is adequate for use where the ramp must depart from the mainline on a horizontal curve. (This condition should be avoided if at all possible.) A parallel lane with an abrupt take-off is good to distinguish the ramp from the mainline roadway and avoid the problem described above.

The tapered type is usually used except on a downgrade or a 2-lane off-ramp.

We (Texas) employ the tapered type in most cases because our experience has shown that most drivers drive a parallel ramp as if it was a taper ramp.

The tapered type is used for most cases. A parallel type is used for a mainline curve to the left to reduce the appearance of the mainline going up the ramp. It is also used where an exit is unavoidably hidden beyond a crest vertical curve with the parallel lane starting prior to the crest.

We find that the tapered section is not used. A parallel type with an 80 ft. taper is employed all the time.

Either can be designed to work well.

The taper is favored in all instances except where the difference in ramp and freeway speeds are so great that the taper would not be practical, and where sight distance is a problem.

The parallel type is favored for all situations. It can be used as a taper or a lane for reducing speed and also provides a storage area if required during excessive traffic problems.

A tapered type is adequate if the ramp permits an exit at 60-65 mph. Too many tapers are used where braking on the taper (thus in mainline) is necessary due to a short finger or a tight loop ramp ahead. In this situation a parallel type is better.

A tapered type is preferred under medium traffic situations and high ramp design speeds. Parallel preferred under dense mainline traffic conditions -- heavy ramp volumes, either urban or rural situation and relatively low ramp design speeds. It is also preferred for a 2-lane exit situation.

Tapered type is preferred for all cases except:

1. When a right-hane exit is unavoidably located on a mainline curve to the left, and it is feared that the use of a taper might result in inadvertent use of the ramp by through traffic. Note that this configuration is undesirable and should be avoided.
2. When an exit is unavoidably located immediately beyond a crest vertical curve or in any other area of restricted visibility.
3. For lane balance, when it is desired to maintain the basic number of through freeway lanes at a 2-lane exit.

I prefer the parallel type in all cases, if it is long enough and signed adequately to encourage drivers to use it for deceleration.

A tapered type is favored in most situations since proper planning should allow the designer to build adequate storage space into this terminal facility to prevent slowdown on the mainline facility. It gets the exiting driver away from the mainline the fastest and does not present a confusing situation to the through driver. When it is not feasible to build adequate storage space into tapered ramps, a parallel type is favored. It does get the driver out of the through stream, and it does present a confusing situation to the through driver as it will appear the facility has added a lane.

35. Figures 13 through 22 show alternative methods for lane drops when turning volumes justify a reduction in the number of through traffic lanes. Place an x under the use category which characterizes the frequency with which you organization currently use each arrangement. (Note: neglect the shape of the deceleration lane and nose geometry.)

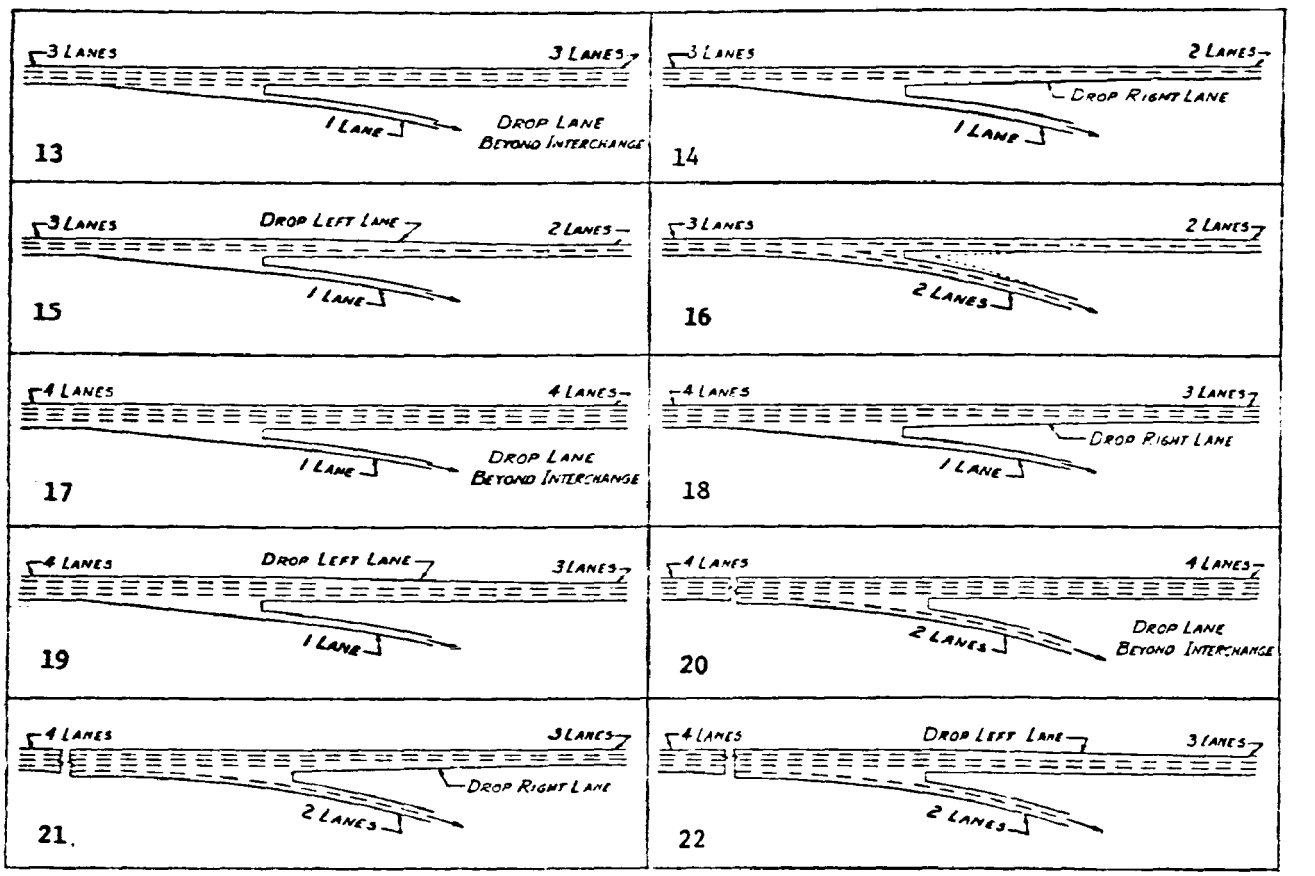


Figure Number		USE CATEGORY					
		Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total
13	Number Responding	2	5	1	4	5	17
	%	11.8	29.4	5.9	23.5	29.4	100
14	Number Responding	1	4	2	8	2	17
	%	5.9	23.5	11.8	47.1	11.8	100
15	Number Responding	0	0	0	3	14	17
	%	0	0	0	17.7	82.4	100
16	Number Responding	1	3	3	9	1	17
	%	5.9	17.7	17.7	53.0	5.9	100
17	Number Responding	1	4	3	4	5	17
	%	5.9	23.5	17.7	23.5	29.4	100
18	Number Responding	1	4	3	7	2	17
	%	5.9	23.5	17.7	41.2	11.8	100
19	Number Responding	0	0	0	3	14	17
	%	0	0	0	17.7	82.4	100
20	Number Responding	0	2	3	3	9	17
	%	0	11.8	17.7	17.7	53.0	100
21	Number Responding	2	7	4	2	2	17
	%	11.8	41.2	23.5	11.8	11.8	100
22	Number Responding	0	0	0	2	15	17
	%	0	0	0	11.8	88.2	100

Comments:

The case that New York State advocated is not indicated. That is, drop the left lane well beyond the interchange where sight distance is unquestionably good. We would use an arrangement similar to Fig. 19 with the lane drop 1/2 mile beyond the last ramp.

Fig. 20 - won't work as a two-lane exit.

Fig. 13 & 17 - very desirable -- although hard to obtain if interchanges are closely spaced. Fig. 16 - very desirable.

Fig. 15 & 18 - Personally, I am not opposed to this method if full width lane is not carried beyond nose.

36. In your opinion, when a lane is dropped beyond the interchange, which lane should be dropped? (Circle one)

	Number Responding	Percent
Left Lane	3	15.0
Right Lane	16	80.0
Either	1	5.0
	20	100

Comments:

Left Lane:

Begin the lane drop taper 1/2 mile beyond the last ramp of an interchange and use "design speed" x "lane width" for taper length.

Prefer not to have the slower moving vehicles generally in the right lane to move over into the faster traffic.

Reasons for the choice:

1) Lane can be extended at a later time without disrupting interchange ramp terminals and bridge pier spacing.
2) Median transition (widening) occurs almost automatically without need for dog legs in alignment (Medians usually are widened at same location of lane drop).
3) Since drivers usually stay to the right, fewer have to change lanes for a left side lane drop.

Either right or left side designs can work OK if designed with good sight distance, proper recovery area beyond the taper, and far enough removed from other points of conflict such as ramp terminals.

Either Lane:

Depends on geometrics at lane drop and traffic distribution.

Right Lane:

Safer to merge right lane

On two-lane (one-way), we have wider paved shoulder on right. Also, I would rather deal with volume than speed. On three- or four-lane (one-way), the shoulders are the same width and the volume is more evenly distributed but speed is higher on left, therefore, prefer right-hand drop.

Keeps speed changes and weaving on the right side where drivers expect it.

This is in conflict with official department policy which states that left lane should be dropped since it carries lower volume. I maintain that it is preferable to drop the lane carrying lower speed traffic.

Left lane usually higher speed and driver has generally poorer visibility of the merging operation.

The left lane is commonly used more for high speed through traffic passing the interchange and should not have a lane drop. The right lane is used by the interchanging traffic which is more desirable for lane drop.

High speed traffic is in the left lane and should not be disrupted by a lane drop.

Through traffic in high-speed left lanes should not be operationally interrupted.

Driver expectancy.

Better operation and greater safety for the higher speed traffic in the remaining lanes.

The burden of caution should be on the slower driver who generally occupies the right lane; and also, he has the least possibility of vehicle "blind spots" as he is closest to the through traffic.

37. Where the turning traffic volume requires a two-lane turning roadway, certain geometrics may be less desirable than others if operational problems are to be minimized. Indicate the frequency with which your organization currently uses each of the features listed below on 2-lane turning roadways in a major interchange.

		USE CATEGORY					
		Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total Number Responding
a) Two-lane exit roadway	No. Responding	2	3	4	7	1	17
	%	11.8	17.7	23.5	41.1	5.9	100
b) One-lane exit followed by widening to 2-lanes	No. Responding	0	2	2	6	7	17
	%	0	11.8	11.8	35.3	41.7	100
c) Two-lane entrance roadway	No. Responding	1	2	4	6	4	17
	%	5.9	11.8	23.5	35.3	23.5	100
d) Two lanes merged to one lane before entrance	No. Responding	1	3	4	5	4	17
	%	5.9	17.7	23.5	29.4	23.5	100
e) Design speed of turning roadway not less than 70% of mainline	No. Responding	2	8	3	4	0	17
	%	11.8	47.0	17.7	23.5	0	100
f) Design speed of turning roadway between 50% and 69% of mainline	No. Responding	0	3	2	10	2	17
	%	0	17.7	11.8	58.7	11.8	100

Comments:

50 mph for part e (often).

Almost never use two-lane entrance roadway (c). Unless two lanes are added this will operate as one wide lane.

38. Describe the more important geometrics which you feel should be avoided on two-lane ramps if operational problems are to be minimized.

Geometrics to be avoided:

a. Stopping sight distance restrictions to decision points.

b. Two lanes at entrances and exits unless mainline auxiliary lanes are provided.

c. Improper combination of horizontal and vertical curves, restrictive alignment.

d. Mainline lane drop unless high percentage of traffic turns, (Exit?).

e. Abrupt tapers and sharp radii, design speeds of less than 50 mph.

f. Less than two lanes at entrance ramp gore or it will operate as one wide lane.

g. Two lanes at exit ramp gore, drop one lane at the gore.

h. Steep grades.

i. Short auxiliary lanes, should be 2000-2500 ft. for capacity.

j. Hidden exits and entrances (related to a, e, h).

k. Jointing and striping which do not coincide.

l. Jointing which requires crossing by preference or major roadway traffic.

m. Two lanes merged to one lane <u>on curve</u> before entrance (related to f).

n. Two lane ramps, if possible.

o. Lack of distance between decision points.

p. Inadequate signing and related traffic control.

q. No escape lane ahead of nose along freeway.

r. Curb at edges of pavement.

s. Proximity of adequate structures.

t. Successive entrance and exit conditions.

u. Lack of lane balance.

v. Use of two lanes on loop ramps.

w. Geometrics of lesser order than single lane ramp.

x. Providing two lanes at exit and entrance ramp terminals. (Two lanes only on ramp, branch out to two for exit, back down for entrance).

39. If a 2-lane turning roadway is merged into one lane before the entrance terminal at the through roadway, what, in your opinion, is the most desirable length and taper ratio to be used? If your answer requires qualification please note the qualifications under comments.

Merging length _____ft.

Taper ratio _____:1

Merging Length (ft.)	No. Responding	Percent	Cumulative %
200	1	5.5	5.5
400	1	5.5	11.0
600	8	44.5	55.5
800	2	11.1	66.6
1000	3	16.7	83.3
1200	3	16.7	100.0
Total	18	100.0	

Taper Ratio X:1	No. Responding	Percent	Cumulative %
20	1	4.8	4.8
35	1	4.8	9.6
40	1	4.8	14.4
50	13	61.8	76.2
60	1	4.8	81.0
70	1	4.8	85.8
80	1	4.8	90.6
100	2	9.4	100.0
Total	21	100.0	

Comments:

1000 ft. and 80:1 taper; however, slightly lower criteria such as 700 ft. or a 60:1 taper are functional.

We would prefer an auxiliary lane 2000-2500 ft. along the mainline rather than the construction on the ramp.

Personally, I do not prefer this method since it usually will occur on a curve and it is not expected by motorists, especially if preceded by a two-lane exit and the ramp proper is adequate for 60-70 mph speeds. I feel that taper ratios much greater than 50:1 are too flat to be noticed and are conducive to sideswipes.

(12 x velocity) concept -- 600 ft. response.

35:1 or possibly shorter will escape provision along the ramp shoulder for drivers unable to merge in alotted distance.

400 ft. and a 50:1 taper assuming a width reduction from 28 to 20 ft. and a design speed of 50 mph.

Merging length depends on offset.

1200 ft. and 100:1 is not a recommended procedure under conditions requiring full two-lane capacity.

200 ft. and 20:1 -- This configuration is somewhat undesirable; it results in the ramp being used for storage. Each case must be carefully examined to insure that traffic does not accumulate to the point that the other mainline is blocked.

This (600 ft. and 50:1) is assuming 12 ft. wide lanes on the ramp. The taper ratio may be adjusted if the lanes are less than 12 ft. wide, but the taper length should not be less than 400 ft. long, and the entrance lane should not be less than 12 ft. wide or the width of the right-hand through lane, which ever is greater.

40. If a two-lane turning roadway is merged into one lane before the entrance terminal at the through roadway, which lane do you think should be dropped?

	Number Responding	Percent
Left Lane	7	33.3
Right Lane	12	51.1
Either Lane	2	9.6
	21	100

Comments:

Right Lane:

Conforms to driver expectancy on ramps -- my answer for a right side merge onto freeway -- (a) is probably better for left side merges to freeway.

Only for consistency.

Wider shoulder and lower speeds are on right side

Right lane should be dropped unless for some reason large majority of traffic is already in the right lane in which case left lane should be dropped.

This question can probably be debated either way -- prefer merging from right to left.

Left lane merge may invite left lane vehicle to enter mainline before desired.

Left-to-right merge is difficult.

Easier to merge.

Caution should be the burden of the right-hand or "slower" driver.

Left Lane:

Dropping the lane on the left will normally affect fewer vehicles. I believe this to be more desirable even though the vehicles on the left may be operating at a higher speed.

Normally, less traffic will utilize the left lane of a two-lane exit roadway, which can effect a lane drop more readily.

Assuming conditions other than capacity require two lanes, such as storage requirements or a truck climbing situation.

Either Lane:

When entering the through roadway from the right, then drop the right turning roadway.

41. Figures 23 and 24 indicate two methods for merging a 2-lane entrance ramp. Place an x under the use category which characterizes the frequency with which your organization currently uses each method.

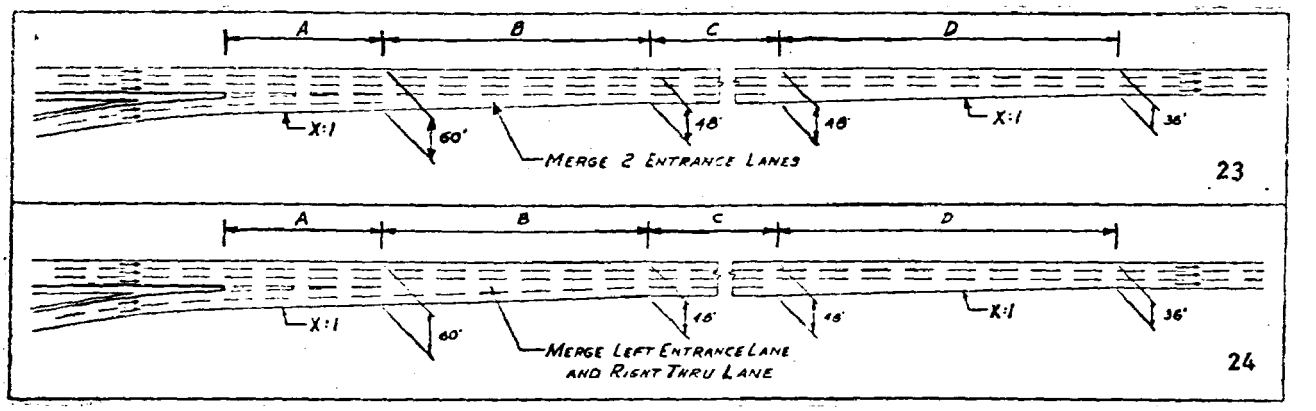


Figure Number		Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total
23	Number Responding	6	4	1	3	2	16
	%	37.5	25.0	6.3	18.8	12.5	100
24	Number Responding	1	1	3	2	9	16
	%	6.3	6.3	18.8	12.5	56.3	100

Comments:

I believe operation as in #24 is very undesirable.

When a two-lane entrance ramp is brought into the mainline, the mainline should have an added lane ahead. Neither figure shows this.

We tried #24 and it does not work. We do not believe double lane drops work and have proved the same on several locations.

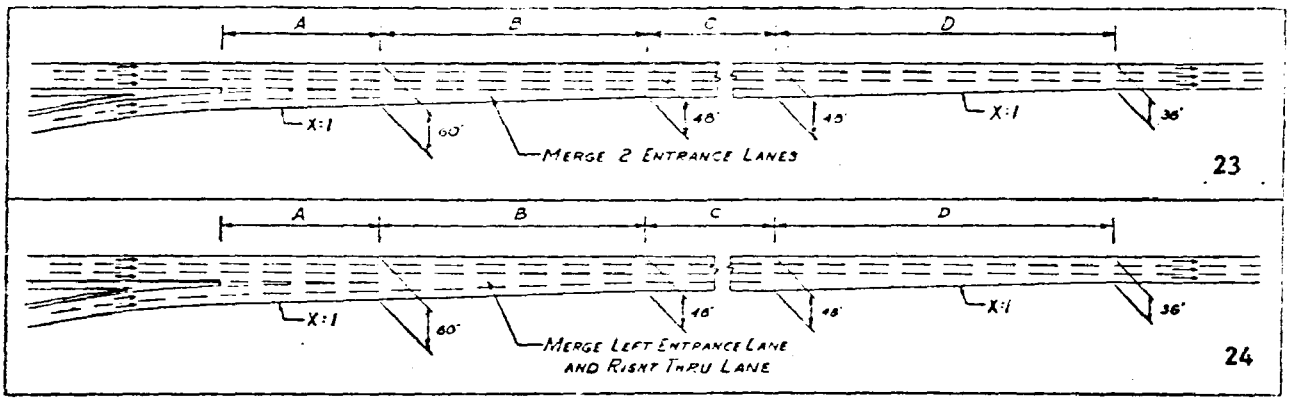
Figure 23 gives a much better alternative for an escale lane (shoulder) than 24.

Does not seem practical to use two lane ramp on and not increase the mainline.

Because most of the existing system is like Figure 24, in the Chicago Region, it prefers to continue this type of design.

My personal preference is for figure 24.

42. Using Figures 23 and 24 as a reference please indicate your opinion as to the minimum and desirable values for the following dimensions for a 2-lane entrance ramp.



Dimensions

A. Distance from nose to start of merge

B. Length of first merge

C. Length of parallel auxilliary value

D. Length of second merge

X. Convergence ratio

Fig. 23 Minimum "A"

A (in feet)	No. Responding	Percent	Cumulative %
0	1	9.1	9.1
200	1	9.1	18.2
280	1	9.1	27.3
400	1	9.1	36.4
450	1	9.1	45.5
500	2	18.2	63.7
600	3	27.2	90.0
900	1	9.1	100.0
Total	11		

Fig. 23 Desirable "A"

A (in feet)	No. Responding	Percent	Cumulative %
0	1	9.1	9.1
300	1	9.1	18.2
400	1	9.1	27.3
450	1	9.1	36.4
500	2	18.2	54.6
600	1	9.1	63.7
750	2	18.2	81.9
800	1	9.1	91.0
1000	1	9.1	100.0
Total	11		

Fig. 23 Minimum "B"

B (in feet)	No. Responding	Percent	Cumulative %
400	1	9.1	9.1
600	6	54.5	63.6
750	1	9.1	72.7
850	1	9.1	81.8
900	1	9.1	90.9
1000	1	9.1	100.0
Total	11		

Figure 23 Desirable "B"

B (in feet)	No. Responding	Percent	Cumulative %
600	5	45.4	45.4
840	1	9.1	54.5
900	2	18.2	72.7
1000	1	9.1	81.8
1200	2	18.2	100.0
Total	11		

Figure 23 Minimum "C"

C (in feet)	No. Responding	Percent	Cumulative %
0	2	18.2	18.2
400	1	9.1	27.3
500	1	9.1	36.4
600	1	9.1	45.5
800	1	9.1	54.6
900	1	9.1	63.7
1300	1	9.1	72.8
1500	1	9.1	81.9
1600	1	9.1	91.0
2000	1	9.1	100.0
Total	11		

Fig. 23 Desirable "C"

C (in feet)	No. Responding	Percent	Cumulative %
600	1	11.1	11.1
750	1	11.1	22.2
1000	2	22.2	44.4
1200	1	11.1	55.5
1600	1	11.1	66.6
2000	1	11.1	77.7
2500	2	22.2	100.0
Total	9		

Fig. 23 Minimum "D"

D (in feet)	No. Responding	Percent	Cumulative %
400	1	9.1	9.1
600	7	63.6	72.7
750	1	9.1	81.8
850	1	9.1	90.9
1000	1	9.1	100.0
Total	11		

Fig. 23 Desirable "D"

D (in feet)	No. Responding	Percent	Cumulative %
600	6	54.5	54.5
840	1	9.1	63.6
900	1	9.1	72.7
1000	1	9.1	81.8
1200	2	18.2	100.0
Total	11		

Fig, 23 Minimum "X"

X:1	No. Responding	Percent	Cumulative %
15	1	9.1	9.1
30	1	9.1	18.2
50	7	63.6	81.8
70	1	9.1	90.9
80	1	9.1	100.0
Total	11		

Fig. 23 Desirable "X"

X:1	No. Responding	Percent	Cumulative %
15	1	9.1	9.1
50	6	54.5	63.6
60	1	9.1	72.7
70	1	9.1	81.8
100	2	18.2	100.0
Total	11		

Fig. 24 Minimum "A"

A (in feet)	No. Responding	Percent	Cumulative %
300	2	50	50
400	1	25	75
660	1	25	100
Total	4		

Fig. 24 Desirable "A"

A (in feet)	No. Responding	Percent	Cumulative %
400	1	25	25
600	2	50	75
660	1	25	100
Total	4		

Fig. 24 Minimum "B"

B (in feet)	No. Responding	Percent	Cumulative %
600	1	25	25
690	1	25	50
700	1	25	75
1000	1	25	100
Total	4		

Fig. 24 Desirable "B"

B (in feet)	No. Responding	Percent	Cumulative %
800	1	33.3	33.3
900	1	33.3	66.6
1200	1	33.3	100.0
Total	3		

Fig. 24 Minimum "C"

C (in feet)	No. Responding	Percent	Cumulative %
300	2	50	50.0
2000	2	50	100.0
Total	4		

Fig. 24 Desirable "C"

C (in feet)	No. Responding	Percent	Cumulative %
400	1	25	25
1300	1	25	50
2500	2	50	100
Total	4		

Fig. 24 Minimum "D"

D (in feet)	No. Responding	Percent	Cumulative %
600	1	25	25
690	1	25	50
700	1	25	75
1000	1	25	100
Total	4		

Fig. 24 Desirable "D"

D (in feet)	No. Responding	Percent	Cumulative %
900	2	66.7	66.7
1200	1	33.3	100.0
Total	3		

Fig. 24 Minimum "X"

X:1	No. Responding	Percent	Cumulative %
50	1	25	25
57	1	25	50
60	1	25	75
70	1	25	100
Total	4		

Fig. 24 Desirable "X"

X:1	No. Responding	Percent	Cumulative %
57	1	25	25
70	1	25	50
75	1	25	75
100	1	25	100
Total	4		

43. Figures 25, 26 and 27 indicate three methods for merging two 2-lane roadways into one 3-lane roadway. Place an "x" under the use category which characterizes the frequency with which your organization currently uses each method.

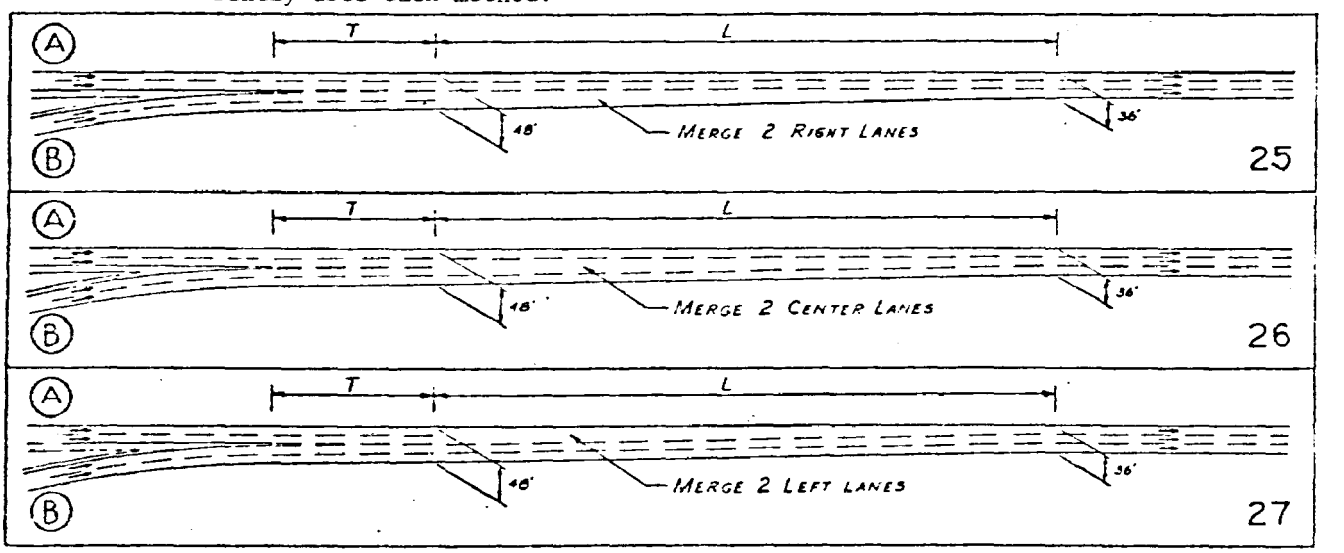


FIGURE		Almost Always (96-100%)	Usually (66-95%)	Often (36-65%)	Sometimes (6-35%)	Almost Never (0-5%)	Total
25	No. Responding	6	4	2	4	1	17
	Percent	35.3	23.5	11.8	23.5	5.9	100
26	No. Responding	1	1	2	4	9	17
	Percent	5.9	5.9	11.8	23.5	53.0	100
27	No. Responding	0	0	1	4	12	17
	Percent	0	0	5.9	23.5	70.6	100

Comments:

Fig. 26 will not work. We have used with some success but we believe that it enhances operation if lane is chopped at exit ramp if T distance is less than 1500'. (Texas)

44. Presented below are several statements concerning the use of weaving areas on major interchanges. Indicate your agreement or disagreement with each of these statements by encircling the appropriate word.

Statement 1. In general, weaving areas in major interchanges should always be avoided.

a. Strongly Agree b. Agree c. Disagree d. Disagree Strongly

	Number Responding	Percent
Strongly Agree	4	19.1
Agree	13	61.8
Disagree	4	19.1
Disagree Strongly	0	0
	21	100

Statement 2. In general, weaving areas in major interchanges can be justified if adequate weaving length is provided.

a. Strongly Agree b. Agree c. Disagree d. Disagree Strongly

	Number Responding	Percent
Strongly Agree	1	4.8
Agree	13	61.9
Disagree	7	33.3
Disagree Strongly	0	0
	21	100

Statement 3. In general, weaving areas in major interchanges can be justified if all weaving occurs off the through roadway.

a. Strongly Agree b. Agree c. Disagree d. Disagree Strongly

	Number Responding	Percent
Strongly Agree	3	14.3
Agree	13	61.9
Disagree	5	23.8
Disagree Strongly	0	0
	21	100

45. (Deleted)

46. The amount and quality of feedback design engineers get regarding operation and safety of interchanges seems to vary considerably from organization to organization. Please comment below on the ways in which you think feedback could be provided so that the information would be useful on subsequent designs.

Research studies.
Operational reports

The Maintenance Division could issue periodic reports on operational problems and supply copies to the Design Subdivision. (New York)

Our Department is on the "non-centralize" system and all of the interchange design is accomplished in the district offices and approved by the Austin Office. Therefore, we have very good feedback because our design engineers live with the problems they create.

Periodic written reports from operations should be made to design on performance of various design features. Standardization of accident reporting would be very desirable so that correlation with design features would be possible.

1) Accident data in a better form (summarized so that designer could more readily get the overall picture).

2) Actual traffic counts taken by experienced men who could make determinations, estimations as to the level of service, and other factors so designers would have a better feel or understanding what 600 vph or 1000 vph would mean. (films would be great.)

All major interchanges constructed should be automatically subjected to a 1-2 year special study of operation and safety aspects. The data should be sent to the Central Headquarters, analyzed, and the results used to influence subsequent designs.

Accident reports with copies of investigation results, if any, should be routinely transmitted to design for the first few years after opening a new facility. Copies of comments from public, police, maintenance should also be sent to design. Traffic studies should be made at frequent intervals and made available to Design.

1) Deliberate program of feedback from maintenance and safety offices regarding outstanding deficiencies in operation and why. Also report outstanding designs.

2) Nationwide reporting of same as above.

Maintenance men
Traffic operations division
Fire departments
Local and state police
Trucking associations
Traffic safety division

Bulletins published through one agency -- AASHO or FHWA. Too many agencies or firms have been publishing safety data, depending on what is being sold or published.

Engineers drive completed facility

1) During and upon completion of construction, receipt of as-built drawings which specify geometric changes because of constructability problems.
2) Visit and record driver behavior at interchange on first day of operations.
3) During operations, feedback concerning accident locations, special sign requirements and experienced operational data regarding speeds, volumes, and geometrics which can be correlated to original or as-built design.

Experienced traffic operations engineers should be an integral part of the design team.

1) Closer communication between design, maintenance, and enforcement personnel.
2) Designers spending more time in field observing traffic operation.

Multi-disciplinary diagnostic team to evaluate projects (see Research Report 606-8 copy enclosed).

APPENDIX C

WORKSHOP ATTENDEES & AGENDA

APPENDIX C: WORKSHOP ATTENDEES & AGENDA

Invitations for the two workshops held in August were sent to selected state highway engineers, Federal Highway Administration officials, private consulting engineers, and academic-research engineers. Representative of the top highway designers and design policy makers in the country, the attendees are identified in Table C-1.

Held over a period of three days, each workshop consisted of 13 discussion sessions approximately 90 minutes in length. (See Table C-2.) 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 completed pre-workshop questionnaires.
- Presentation of a set of prepared questions, and encouragement of expressions of opinions and relating of pertinent experience by the workshop participants. 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.

TABLE C-1. Workshop Attendees

First Workshop (August 1-3, 1972)

Biggs, Raymond G.
Project Superxisor (Houston)
Texas Dept. of Transportation

Coble, Kenneth L.
Consulting Engineer
Sverdrup & Parcel

Fields, Marvin
Field Engineer, Bureau of Traffic
Ohio Department of Highways

Foy, Robert A.
Chief Engineer, Design Division
Wilbur Smith & Associates

Gray C. William
Design Development Engineer
Ohio Department of Highways

Hall, Parker L.
Assistant Engineer of Design
California Division of Highways

Hess, Joseph W.
Office of Research (FHRS)
Federal Highway Administration

Housworth, Jack L.
Supervising Design Engineer
Texas Dept. of Transportation

Hurd, Fred W.
Professor, Civil Engineering
The Penna. State University

Kenyon, Alan D.
Associate Civil Engineer
New York Dept. of Transportation

Link, James
Office of Development
Federal Highway Administration

McCausland, Walter
Design Engineer
Federal Highway Administration

McCoy, William D.
Assistant State Highway Urban Engineer
Georgia Highway Division

Mueser, Robert R.
Deputy Chief Highway Engineer
Penna. Department of Transportation

Nemeth, Dr. Zoltan
Civil Engineering Department
Ohio State University

Stockfisch, Charles R.
Office of Research (FHRS)
Federal Highway Administration

Second Workshop (August 22-24, 1972)

Alexander, Dr. Gerson J.
Office of Traffic Operations
Federal Highway Administration

Byington, Stanley R.
Office of Research
Federal Highway Administration

Churchill, Robert R.
Deputy Design Engineer
Florida Dept. of Transportation

Lins, William F.
Chief, Bureau of Highway Design
Maryland Dept. of Transportation

Loutzenheiser, Donald W.
Director, Office of Engineering
Federal Highway Administration

McCausland, Walter
Design Engineer
Federal Highway Administration

TABLE C-1 (Continued)

Ebersole, Glenn
Traffic Research Engineer
Pennsylvania Dept. of Trans.

Foster, W. M.
Assistant Director
Washington State Highway Dept.

Gazda, Andrew J.
Engineer of Geometric Design
Illinois Dept. of Transportation

Glennon, Dr. John C.
Manager, Traffic Safety Center
Midwest Research Institute

Hofmann, Frederick J.
Senior Highway Engineer
Edwards and Kelcey, Inc.

Huckins, Edgar W.
Assistant Highway Design Engineer
New Hampshire Dept. of Pub. Wks.

Lee, Bumjung
Research Associate
Polytechnic Institute of Brooklyn

Pilkington, George
Office of Research
Federal Highway Administration

Randich, Gene M.
Vice-President
Deleuw, Cather Organization

Ricker, Edmund
Chief - Highway Safety Group
Pennsylvania Dept. of Transportation

Sigal, Andre H.
Associate Civil Engineer
N.Y. State Dept. of Transportation

Stockfisch, Charles R.
Office of Research
Federal Highway Administration

Taragin, A.
Office of Traffic Operations
Federal Highway Administration

TABLE C-2. Workshop Agenda

First Day

Introduction by James I. Taylor

Discussion on Standardization; Classification; Adaptability led by Richard A. Olsen

Discussion on Configuration Evolution Led by Robert Hostetter

Discussion on Design Sequence; Checklists led by John Hayward

Second Day

Discussion on Trade-offs; Cost-effectiveness; Level of Design led by James I. Taylor

Discussion on Visibility Analyses, Driver Perception, design led by Richard A. Olsen

Discussion on Exits led by Ronald J. Slavecki

Discussion on Entrances led by Robert Hostetter

Discussion on New Designs led by John C. Hayward

Third Day

Discussion on Lane Drops; Lane Balance led by Ronald J. Slavecki

Discussion on Route Continuity; Ramp Arrangements led by John Hayward

Discussion on Local Access; Freeway Control; Bus Lanes led by Robert Hostetter

Conclusion by James I. Taylor

APPENDIX D

SELECTED POST-SESSION QUESTIONNAIRE RESULTS

APPENDIX D: SELECTED POST-SESSION QUESTIONNAIRE RESULTS

This appendix contains the results of the post-session questionnaires which were distributed to all workshop attendees immediately following the workshop discussion sessions. The questionnaires were distributed so that the workshop participants could provide written opinions or statements concerning topics which had been discussed during the preceeding workshop session. These written responses were intended to reinforce the findings and trends noted in the pre-workshop questionnaire and to distill the session discussions, as well as draw out opinions from some of the less vocal workshop attendees.

The post-session questionnaire results for some of the sessions have been deleted from this appendix since they are directly contained within the text of the preceeding report. Results from the following workshop sessions are tabulated in this appendix in the following order:

1) Exits;
2) Two-lane entrances;
3) Lane drops and lane balance;
4) Route continuity.

Each question will be reproduced as it was presented to the participants followed by a summary of the answers received. Except for those questions which are indicated, the answer matrix will include the results of both workshops combined. Comments on each question were encouraged and many of those received will be presented following the answer matrix.

EXITS

1. Cloverleafs are not adaptable for freeway-to-freeway interchanges, except possibly in rural areas where turning volumes are relatively low, and then only when the design includes collector-distributor (C-D) roads. (Agree or Disagree?)

	Strongly Agree	Agree	Disagree	Strongly Disagree	Totals
Design	6	9	3	1	19
Operations	1	1	3	0	5
Research	1	1	4	1	7
Total	8	11	10	2	31

Comments:

The design can be made without C-D roads if the design is such that C-D roads can be provided in the future when volumes and weaving indicate need.

Cloverleafs are quite acceptable in low volume rural areas without C-D roads and where a route is not turning. Even in some suburban areas where 50 mph speeds are used and weaving can be accommodated at level of service D and space is available, cloverleafs are acceptable if space is available and the level of service on the mainline is D or less. C-D roads on cloverleafs are not always cost-effective.

Although the construction cost is higher, directional interchanges are the only type we should build because they will be adequate for more years.

Only because of cost do I believe that cloverleafs should ever be used.

Weaving sections should not be placed on the mainline.

From a safety standpoint, cloverleafs are usually no problem if the loop is on an upgrade and clearly visible to the driver.

Loops might be used for turns of minor volumes.

Generally I agree; however, we feel that the C-D roads are the controls. If you can design C-D roads of adequate length to handle the weaving volumes, we see no reason to abandon the cloverleaf design as an alternate in any location.

They can be used successfully without C-D roads, most likely where there are not directional interchanges in the area and where volumes are low.

When through volumes are also very low, the weaving adjacent to the main lanes may be accetable.

Short weaves of < 1,800 ft. should not be permitted on any freeway mainline.

It depends on several factors; e.g., the balance of turning volumes. A C-D is not always a good answer.

Where turning volumes are low, a C-D should not be needed.

In my opinion, if low volumes are prevalent, an adequately long deceleration lane for achieving an appropriate speed reduction appears satisfactory.

It is not quite practical to spend a million dollars to save a few injuries when there are pressing social problems which also need funds.

In rural areas C-D roads may not be required. But this would seem to be an exception rather than a rule. If C-D roads are not used, the designer should be made to justify it just as strongly as if he were proposing a very complex design.

2. The disfavor with which left-hand exits are held by engineers stems more from subjective speculation than from the results of factual, objective studies. (Agree or Disagree?)

	Strongly Agree	Agree	Disagree	Strongly Disagree	Totals
Design	0	3	12	3	18
Operations	1	2	1	1	5
Research	0	4	2	1	7
Total	1	9	15	5	30

Comments:

Left-hand exits are adequate with a two-lane roadway, if they are signed and lighted properly.

I have formed my opinion of disfavor with left exits from the combination of having read results of studies of the subject, from my own experience with many such designs, and from knowing the opinions of other highway engineers. Any single published study I know of would not by itself prove the point.

I think facts are available. The definition of a left-hand exit needs clarification. The following is bad:

There are several reports and a Congressional Hearing that indicate left-hand exits are not desirable.

My state collects some accident data, and I am sure that it will show that left exits have high accident experience.

All variables have not yet been controlled in existing research.

Both factors are involved; however, study results appear to substantially predominate.

Although not many states, agencies, etc. have conducted their own research since most do not have enough situations to warrant statistical significance, they probably have, as Illinois, contributed funds toward regional or national studies by professional organizations to research the problem and provide results. One such study is the Illinois Cooperative Highway Research Project, IHR-61. I do not feel these can be overlooked just because they are not conducted by the utilizing agency.

I must admit that good documentary evidence is not what it should be, but we have good, practical, visual evidence that left-hand exits are poor. We have designed and built a number of them. Without exception, traffic conflicts can readily be observed at each, far out of proportion to the volumes.

I do not really know. I would like to think that some operational research is available.

It is based on known operational problems.

Local accident data has led many teams to this conclusion. There has been no big national "pull-together" report.

The disfavor is based mostly on sad experience (operations and safety) with left-hand off-ramps.

I do not believe that the speed difference between lanes on most freeways is so large that an adequate deceleration lane will not do the job. Perhaps the only problem that arises is where an entrance ramp is so close to the exit ramp that commercial vehicles who entered at the entrance ramp have inadequate space (longitudinal) to merge left to exit.

There are a few studies related to left-hand ramps.

Accident statistics will usually reflect the poor design in choosing to use a left-hand exit.

Left entrances have been found to be worse than left exits from a safety standpoint.

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3. If a left-hand exit must be used, a parallel-type left lane should be added to the mainline to remove exiting traffic from the high-speed through lane. (Always or Never?)

	Always	Usually	Sometimes	Rarely	Never	Total
Design	10	6	0	1	0	17
Operations	2	0	1	0	0	3
Research	2	3	0	1	0	6
Total	14	9	1	2	0	26

Comments:

This is Ohio's standard for either left or right exits.

The parallel type deceleration lane is rarely used in Texas.

It is highly desirable to facilitate advance signing and provide vehicle orientation. In addition, this provides a deceleration lane out of the high-speed lane of the through facility.

I want to say "always," but I cannot be quite that positive! However, the more opportunity for early decision-making on left-hand exits the better. This feature should be included as a "given" to be deleted only in rare or the most unusual situations and then only if a good escape zone can be included ahead of the ramp.

It depends some on mainline alignments. It needs special treatment, and an added lane is one part. Special advance signs are also needed.

You need adequate distance for signing, preferably overhead.

Sometimes, depending on volumes, percent of heavy trucks, and topography (vertical alignment).

To prevent a reduction of speed on the mainline.

Especially true if the mainline is curving to the right. A parallel lane will help reduce the speed differential between successive vehicles in the high-speed lane.

4. Should a federal standard be adopted prohibiting the use of left-hand exits at major interchanges? Why or why not?

	Yes	No	Total
Design	2	15	17
Operations	1	3	4
Research	2	5	7
Total	5	23	28

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Yes

They are accident prone.

Left-hand exits are inherently hazardous.

On Interstate and maybe U.S. Routes.

If it allows leeway in cases where there is no other feasible alternative.

No

If properly designed, they are safe.

The standard should not prohibit, but should make their use very restricted. There are cases where all right-side exits would not be feasible.

The FHWA will not approve left-hand exits unless they are major splits.

There is always the unusual case where one may be necessary although not desirable.

There are instances when they must be provided. This should be left to the states.

No, because there is little difference to the driver between a directional split of a freeway and a freeway-to-freeway left ramp.

Under certain conditions left-hand exits may be warranted.

With good signing it does not have to be a problem. At complex interchanges it may be a very economical solution (less ROW).

Such a policy would not be desirable. Although the left-hand exit is not generally preferred, it can be successfully used when properly designed and signed.

No, because there is no standard which can be substituted for engineering judgment.

These exits are poor. They should be used only as a last resort. Nevertheless, they are a tool or a method that can be used, if well designed, with at least satisfactory results where the topography may be such that nothing else would be tolerated. Federal standards are totally inflexible and do not recognize special conditions.

Extreme conditions may dictate the use of the left exit.

They may have to be used. They can be designed given the right situation.

There may be certain cases where this is the only practical solution to the problem.

They are second choice forms, but may be better than none at all. We can design, sign, and mark them to work well, but in earlier cases we did not do so.

I do not consider federal standards to be the proper method to obtain good designs. There are cases in major interchanges where the left exit may be the most desirable solution.

The designer should have a wide degree of flexibility in design approach -- weighing cost, site, etc.

No, because a general prohibition may create problems in some cases when a left-hand exit is the only way to solve the problem.

There are exceptions in which left exits can be justified -- depending on volumes; land available for building the interchange, etc.

Other

If this includes a major fork, no; if not, then yes.

5. Direct taper off-ramps are, in general, superior to parallel-lane off-ramps. (Agree or Disagree?)

	Strongly Agree	Agree	Disagree	Strongly Disagree	Total
Design	4	9	3	3	19
Operations	0	3	1	1	5
Research	0	3	3	1	7
Total	4	15	7	5	31

Comments:

In the case of an accident on the off-ramp the parallel lane is good for safe storage (the paved shoulder also).

I believe that one type of exit should not be used exclusively. The tapered exit is advantageous under normal traffic conditions. Where excessive ramp volumes may occur, the parallel lane should be used.

Parallel-type deceleration lanes are superior to tapered-types!

I strongly agree for a single lane and low volumes. This is not true for two-lane branches, and not necessarily true for high volume single exits.

One is as good as another, though we use the direct taper.

The parallel, if designed properly, will have all the advantages of the direct taper as well as its own advantages.

It would depend on the location with relation to cross streets which may cause capacity problems requiring storage adjacent to the freeway.

The parallel lane has merit where the exit condition is less than desirable for various possible reasons.

I believe the results of the 1960 AASHO Special Study indicate that it is generally true that direct taper deceleration lanes are superior.

Generally I agree since many motorists mistake the parallel lane for an added lane. I agree only if the direct taper has an adequate escape zone, however.

It more nearly approximates driver activity.

Best for low volume rural areas -- driver use pattern verifies this. It is the natural turning motion.

It must be kept in mind that certain factors, such as curves creating poor sight distance, could dictate parallel-lane off-ramps.

Either can be designed to work well. Tapered is in favor for non-major interchanges, but views are divided for use at major ones. It is not correct to say that a tapered type is "superior" for all cases.

There are special cases, such as exits on the outside of horizontal curves, where the use of the parallel lane is preferable.

On high volume highways, I believe that long parallel lanes are safer and less restrictive to possible capacity problems.

Only when there is no tangential off-ramp.

New Jersey Turnpike experience indicates that parallel lanes are good.

Tapers, in my opinion, provide less direction to drivers -- depends on striping. How is the driver guided along the tapered section? I believe the only problem with acceleration and deceleration lanes is that we have not taught drivers what they are for and how to use them.

Getting exiting traffic away from the through lane is desirable.

Both can be used depending on the situation. If the road curves to the left, a taper may be used rather than a parallel lane. In urban areas where density is usually high, the parallel lane can increase capacity at the ramp. Parallel lanes can operate as a taper when traffic demands drop off.

6. Indicate with an "X" in the appropriate column which type of off-ramp you would prefer under the following conditions: (Direct taper vs. Parallel for various locations).

RIGHT HAND EXITS

	A. Mainline Tangent			B. Mainline Turning Left (Right Below)		
	Direct Taper	Parallel Lane	Total	Direct Taper	Parallel Lane	Total
Design	11	5	16	3	13	16
Operations	2	2	4	2	2	4
Research	4	3	7	3	4	7
Total	17	10	27	8	19	27

	C. Exit Over Crest			D. Mainline Downgrade		
	Direct Taper	Parallel Lane	Total	Direct Taper	Parallel Lane	Total
Design	3	13	16	10	5	15
Operations	1	3	4	4	1	5
Research	1	6	7	5	2	7
Total	5	22	27	19	8	27

6. (Continued)

LEFT-HAND EXITS

	A. Mainline Tangent			B. Mainline Turning Right		
	Direct Taper	Parallel Lane	Total	Direct Taper	Parallel Lane	Total
Design	5	9	14	1	13	14
Operations	2	2	4	1	3	4
Research	2	4	6	2	4	6
Total	9	15	24	4	20	24

	C. Exit Over Crest			D. Mainline Downgrade		
	Direct Taper	Parallel Lane	Total	Direct Taper	Parallel Lane	Total
Design	1	13	14	5	9	14
Operations	1	3	4	2	2	4
Research	1	5	6	3	3	6
Total	3	21	24	10	14	24

7. Assuming a highway design speed of 70 mph and an exit ramp speed of 50 mph, the "Blue Book" (1965a) suggests a deceleration lane of 350 feet (of which 300 feet is the length of the taper).

a. Do you feel this length is adequate?

	Yes	No	Total
Design	7	10	17
Operations	1	4	5
Research	2	4	6
Total	10	18	28

b. If you feel the length is inadequate, what length would you suggest?

Minimum Length (ft.)	Number Responding
335	1
400	1
450	1
500	6
600	2
700	2
800	3
1,000	1
Total	17

Desirable Length (ft.)	Number Responding
650	2
700	1
800	2
1,000	5
1,200	2
2,000	1
Total	13

8. In general, a single exit is superior to a double exit (two successive exits) in terms of driver comfort and confidence in a semi-directional interchange. (Always or Never?)

	Almost Always	Usually	Occa-sionally	Rarely or Never	Total
Design	8	8	2	1	19
Operations	1	1	1	0	3
Research	4	3	0	0	7
Total	13	12	3	1	29

9. Has enough research been done, or do you have enough results from your own experience that you are confident you understand the benefits of a single or a double exit on a semi-directional interchange? (Confident or Unsure?)

	Totally Confident	Reasonably Confident	Some Doubt	Unsure	Total
Design	4	12	1	1	18
Operations	1	1	1	1	4
Research	0	1	5	1	7
Total	5	14	7	3	29

TWO-LANE ENTRANCES

1. What are the primary operational problems you have observed on two-lane entrance ramps? (Comment)

   Inadequate signing, lighting, and overhead signal control. These three elements can increase safety and capacity. The cost is low, but present policy prevents it.

   The breakdown of traffic flow in at least one lane occurs at a critical decision-making point of traffic flow.

   The merging length is too short and there is usually a lack of extra lane length beyond the entrance nose before merging to the minimum number of through lanes.

   Capacity, lane orientation, striping difficulties.

   Insufficient gaps in the right lane of the through roadway during peaks.

   The design of entrance and exit terminals. When designed as a major fork, they have been much more effective and provide smoother operations.

   Merging where separate lanes cannot be carried ahead.

   They function as only one lane due to inadequate merge distance. There are problems if one lane is not continued.

   Compounded merging.

   Insufficient lanes going ahead. If L_m = the number of mainline lanes and L_R = the number of ramp lanes:

   $$L_m + L_R \text{ to } L_m + L_R \ \underline{\text{or}}\ L_m + L_R - 1 \text{ lane} = \text{O.K.}$$

   $$L_m + L_R \text{ to } L_m + L_R - 2 \text{ lanes} = \text{insufficient lanes.}$$

   The right-hand ramp lane traffic attempting to merge with the mainline too early; i.e., moving across the left-hand ramp line immediately past the gore area.

   Excessive lane changing; failure to use both lanes by "country" drivers; operating too slowly in the inner lane must merge. (The above operational experience is based on older designs where the inner lanes were merged and the overall design was too short.)

   We have two-lane directions merging with two lanes and have no problems.

Merging is a problem if an additional lane is not provided, the length of acceleration lanes are not long enough, or acceleration lanes are not dropped as two separate lanes but as one continuous merge.

I have heard of merge backup and accident problems.

When an additional lane is not added to the freeway, merging creates congestion when the main lanes are near capacity.

Erratic driver patterns at or just before the nose; uncertainty in desired position.

Confusion regarding lane assignments; sideswipe collisions; outer lane drivers trapped on the acceleration lane.

There is a problem with weaves, unless the exit ramp is well down the road.

Driver confusion.

a. Not functional in the sense that drivers will not use it as a two-lane facility, unless they are operating at capacity, and then the merging problem is paramount.

b. Their safety record is not very good.

Unbalanced use of lanes; merging difficulties for inner lane traffic resulting in vehicles stopping on a through-type facility.

2. Given a two-lane entrance ramp where only one freeway lane is added, please indicate your preference for the merging lane configurations shown in Figure D-1 by giving the most desirable configuration a 1 and the least desirable a 3.

	Average Rank (Both Workshops Combined)		
	A Inner Lane Merged	B Outer Lane Merged	C Non-Compulsory Merge
Design	2.4	1.6	2.1
Operations	2.8	1.5	1.8
Research	2.4	1.5	2.2
Total	2.4	1.5	2.2

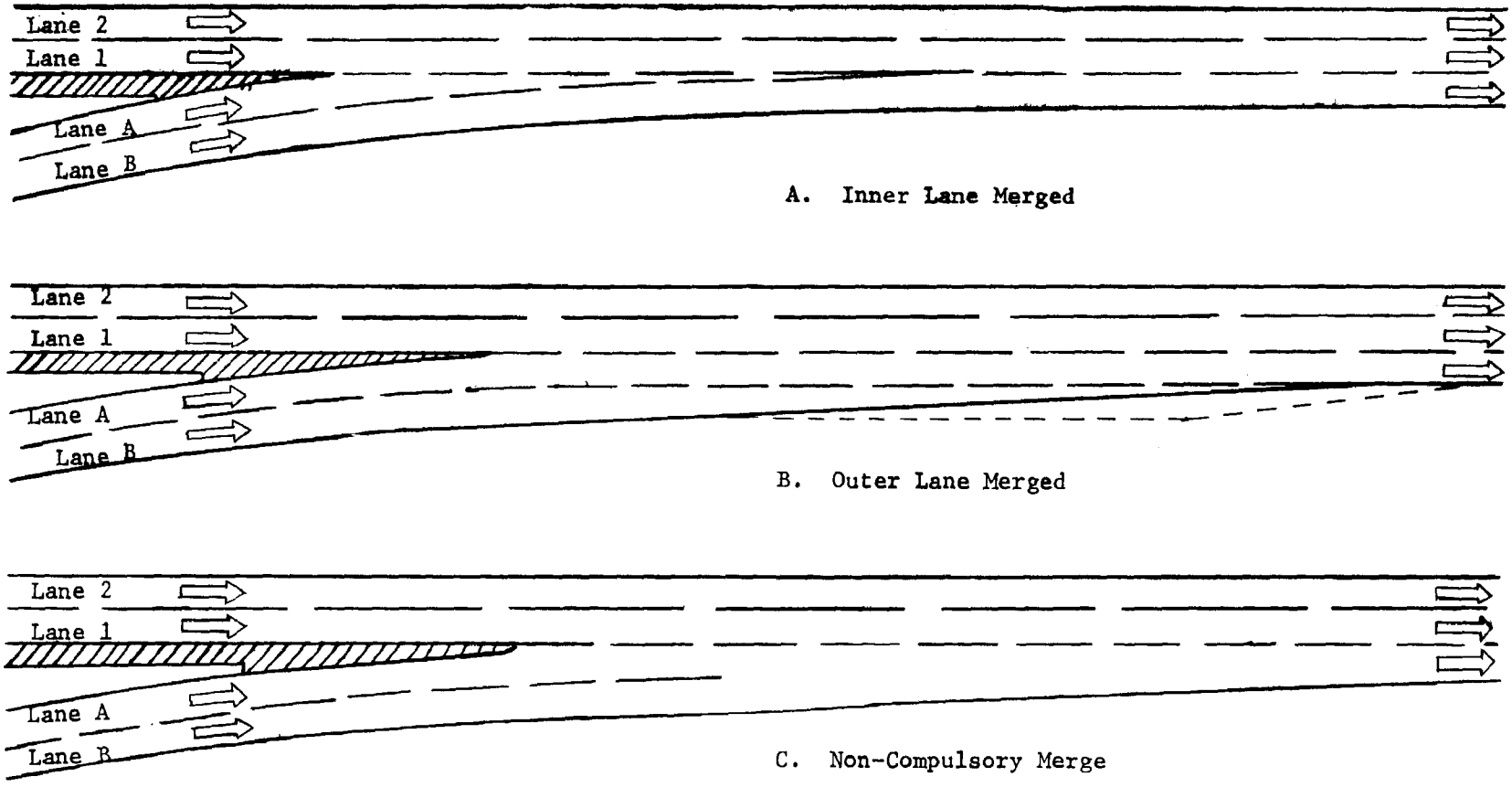


Figure D-1. Two-Lane Entrance Ramps, Merging Lane Configurations

3. Please comment on the factors which determined your ranking of the merging configurations.

We need lighting, good signing, lane marking, and public education to make these things work. A stranger, when traffic is heavy, follows the car in front. Good traffic control devices take care of the stranger during low volume periods and at night.

In my view, the merging process is a merging of ramp lanes and should be accomplished as a ramp function so that conflicts and interruptions of flow are not created for the mainline lanes.

1. Flexibility of traffic volume ranges on each roadway.
2. When requiring the outer lane (lane B) to merge, it is believed to require more vehicles to change lanes, especially during lower-volume, non-peak hours.

It is best to lose the lower-speed outside lane. It is more consistent with the merge left concept. Drivers expect this. "A" creates a "squeeze play."

New York State will not build a ramp with the left lane merged.

"A" traps the unfamiliar driver with a merge left or right with no place to go.

I feel a person should always be given an escape lane; therefore, I prefer the outer lane merge.

B and C are basically the same.

A -- the squeeze merge is unsatisfactory.

I would prefer the addition of two lanes with a right-side lane drop after the maneuver. It would change from a merge to a lane change.
Eliminate the squeeze.
Elemental merge.
Keep intersectional maneuver simple.
One decision at a time.
Provide more distance for the lane change.

B - Clear case - Lane B must merge, but it has an escape route (shoulder).
C - Not so clear - Lanes A and B merge into one lane on a "first come-first served" basis.
A - Drivers in lane A have no escape route.

The outer lane merge has better driver visibility, lower operating speeds, and normally more available recovery area than either of the other designs. The non-compulsory merge is indecisive in its instructions, leading to gross confusion.

The outer lane, being of generally lower speed, has the best opportunity with the least conflict, and also has more escape options.

Reason for ranking (C,1; B,2; A,3) include:

1. My observation of the inner lane merge design (our former policy).
2. When turbulence occurs near peak hours, the inner merge accidents have always been high number multiple-types and more serious because of no escape route.
3. The outer lane merge has a full paved shoulder for escape or auxiliary lane usage, and the accident experience usually involves only minor rear-end collisions.
4. It is my recollection that the special ITE study of this matter concluded that either design is satisfactory if uniformity is used within the locate.

The shoulder provides an escape value, and there are less potential vehicle conflicts if the outer lane is merged.

The public expects the right or outer lane to merge. It has worked better in Maryland. It is simpler to sign, and the shoulder can be used for an escape lane. If an accident occurs, it will involve less cars at lower speed.

Do not force the inner lane driver into a squeeze by ending his lane where he is trying to pull into the through lane.

The inner lane merge does not permit any area for escape if a gap is not available. The outer lane traffic always has the shoulder as an escape area which is a safety factor.

I do not like pinching out an interior lane.

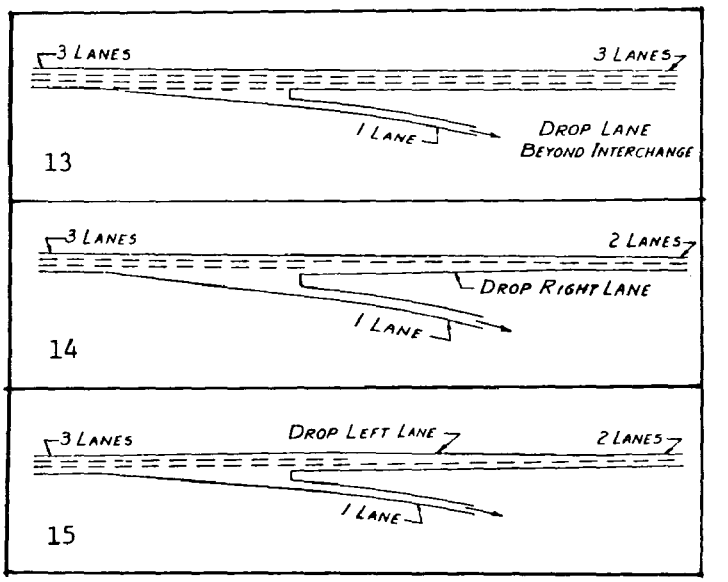
Desirability of having an escape shoulder on the right.
Normal merge is to the left.
Prevent a driver from having to decide whether to merge left or right.

The effects on the through system (preferred configuration A)
Safety
Delay at the ramp

By dropping the outer lane the driver has some flexibility due to the availability of the shoulder as an escape. The inner lane merge gives the driver no alternative. The non-compulsory merge forces the driver to make a decision that could be made for him by designating which lane will drop.

LANE DROPS AND LANE BALANCE

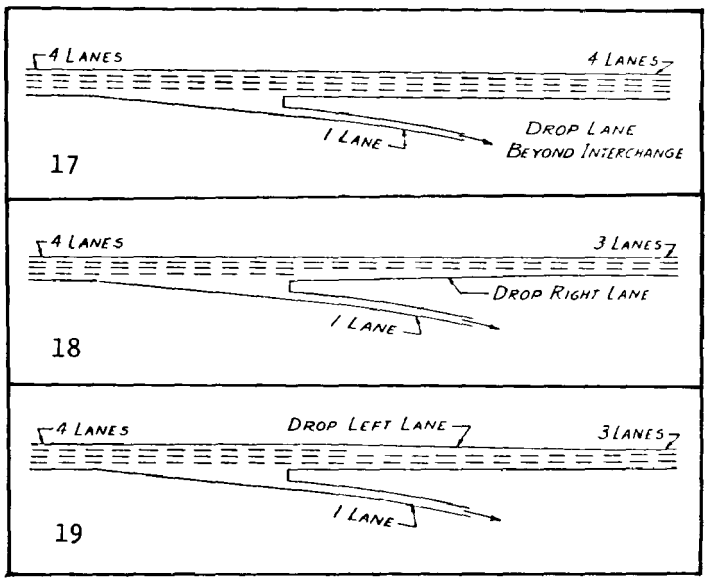
1. In the pre-workshop questionnaire, Figure 14 was asserted to be the configuration most often used by the organizations and agencies of the respondents to reduce the number of mainline lanes from three to two. Please rank these three configurations in the order you personally prefer to employ them.



The Number of Experts Assigning the Specified Rank

	Relative Ranking	Figure 13	Figure 14	Figure 15
Design	1	11	6	0
	2	5	10	1
	3	0	1	16
Operations	1	2	3	0
	2	3	2	0
	3	0	0	5
Research	1	2	3	0
	2	3	2	0
	3	0	0	5

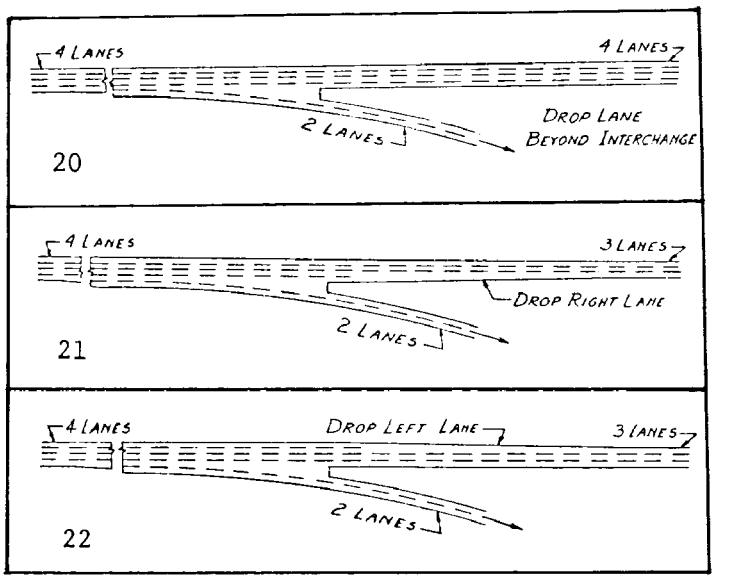
2. In the pre-workshop questionnaire, Figure 18 was asserted to be the configuration most often used by the organization and agencies of the respondents to reduce the number of mainline lanes from four to three following a one-lane exit. Please rank these three configurations in the order you personally prefer to employ them.



The Number of Experts Assigning the Specified Rank

	Relative Ranking	Figure 17	Figure 18	Figure 19
Design	1	12	6	0
	2	5	10	1
	3	0	1	16
Operations	1	2	3	0
	2	3	2	0
	3	0	0	5
Research	1	3	2	0
	2	2	3	0
	3	0	0	5

3. In the pre-workshop questionnaire, Figure 21 was asserted to be the configuration most often used by the organizations and agencies of the respondents to reduce the number of mainline lanes from four to three following a two-lane exit. Please rank these three configurations in the order that you personally prefer to employ them.



The Number of Experts Assigning the Specified Rank

	Relative Ranking	Figure 20	Figure 21	Figure 22
Design	1	10	8	0
	2	7	5	1
	3	0	4	16
Operations	1	3	2	0
	2	2	3	0
	3	0	0	5
Research	1	2	3	0
	2	3	2	0
	3	0	0	5

4. In conjunction with exits requiring two lanes, the number of lanes on the mainline beyond the ramp should be reduced by one. (Always or Never?)

	Always	Usually	Occasionally	Never
Design	2	10	4	1
Operations	0	4	1	0
Research	0	2	2	0
Total	2	16	7	1

5. The mainline traveled way should never be reduced by more than one traffic lane at a time. (Always or Never?)

	Always	Usually	Occasionally	Never
Design	7	8	0	1
Operations	2	3	0	0
Research	4	1	0	0
Total	13	12	0	1

6. When a lane is to be dropped in the vicinity of a major interchange, it should be carried beyond the interchange and then terminated beyond the influence of the interchange. (Always or Rarely?)

	Always	Usually	Occasionally	Never
Design	2	11	3	1
Operations	2	1	2	0
Research	2	0	2	0
Total	6	12	7	1

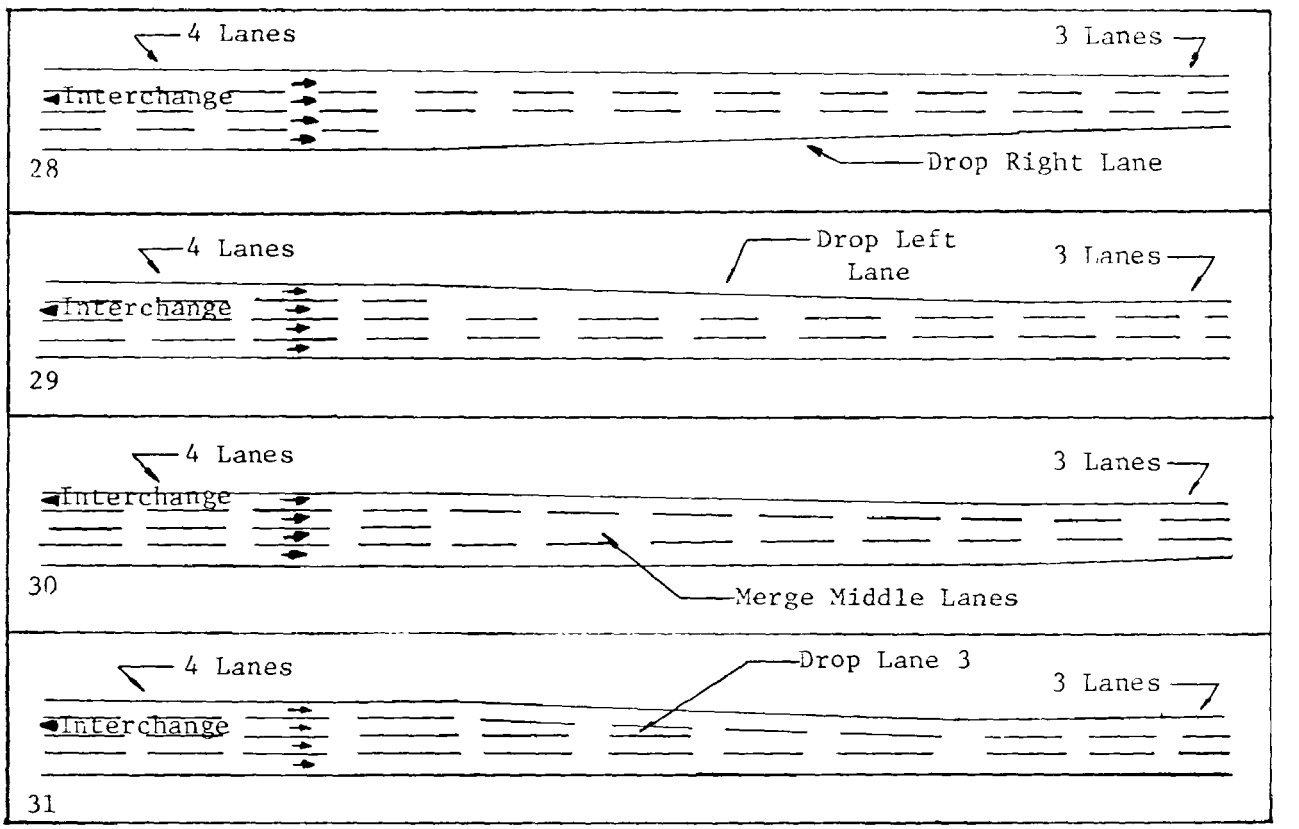
7. When a lane is dropped beyond an interchange, in general, it should be carried beyond the end of the entrance ramp acceleration lane for a distance of:

	0-500 ft.	500-1,000 ft.	1,000-2,000 ft.	2,000-3,000 ft.	3,000 ft. +
Design	0	2	7	6	0
Operations	0	0	3	0	2
Research	1	1	1	0	1
Total	1	3	11	6	3

8. Ideally, lane drops should be located at major diverging forks. (Agree or Disagree?)

	Strongly Agree	Agree	Disagree	Strongly Disagree
Design	2	9	4	1
Operations	0	3	0	0
Research	0	2	1	2
Total	2	14	5	3

9. Assuming that for appropriate reasons a lane must be dropped beyond an interchange, please rank these four configurations in the order that you personally prefer to employ them.

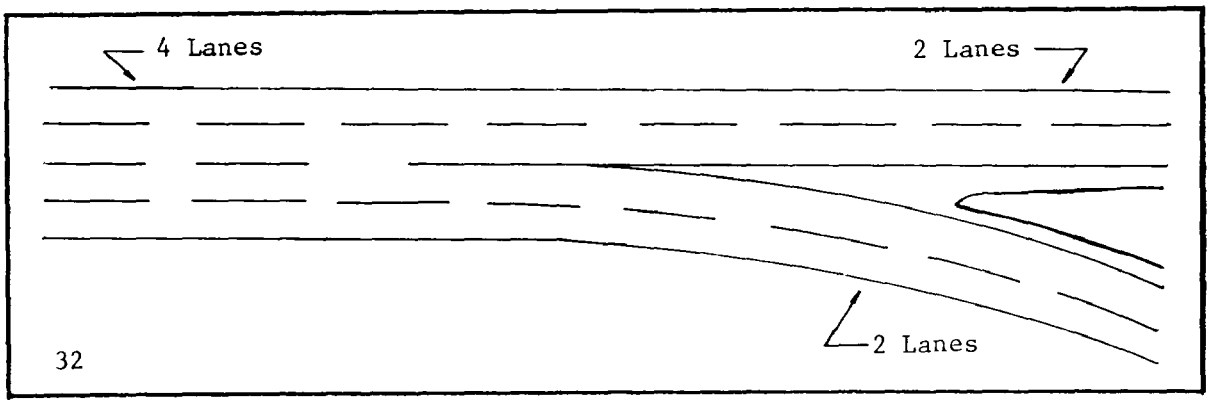


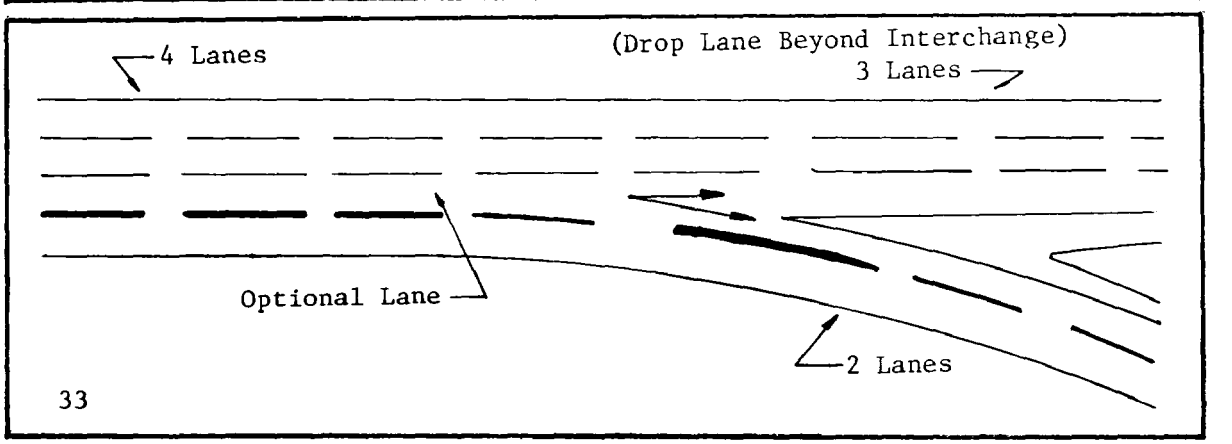
The Number of Experts Assigning the Specified Rank

	Rank	Figure 28	Figure 29	Figure 30	Figure 31
Design	1	11	0	0	0
	2	0	10	1	0
	3	0	0	9	2
	4	0	1	1	9
Operation	1	3	0	0	0
	2	0	3	0	0
	3	0	0	3	0
	4	0	0	0	3
Research	1	4	0	0	0
	2	0	3	0	0
	3	0	1	3	2
	4	0	0	1	2

Note: These responses were given by the participants of the second workshop only.

10. Assuming that at a major fork two lanes are to be dropped, please rank these two configurations in the order that you personally prefer to employ them.





The Number of Experts Assigning the Specified Rank

	Rank	Figure 32	Figure 33
Design	1	6	4
	2	4	6
Operations	1	2	1
	2	1	2
Research	1	2	2
	2	2	2

11. In general, what taper rates do you feel are most appropriate for lane drop treatments beyond the interchange (assuming that there are valid reasons for dropping the lane beyond the interchange)?

60 MPH SPEED

	Minimum Rate ?: 1						Desirable Rate ?: 1						
	25	35	40	50	60	70	40	50	70	75	80	90	100
Design	0	1	3	4	1	2	0	4	2	1	2	1	1
Operations	0	0	1	0	0	0	0	1	0	0	0	0	0
Research	1	0	0	2	0	0	1	2	0	0	0	0	0
Total	1	1	4	6	1	2	1	7	2	1	2	1	1

70 MPH SPEED

	Minimum Rate ?: 1				Desirable Rate ?: 1						
	35	50	70	80	50	55	60	70	75	80	100
Design	1	5	3	1	2	0	1	2	0	1	4
Operations	0	1	0	0	0	0	0	0	1	0	0
Research	1	2	0	0	2	1	0	0	0	0	0
Total	2	8	3	1	4	1	1	2	1	1	4

Note: These responses were given by the participants of the second workshop only.

12. If overriding width restrictions make it necessary to drop a lane just past an exit terminal, what design configuration do you feel should be used?

a. A taper beginning at the exit gore nose (specify taper rate, __: 1)
b. A full width recovery lane followed by a taper (specify length and rate)
c. There are no restrictions or conditions which justify dropping a lane just past an exit terminal.

a.

	Taper Rate (?:1)					
	30	35	50	55	70	100
Design	1	1	3	0	1	1
Operations	0	0	0	0	0	0
Research	0	0	1	1	0	0
Subtotal	1	1	4	1	1	1
Total			9			

	b.									c.
	Length (ft.)					Taper Rate				
	150	360	650	800	1,000	25	30	50	70	
Design	1	0	0	1	2	0	0	3	1	0
Operations	0	0	0	0	1	0	0	0	0	0
Research	0	1	1	0	0	1	1	1	0	0
Subtotal	1	1	1	1	3	1	1	4	1	0
Total					7					0

Note: These responses were given by the participants of the second workshop only.

ROUTE CONTINUITY

Workshop #1 ONLY

1. Which variable should control in the decision to make one movement an exiting movement and the other a through movement?

   a. The numbered route should always be designed as the through road regardless of turning volume.

   b. The relative volumes of movements determine which is through and which exits. If the route changing volume is substantially greater than the continuing route traffic, the changing route volume should be designated the through movement.

	a.	b.	Total
Design	3	6	9
Operations	1	1	2
Research	0	1	1
Total	4	8	12

2. If you checked (b) of question 1, please indicate below the relative volume which must occur before the (b) alternative is used.

$$\text{Ratio} = \frac{\text{Continuing Route Volume}}{\text{Changing Route Volume}}$$

a. ___ 1.00 > Ratio ≥ .90

b. ___ .90 > Ratio ≥ .80

c. ___ .80 > Ratio ≥ .70

d. ___ .70 > Ratio ≥ .60

e. ___ .60 > Ratio ≥ .50

f. ___ .50 > Ratio ≥ .40

g. ___ .40 > Ratio ≥ .30

h. ___ .30 > Ratio ≥ .20

i. ___ .20 > Ratio ≥ .10

j. ___ .10 > Ratio 0

	a.	b.	c.	d.	e.	f.	g.	h.	i.	j.	Total
Design	0	0	0	1	0	2	2	1	0	0	6
Operations	0	0	0	0	0	1	0	0	0	0	1
Research	0	0	0	0	0	0	0	1	0	0	1
Total	0	0	0	1	0	3	2	2	0	0	8

3. Should all weaving be confined to collector-distributor roads on major interchanges?

	Yes	No	Total
Design	7	2	9
Operations	2	0	2
Research	0	0	0
Total	9	2	11

Workshop #2 ONLY

1. Which variable should control in the decision to make one movement an exiting movement and the other a through movement in an urban area with high volumes?

   a. The numbered route should always be designed as the through road regardless of turning volume.

   b. The relative volumes of movements determine which is through and which exits. If the route changing volume is substantially greater than the continuing route traffic, the changing route volume should be designated the through movement.

	a.	b.	Total
Design	2	7	9
Operations	2	1	3
Research	3	2	5
Total	7	10	17

2. If you checked (b) of question 1, please indicate below the relative volume which must occur before the (b) alternative is used. Percentages refer to total volume approaching a split. (Note that it is still on urban area.)

   a. 90% continue, 10% change route
   b. 80% continue, 20% change route
   c. 70% continue, 30% change route
   d. 60% continue, 40% change route
   e. 50% continue, 50% change route
   f. 40% continue, 60% change route
   g. 30% continue, 70% change route
   h. 20% continue, 80% change route
   i. 10% continue, 90% change route

	a.	b.	c.	d.	e.	f.	g.	h.	i.	Total
Design	0	0	3	0	0	0	1	1	0	5
Operations	0	0	0	0	0	0	1	0	0	1
Research	0	0	0	0	0	1	1	0	0	2
Total	0	0	3	0	0	1	3	1	0	8

3. It appears that for rural major interchanges where volumes are not high it is preferable to take the continuing-on-the-same-route traffic through and the changing route traffic off on a connection. Do you agree?

	Yes	No	Total
Design	8	0	8
Operations	3	0	3
Research	5	0	5
Total	16	0	16

4. At what total approach volume would you consider discarding the route continuity approach and consider letting the volume splits determine the through road?

ADT Approaching	
ADT	Number Responding
15,000	1
30,000	1
20,000 - 25,000	1
Total	3

5. If a weaving section within a major interchange is unavoidable because two adjacent loop ramps had to be used in the design, should a collector-distributor road be used?

	Yes	No	Total
Design	8	1	9
Operations	2	1	3
Research	5	0	5
Total	15	2	17

6. In urban areas where ramps are often closely spaced is it often difficult to get adequate weaving distances between the major interchange and the nearest upstream or downstream ramp?

	Yes	No	Total
Design	9	0	9
Operations	5	0	5
Research	2	0	2
Total	16	0	16

7. In your experience has the problem of weaving between interchanges as described above ever resulted in design changes which produced undesirable operations at a major interchange?

	Yes	No	Total
Design	4	3	7
Operations	1	0	1
Research	2	0	2
Total	7	3	10

8. In a rural area with practically no land restrictions and very low turning movements would a design utilizing loop ramps which would produce a weaving section on a collector-distributor road be a feasible alternative?

   a. Yes
   b. Probably not, but under special circumstances I would consider it
   c. I would never consider loops and weaving sections in a major I/C

	a.	b.	c.	Total
Design	8	1	0	9
Operations	1	1	1	3
Research	4	0	1	5
Total	13	2	2	17

9. If you circled (b) above please describe the circumstances which would have affected you.

Comments:

1. Weaving on a tight vertical and/or horizontal alignment.
2. Sometimes, even in a rural area, existing routes requiring an interchange are spaced closely. Such proximity might cause you to place both interchanges on a C-D road. This case is rare with freeway-to-freeway, but not so with local access interchanges.

The kind of traffic; e.g., commuter and adjacent designs.

APPENDIX E

DECISION THEORY APPROACH TO INTERCHANGE DESIGN

John C. Hayward, The Pennsylvania State University

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APPENDIX E: DECISION THEORY APPROACH TO INTERCHANGE DESIGN

Introduction to Bayesian Decision Theory

One criticism of the interchange design process is the apparent lack of an acceptable evaluation method for choosing among alternative designs. This void in technique is disconcerting for two reasons. First, the possibility exists that a wrong selection will be made and interchange which does not operate properly, or which has a disastrous effect on the area around it, will be constructed. The second reason for concern is that the highway designer has no effective means for communicating the logic behind his decision to both his superiors and to the public.

This appendix describes one approach for evaluating design alternatives which should reduce the possibility of a wrong decision and enable the decision maker to better explain his reasoning for the final choice. The approach draws heavily on Bayesian management decision theory developed largely at the Harvard Business School and recently applied to business decisions by many large corporations.

Decisions Under Uncertainty

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, ultimately, 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 designer makes his decision on the basis of incomplete and largely uncertain information.

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. To ignore uncertainty would mean that the decision would be based only on calculatable effects, not on immeasurable ones. A decision based purely on a benefit/cost ratio calculated using only user time savings and construction and maintenance costs is one example of ignoring immeasurable (but no less real) factors. The decision theory approach seeks to identify all factors which have relevance to the decision and to explicitly judge what effect each factor will have on each alternative. The assumption is that by explicitly stating factors and values, the decision maker can approach the problem in a more systematic fashion, leading to a better understanding of the problem and, therefore, increased confidence that the resulting decision is correct. A decision theory approach seeks not to replace judgment, only refine it.

Two Concepts of Probability

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. This classical statistical view of probability can be illustrated by the example of red and green balls contained in an urn. If we know that there are 10 red balls and 20 green ones, we consider that the "probability" of randomly drawing a red one is 1/3. This mathematical probability can be computed when the quantities or frequencies of different events are known.

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. An example of this type of probability can be illustrated by asking a person what he feels the chances are that it is going to rain tomorrow. If he says that he feels that there is a 35% chance of rain, then he has given a subjective probability of .35 that it will rain. A Bayesian statistical approach will admit this kind of subjective information into a subsequent analysis or will, in effect, place a value on the decision-maker's judgment.

Obviously, there are some problems which are better analyzed using mathematical uncertainty, other problems which require judgmental probabilities and still others which need both kinds of analysis. A laboratory experiment where all factors can be adequately controlled can best be analyzed by measuring effects and then making inferences based strictly upon one's observations. In the uncontrollable "laboratory" of the highway system, we must often resort to subjective approaches as the commonly used "diagnostic study team" evaluation method illustrates. The point is that until mathematical probabilities can be accurately measured and replicated, the decision maker must make use of subjective probabilities or, as more commonly referred to by designers, engineering judgment.

Payoff Matrices

The basic tool of a decision theory analysis is known as a payoff matrix or payoff table. The matrix is two dimensional, with one side being described by the set of alternative choices available and the remaining side being described by various "state of nature" or uncertain future events, only one of which will occur.

A simple example is the rain example again. The set of alternative actions could be: (1) to carry an umbrella, or (2) not to carry an umbrella. Two states of nature may be: (1) it rains, or (2) it does not rain. The payoff matrix (without payoff entries) is shown below as Figure E-1.

		ALTERNATIVE ACTIONS	
		(1) Carry Umbrella	(2) Do Not Carry Umbrella
STATES OF NATURE	(1) Rain		
	(2) No Rain		

Figure E-1. Example Payoff Matrix

One necessary stipulation in formulating a decision in this manner is that the states of nature must be independent of the alternative actions or, in our case, that carrying an umbrella will not cause it to rain.

The cells of the matrix are filled in with payoffs of each combination of action and state of nature. The unit of payoff most readily brought to mind is the dollar, and although not always the most appropriate, we will use it here for simplicity. Suppose that if we carry

the umbrella and it rains that we will not be paid anything (but we don't have to pay out anything either), so we value the "payoff" at $0. On the other hand, if we don't carry the umbrella and it rains, our suit may be ruined at a cost of $100 or a payoff of -$100. Carrying an umbrella when there is no rain has some cost associated with the inconvenience -- assumed here to be $2.50. That is, the payoff is -$2.50. Finally, if we choose not to carry an umbrella and it doesn't rain our cost and our gain are zero. The filled payoff matrix is shown in Figure E-2.

		ALTERNATIVE ACTIONS	
		(1) Carry Umbrella	(2) Do Not Carry Umbrella
STATES OF NATURE	(1) Rain	$0	-$100.00
	(2) No Rain	-$2.50	$0

Figure E-2. Filled Payoff Matrix

The final inputs required to complete this simple example are called prior probabilities[1] and are used to represent the uncertainty

[1]The term "prior" is consistent with the literature but its meaning may not be clear to the reader. These "prior probabilities" are often modified through experimental results before the analysis is completed, hence the qualifier, prior.

regarding the states of nature. These are subjective probabilities and may be obtained from expert judgment (the weather forecast) or from a novice impression (looking at the sky in the morning). Assume that in the example, the probability of rain is estimated to be 0.20 and that the probability of no rain is 0.80. The restriction on setting prior probabilities is that they must sum to unity, but this is usually handled by careful definition of the states of nature so as to include all possible states. In our example, one would not choose as states (1) rain and (2) sunshine, since there are many other possibilities. Rain and no rain cover all possibilities, however.

The decision rule generally followed is to choose the alternative which maximizes <u>expected</u> payoff. One essentially computes an average payoff, weighted by the prior probabilities, for each alternative and selects the action which gives the highest expected value.

The computation for our example is simple but illustrates the concept. Considering alternate (1), carrying the umbrella:

Expected Payoff (carry umbrella) = Prob (Rain) x Payoff (carry umbrella, rain)

+ Prob (No Rain) x Payoff (carry umbrella, no rain)

Expected Payoff (carry umbrella) = .2(\$0) + .8(-\$2.50) = -\$2.00

Expected Payoff (don't carry) = .2(-\$100) + .8(0) = -\$20.00

Applying the decision rule which says to act so as to maximize payoff, we would choose to carry the umbrella -- thereby selecting the \$2.00 expected loss (-\$2.00 expected payoff) over the \$20.00 expected loss.

Describing the payoff matrix mathematically, let the set of n alternative actions be represented by a_i, and the set of m states of

nature be represented by θ_j. Prior probabilities are given as $P(\theta_j)$ and payoffs as $R(a_i, \theta_j)$. The payoff matrix reduces to

	a_1	a_2	. . .	a_n
θ_1	$R(a_1, \theta_1)$	$R(a_2, \theta_1)$	. . .	$R(a_n, \theta_1)$
θ_2	$R(a_1, \theta_2)$			•
θ_3	.			•
⋮	⋮			
θ_m	$R(a_1, \theta_m)$	• • •		$R(a_n, \theta_m)$.

The expected payoff for each alternative is computed by the formula:

$$ER(a_i) = \sum_{j=1}^{m} P(\theta_j) \; R(a_i \; \theta_j),$$

where $ER(a_i)$ = expected payoff for alternate i. The decision rule to maximize expected payoff would be:

Choose a_z so that $ER(a_z)$ = Maximum $[ER(a_1), ER(a_2), \ldots ER(a_n)]$.

Utility as a Payoff

In the simple example of the decision whether to carry an umbrella or not, the unit of payoff was assumed to be money. In most complex decisions dollar costs and benefits are not the most appropriate units to use in the payoff matrices for two reasons. First, many evaluative categories or attributes of a particular alternative state-of-nature cell cannot adequately be expressed in dollar terms. An example might be the choice of sending one's son or daughter to one of three different colleges. Each cell has an entire set of payoff attributes, all of

which must be considered in the evaluation. Some are quantifiable in dollar terms, such as the costs of tuition and room and board, but many attributes such as quality of instruction, exposure to undesirable elements, or stress on athletic programs, are not amenable to expression in relative dollar amounts. This means that the final payoff must be expressed in something other than money and the decision criteria must seek to maximize this other unit.

The second reason why money is a poor indicator of the value or worth of an alternative choice is that the value one places on money is not linear. This may be illustrated by considering a betting situation where one chooses to bet on a football game. Suppose you are given the opportunity to bet one dollar, to bet $1,000, or not to bet at all. If you think that team A will win with a probability of .60, then your expected dollar payoffs will be:

Action	Expected Payoff
Bet $1	$ .20
Bet $1,000	200.00
Don't Bet	0.00

If you act to maximize expected dollar payoffs you will choose to bet $1,000 on team A to get a $200 expected payoff.[2]

Many people would choose not to take the $1,000 bet unless they were relatively unconcerned with the 40% chance of losing $1,000. Therefore, the assumption is that they must be acting not to maximize expected dollars, but rather to maximize the utility or worth of money.

[2]The payoff is equal to .6($1,000) + .4(-$1,000) = $200.

Utility can be defined as a unitless measure of the relative worth or value of different actions under a given state of nature. If a person finds an orange more appealing than an apple, the utility of an orange is greater to him than that of an apple.

In general, quantities of each payoff affect the utility value given to the attribute in a nonlinear fashion. The example of money can be given again.

Suppose one were to measure a person's utility for incremental amounts of money from zero to a million dollars. The graph of utility versus money, called a utility function for money, may look like Figure E-3. The endpoints are simple to fix; a person has the most utility or assigns greatest worth (1.0) to obtaining $1 million and least (0.0) to obtaining zero dollars. The curve goes up more rapidly at first than it does later on. There is more difference in utility between gaining half a million and gaining zero, than there is between half a million and a million. This general shape of curve represents the economist's notion of diminishing marginal utility for money, although it may vary in the deceleration constant. It will probably be flatter for millionaires.

This notion of utility or worth of some measured quantity gives the decision maker a normalizing scale for comparing unlike quantities. For an action which has more than one kind of payoff (e.g., safety improvement which will affect accident rate and severity) it allows one to reduce the payoffs to a common scale. Utility alone does not give trade-off information about the worth of a fatality versus the worth of an injury accident, but only the worth of a number of fatalities versus another number of fatalities. Trade-off can be incorporated into the evaluative methodology later.

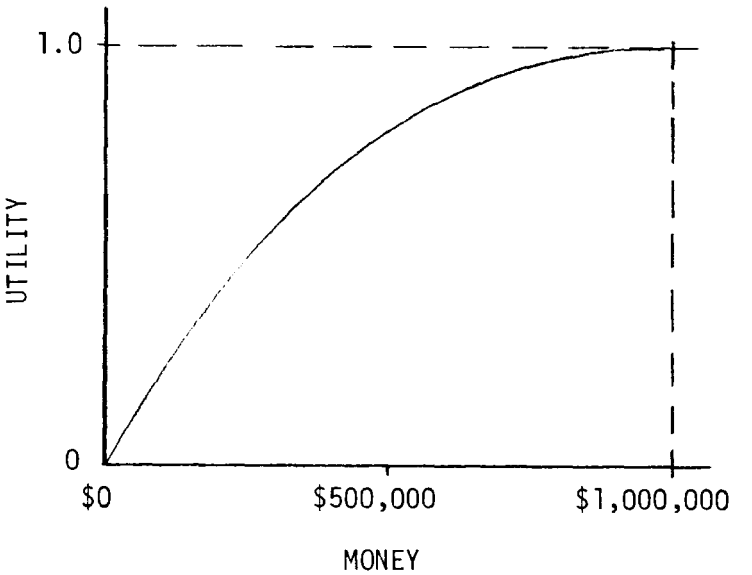

Figure E-3. Sample Utility Function for Money

Utility also gives the decision maker a tool for assessing or ranking those immeasurable categories which must often be considered in the analysis. That is, those categories which have no measuring scale, such as neighborhood disruption or visual impact on the non-user, can be directly estimated as point values on a utility scale. In effect, this amounts to a rating of certain variables for each alternative on a utility rating scale.

This concept gives one rational approach to the twofold difficulty of incorporating the two types of immeasurable evaluative categories discussed in Chapter Two. First, it coverts all evaluation parameters onto a common scale so we may compare dollars to decibels, for example, and second, it allows us to directly assess the value of those variables without universal scales of reference. Ultimately, we arrive at a measure common to dollars, decibels, and visual impact, and any other evaluative category one may think of.

The utility concept further implies that one makes his decision using a criterion of maximizing utility rather than payoff units. This logic has been substantiated through experimental devices and can be supported intuitively by considering various kinds of betting situations such as the football game example.

Assessing Prior Probability and Utility Functions

Two major inputs to the decision theory framework described above are prior probabilities and utility functions. It is tempting to ignore discussion of the serious difficulties encountered in assessing probabilities and utility. In fact, many of the researchers in the field have preferred to dwell on the more intricate mathematical nuances of Bayesian statistics and have assumed that the inputs, probabilities and utilities, would come easily. This step in the analysis is perhaps the most crucial to obtaining good results, however, and should not be passed over lightly in the development of an implementable procedure.

Prior Probability Assessment

Several approaches can be taken in assessing an individual's prior probabilities, but the best one seems to depend upon the decision maker's personal background. The analyst (the individual who is trying either to obtain the values so that he may make the analysis himself to teach the decision maker to do it) must work closely with the decision maker, explaining the concepts, leading discussions to determine prior probabilities, constructing a distribution, and then obtaining the decision maker's approval. The process is an iterative one, with the prior distribution being refined until the decision maker is satisfied that the distribution reflects his best judgment.

Five assessment methods which may be used to obtain a decision maker's probabilities on states of nature are discussed briefly on the following pages.

(1) Direct assignment

This method is the easiest for those decision makers who are experienced in the use of decision theory or who have enough statistical background to fully understand probabilities. If the states of nature are discrete and defined fully, the analyst simply asks for the probability of each occurring.

(2) Betting odds

Other decision makers may feel more comfortable expressing their prior probabilities as betting odds, particularly when there are only two possible states of nature (A and B). If the decision maker indicates that the odds are 3 to 1 that A will happen instead of B, he is assigning a 0.75 probability to A and a 0.25 probability to B.

(3) Lottery methods

This approach involves asking the decision maker to choose between two lotteries which are constructed so as to assess his prior probability. For example:

Lottery A. You win \$25 with probability 0.35,
or you win \$0 with probability 0.65.

Lottery B. You win \$25 if state of nature θ_1 occurs,
or you win \$0 if state of nature θ_1 does not occur.

If the decision maker (DM) chooses Lottery B, he must feel that the probability of state of nature θ_1 occurring is greater than 0.35. If the analyst then varies the probability of winning in Lottery A until

the decision maker is indifferent between A and B, the final probability of winning $25 in Lottery A is a measure to the DM's prior probability for state of nature θ_1.

(4) Probability density function

Many times states of nature are continuous rather than discrete. For example, consider that a set of "states of nature" is a traffic forecast and the decision deals with how much capacity to provide. The prior probability on the states of nature (individual values of traffic forecasts) can be expressed as a continuous probability density function similar to the one in Figure E-4. Decision makers may be able to draw the density function with help from the analyst if they are comfortable expressing uncertainty as probability distributions.

The probability of the actual value falling between two points is shown as the area under the curve between those two points on the abscissa. Therefore, in Figure E-4 the probability that the actual traffic T_a will be less than or equal to 14,000 can be represented by:

$$P(T_a \leq 14{,}000) = \int_{10{,}000}^{14{,}000} f(t)\, dt$$

or, in general:

$$P(T_a \leq X) = \int_{o}^{x} f(t)\, dt.$$

The restriction on f(t) is that the area under it sums to unity.

$$\int_{o}^{\infty} f(t)\, dt = 1$$

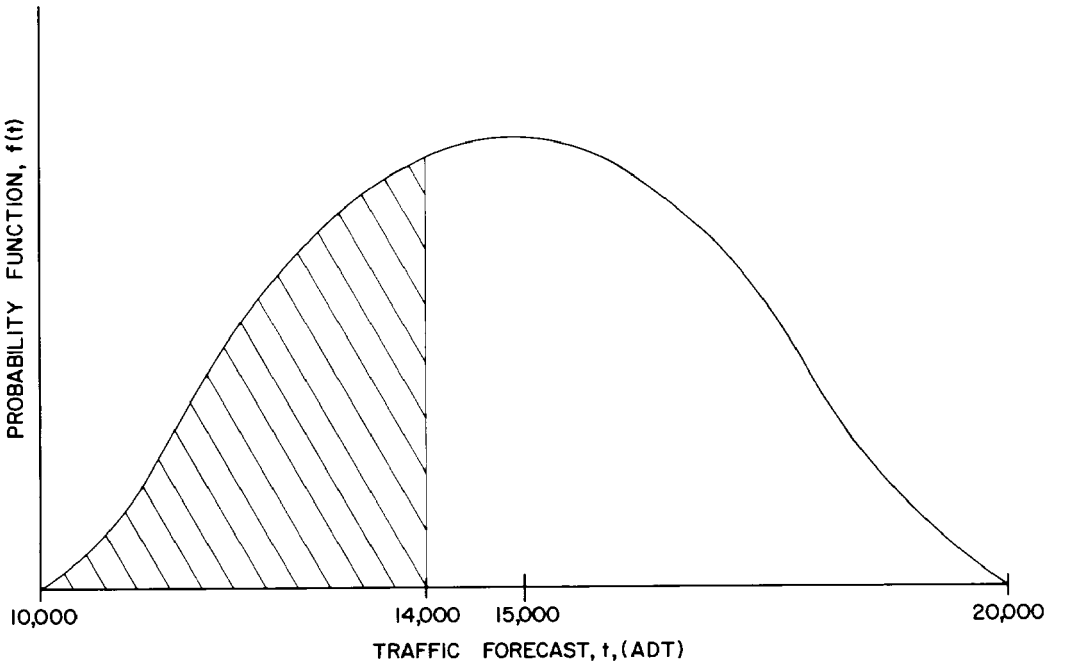

Figure E-4. Example Probability Density Function

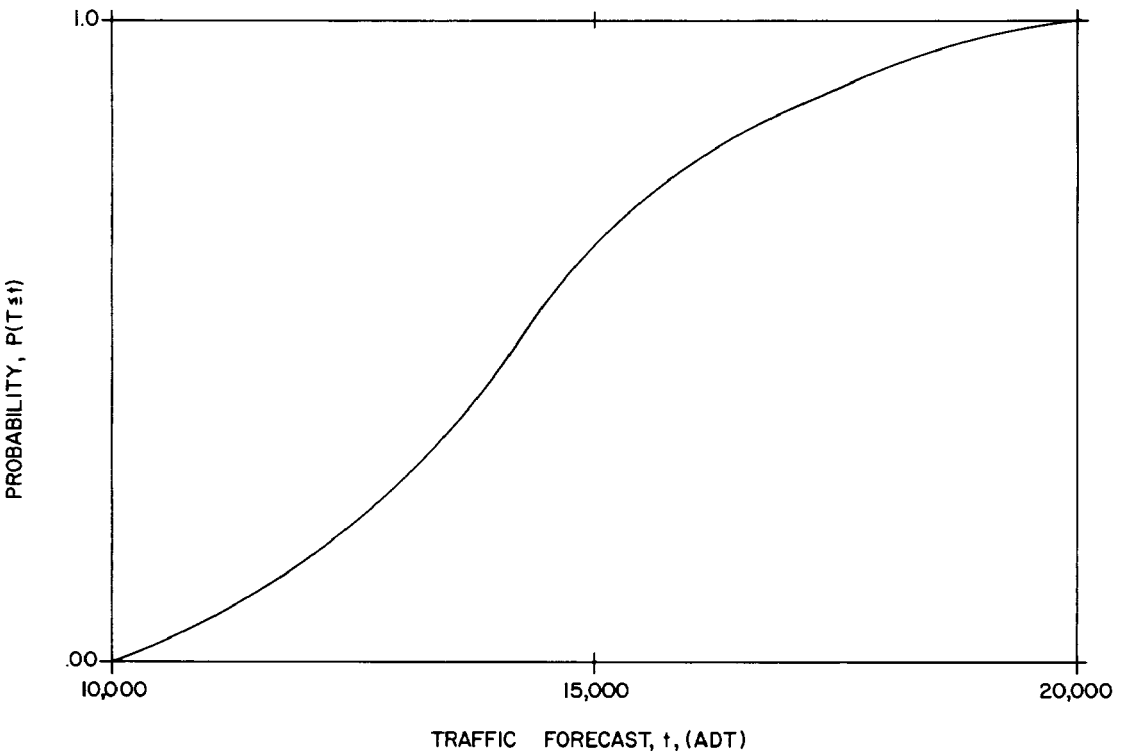

Figure E-5. Example Cumulative Distribution Function

(5) Cumulative distribution function

Another way to represent continuous probability is with the cumulative density function, or a function which represents the change in area under a probability density function as the variable of interest is increased. The traffic forecast prior probability is shown in Figure E-5 as a cumulative function. This gives the probability that the actual value, T_a, will be less than some value of t directly off the vertical axis.

Some decision makers can relate to this kind of continuous representation better than the probability density form, although both represent the same thing. The analyst must elicit the information from the DM by asking him how often he feels the actual value will fall below some specific value within the range of the distribution. The answer is an indication of the prior probability of something less than that value occurring.

Utility Function Assessment

First, it must be noted that we are generally interested in a utility function for an attribute measure bounded by some prescribed limits. Suppose we consider the utility of money, as represented by a function such as depicted in Figure E-3. How could one determine such a function?

To assess utility functions, the analyst most commonly employs a lottery scenario with the decision maker. For many attributes, like money, it is obvious which end of the scale is preferable over all other points and also which is least preferable. These form the starting points for the utility function construction by serving as the maximum and minimum values on the utility scale. In the case of money

between zero and $1 million where the utility scale is from 0 to 1, the zero amount is given the zero and the highest incremental amount the 1.

For the remaining points in the money example, the decision maker is offered his choice of two lotteries. He may take x dollars for certain (Lottery A), or enter a lottery where he can win $1 million with probability P or win zero with probability (1-P). The P value where he is indifferent is the utility value for x dollars. In the actual interviewing process either x or P is varied with the other held constant until the DM says that both lotteries are equally appealing to him.

The logical proof of this method rests in the assumption that people behave so as to maximize expected utility. If the DM says he is indifferent between Lotteries A and B, he is indicating that the expected utilities of each are equal. Consider the following example of after tax gains: If a subject is indifferent between getting $500,000 with certainty and winning $1,000,000 with a probability of .8 (and winning zero with a probability of .2), his utility for $500,000 is .8 (relative to utility of $1,000,000 set at 1.0). In a proof form consider the two lotteries:

Lottery A: Get $500,000 for certain

Lottery B: Win $1,000,000 with probability .8 or win zero with probability .2

If indifferent: Expected Utility of A = Expected Utility of B

Expected Utility of B = .8 (Utility for $1,000,000) + .2 (Utility for zero)

= .8(1) + .2(0) = .8

∴ Expected Utility A = 0.8 = Utility for $500,000.

Several points on the curve can be determined in this fashion and the function drawn in.

For the utility functions which describe discrete and unlike objects the DM is first asked to rank the objects. The most preferred is given a utility of one and the least preferred is given a value of zero.

An example might be the utility function one has for several types of comparable automobiles; Chevrolet, Ford, Plymouth, Chrysler, or Mercury. He ranks them as to preference in the following manner.

Most Preferred	(1) Mercury
	(2) Chrysler
	(3) Plymouth
	(4) Chevrolet
Least Preferred	(5) Ford

The Mercury is assigned a one and the Ford a zero value of utility. He then asks himself where he is indifferent between a Chrysler for certain or a P chance on a Mercury with a (1-P) chance on a Ford. The value of P represents his utility for the Chrysler car.

Both the money example and the car example are simple ones to demonstrate how utility functions are arrived at. The method assumes that one can rank his preferences, at least as far as selecting one as the best and another as the poorest. The second premise is that a decision maker chooses to maximize his expected utility, not the expected value of the two lotteries.

Previous Applications

Will a decision theory approach improve the quality of complex decisions and is the improvement worth the trouble? It would be ideal to answer

such a question with a review of past successful applications of the methods outlined above, which would prove that wrong decision had been avoided and costly errors eliminated. Unfortunately, such studies are really impossible to perform since most complex decisions are one of a kind and after they are made and implemented one doesn't know for certain that the best choice was made. Every major interchange involves a unique decision or choice and after a configuration is chosen and built the designer-decision maker does not have the opportunity to try another design. Therefore, it is impossible to design an experiment which proves the value of the decision theory approach.

The measure of the value of a decision theory approach must come from the users of the methodology, not from comparisons of choices with and without the method. The justification for adoption can be made by outlining where it has been tried and what the people who tried it thought of it. Applications have been made primarily in the business fields, in military decisions, and the field of medicine. Methodologies have been proposed for use in transport planning decisions and a few for interchange design. The business applications will be discussed below, followed by a brief section on proposed transportation planning methodologies.

Business Applications

An interesting review of decision theory applications in business, written by R. V. Brown, appeared in the Harvard Business Review in

1970 (Brown, 1970). The author surveyed 20 companies in 1969 to determine what impact decision theory analysis has had on decision and what difficulties have been encountered in application. All of the companies were known to have used decision theory approaches.

He found that use of the technique is expanding rapidly due to initial "successful" application by a few pioneering companies (DuPont, Pillsbury, and General Electric) as well as increased production of decision theory-trained business school graduates, principally from the Harvard Business School. Brown discovered that adoption of the tool has caused little change in the decision-making process, but it has affected individual decisions.

Some problems are encountered in the application of decision theory techniques to business decisions. It is not applicable to all problems and the user must expect some disappointing experience at first. The company must have competent practitioners who can deal effectively with the executives that are doubtful of the value of the method. Often the logic and language of the procedure is new and uncomfortable for the decision maker. If the technique is pushed by the "front office," the lower level decision makers feel threatened and resist using it. If the analysis is performed by a staff group without much personal contact, the decision maker may feel that his power is being usurped and that he no longer has control over the decision. The cost of the procedure is a real consideration since many of the more sophisticated analyses require computer applications.

In spite of the problems, the trend in business analysis seems to be moving toward a more widespread use of the technique. This trend would lead one to the conclusion that the decision theory approach is sound and leads to good decisions.

Transportation Planning Methodologies

Several schemes using decision theory concepts have been proposed for use in transport planning evaluation. Though the majority of the work has been aimed at evaluation of alternative systems of a more macroscopic scale than interchange design,[3] a few have been directed specifically at evaluating alternative interchanges.[4] All of the methodologies follow basically the same logic.

The first step is to determine those attributes upon which the alternatives will be evaluated. This can be done either through a formal or informal hierarchical goal development process where the "super goal" of improving the quality of life is broken into increasingly finer subgoals until a list is arrived at which can be adequately represented by performance measures.

After the list of attributes has been agreed upon, performance measures or goal quantifiers must be established. For the goal of minimizing fatal accidents, for example, the performance measure might be the expected number of fatal accidents reduced from the "do-nothing" case for a particular alternative action.

Some attributes have no scale which adequately measures or quantifies the degree to which the goals are met. For these, an artificial scale is constructed which can be used to rank the alternatives.

Performance measures other than direct worth are transformed into utility or worth via some utility function, thereby reducing all evaluative attributes to the same scale. This enables the analyst to add the

[3] See (Miller, April 1969), (Institute for Analysis, Sept. 1971), and (Manheim, 1970).

[4] See (Alexander and Manheim, 1965) and (Leisck May 1972).

utility score based on some weighting scheme which is intended to express the relative importance of each attribute to the total. In some cases benefits and costs are considered separately and the resulting over-all worth measure is a ratio. In other examples the worth values of the costs are simply added to the worth of the benefits and the total worth becomes a summation of the weighted values.

Interchange Evaluation in Decision Theory Framework

Having introduced the decision theory concepts of evaluation and pointed out some previous applications of the technique to business and transportation planning problems, it is desirable to define interchange evaluation as a decision theory problem. Each of the three components, alternative actions, payoffs, and states of nature, will be discussed individually.

Alternative Actions

It is obvious that the actions in the decision theory framework correspond to alternative interchange designs. At one decision level one may consider an entire interchange configuration as an alternative, but at a finer level alternate actions might be restricted to different designs for one approach leg of an interchange. The actions available in the latter case may be as specific as the alternatives of placing an exit ramp on the left or on the right side of the roadway, while configuration actions may compare a cloverleaf to a direct connection configuration. The number of actions which may be considered are bounded only by the costs involved in working up the design to a level where decent predictions of performance measures may be made.

Payoffs

The "payoff" of a particular interchange is a multi-dimensional one requiring a large set of units to describe adequately. P.P.M. 20-8 requires that the interchange be evaluated over 23 separate categories or attributes. Some of these have acceptable performance measures while others are qualitative attributes at best. Therefore, the interchange payoff in a decision theory framework is a set of unrelated, individual performance measure estimates.

This set approach or multi-attributed problem requires some normalizing approach which will reduce all evaluative categories to the same scale. At this point the utility concept comes into play, providing the needed tool for combining multiple attributes into a single unit system.

The utility concept only provides an indication of the worth of a particular quantity of some attribute in terms of more or less of that same attribute, however. To get the total utility or worth, (the single value), a weighting scheme is required to obtain a weighted summation of the individual attribute utilities.

The payoff value for the interchange problem is derived via a four-stage process. First, the attributes and their performance measures are fixed and are held constant for all alternatives. Second, estimates of each performance measure for each alternative action are given in the appropriate units. Third, the performance measure units are transformed to utility or worth values via a utility function unique to that attribute. Finally, the utility values for each performance measure are summed according to a predetermined weighting scheme which allows between-attribute comparisons. The result is a single worth value (or, as will

be shown later, a distribution of worth values) for each cell of the payoff matrix.

States of Nature

The states of nature in an interchange problem may not be as easily visualized as the alternative actions or the payoff values. The methodology presented below will consider different weighting schemes as the states of nature. By different weighting schemes it is meant that there are many different ways to combine the many performance-utility values in the payoff cell. That is, the feeling of relative importance between unlike attributes such as vehicular safety and non-user noise varies substantially from person to person. Ideally, we would like to use a weighting strategy which reflects the feelings of the entire society, but this kind of information is not currently available.

An alternative to complete public assessment is to devise several weighting schemes which may have support in different sections of the society. Each one of these schemes would comprise a state of nature and could be represented by a prior probability. The prior probability would be related to the percentage of society which feels that the particular weighting scheme best reflects their own opinion. For example, if the decision maker felt that 40% of the society that was to be affected by the interchange favored a weighting scheme which was safety oriented then the prior probability of that state of nature would be 0.4. Another way of explaining a prior probability would be that the decision maker felt, with a 40% certainty, that the state of nature was the "true" feeling of the soceity.

Summary of Interchange Problem Framework

Figure E-6 represents the interchange problem in the decision matrix form.

		DESIGN ALTERNATIVES			
		A	B	C	D
STATES OF NATURE	θ_1 Weighting Scheme 1	$R^*_{A,1}$	$R_{B,1}$	$R_{C,1}$	$R_{D,1}$
	θ_2 Weighting Scheme 2	$R_{A,2}$	$R_{B,2}$	$R_{C,2}$	$R_{D,2}$
	θ_3 Weighting Scheme 3	$R_{A,3}$	$R_{B,3}$	$R_{C,3}$	$R_{D,3}$
	θ_4 Weighting Scheme 4	$R_{A,4}$	$R_{B,4}$	$R_{C,4}$	$R_{D,4}$

*$R_{i,j}$ is the cumulative worth of alternative i combined under weighting scheme j

Figure E-6. Payoff Matrix for Interchange Problem

If each of the "m" states of nature, θ_j, has a prior probability of $P(\theta_j)$, then to maximize expected utility, the decision maker should choose the alternative, i, so that

$$\sum_{j=1}^{m} R_{i,j} \times P(\theta_j)$$

is the maximum value.

A Decision Theory Method for Interchange Evaluation

The following section will outline a step-by-step approach to the interchange evaluation problem using the payoff matrix concept formulated above.

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 increasingly 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.

One goal hierarchy example in a transportation context is given by Manheim and Hall (1968). Their ultimate goal is called "the good life" which is characterized at the next lowest level by convenience, safety, aesthetics and economic considerations. Each of these four subgoals are broken down further; for example, safety means decreasing fatalities, decreasing injuries, and decreasing property damage accidents on the highway. By subdividing the super goal of the good life the authors derive twenty measurable subgoals or evaluative attributes.

Structuring this type of hierarchy allows the decision maker to use different levels of goals for different decisions in the process. The level of detail upon which final design decisions should be based, for example, would be a much more explicit goal level than corridor choice decisions. The decision maker may consider "safety" in evaluating alternative corridors, while considering property damage, injury and fatal accidents separately for a final design decision.

The goal hierarchy exercise also eliminates using an attribute list which uses different levels of the same goal as separate attributes. For example, the three different types of accidents are subsets of the goal, "safety." All four, safety, fatal, injury, PDO accidents, should not be considered simultaneously in the evaluation; rather the attribute should be either safety or the set of three types of accidents. (Level confusion is apparent in the list presented in PPM 20-8.)

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 attendant 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. The goal of safety cannot be quantified directly without breaking it down further into different types of safety. In both these cases the performance measure may have to be a direct worth estimate of the value of the alternative rather than a physical measure. By using direct worth estiamtes the need for transforming the physical measures into worth or utility measures is eliminated.

Figure E-7 presents an example of the goal hierarchy concept and the matching performance measure notion. It is not intended to be a recommended format for all projects, but is given only to illustrate

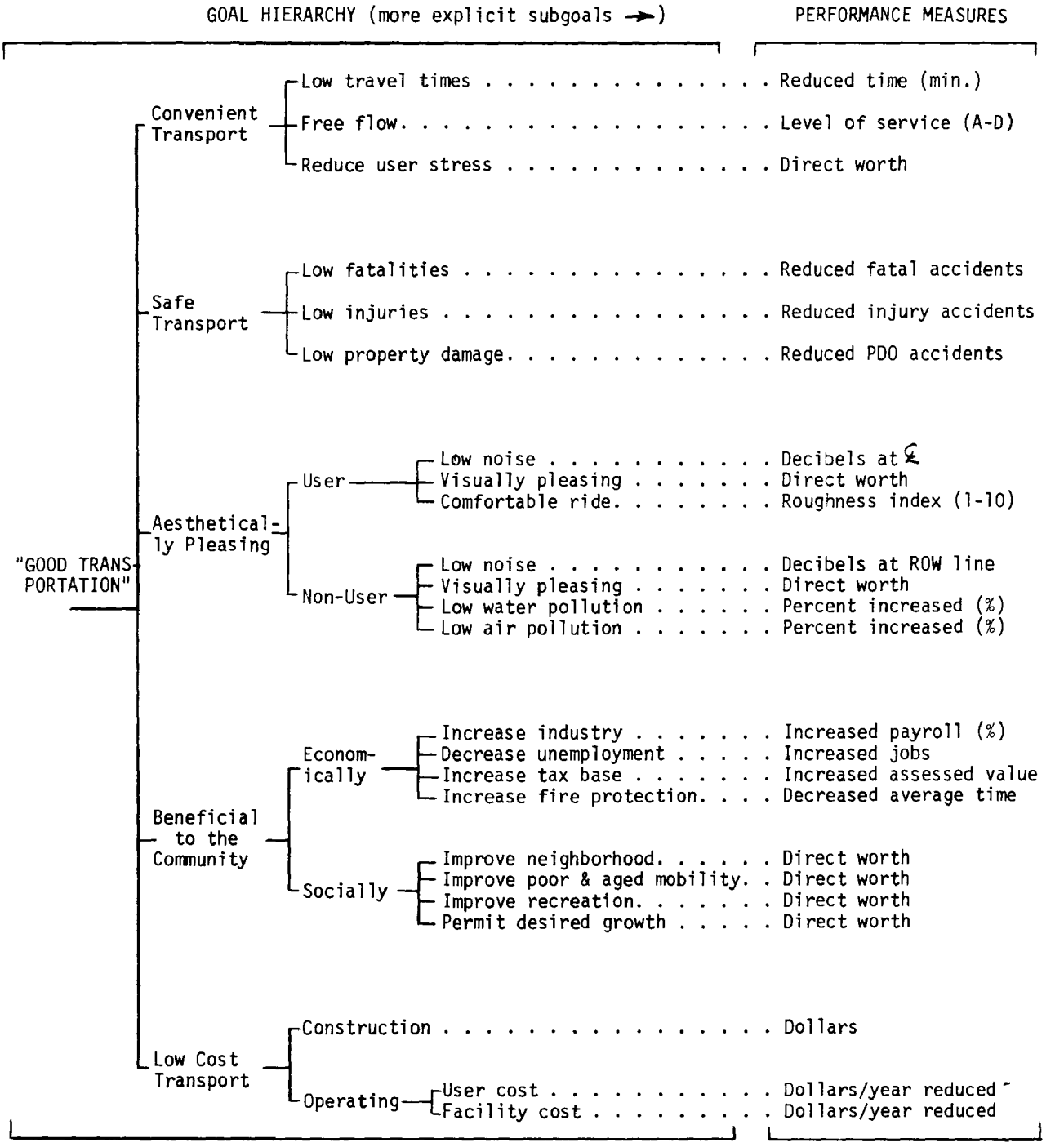

Figure E-7. Example Goal Structure and Performance Measures

the output of a goal hierarchical structure and performance measure procedure. This example will be continued through the remainder of the Appendix E discussion.

Step 3: Generate Alternatives

Major interchange design is essentially a search and selection procedure where the generation of alternative designs constituting the search and the evaluation procedure is used to select the best of the alternatives. In such a process the concept of optimal design becomes meaningless since the design procedure will lead to an optimal facility only by chance. That is, the alternative set must include the optimal design before any evaluative technique can select it. Therefore, to increase the probability of hitting upon the optimal design, the engineer may either increase his alternative set size or be more selective in his choice of alternative designs. Because increased numbers of alternatives mean added design costs and, according to the workshop experts, would increase the chances of optimal selection only slightly, the second approach is more desirable.

Alternative designs should be generated to portray the wide range of goals which appear in the goal structure. Each alternative design might be directed primarily toward achieving one of the higher level goals with secondary consideration of the remaining goals. For example, one alternative might be designed with safety as the ultimate goal, another stressing low costs and still another may be intended to provide ultimate user convenience.

A second consideration in generating alternative solutions is that they satisfy minimum performance measure standards. This feasibility constraint insures that all alternatives which are to be evaluated

are in conformance with the policies and guidelines presented in the design manuals of the particular state. Such constraints might be in the form of a ceiling cost on construction or maintenance, minimum level of service requirements, or provision of adequate stopping sight distance.

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. Often the decision maker, himself, is not in the best position to make predictions in all the evaluative categories, and he will assign the scoring task for each attribute to a person or group expert in the area. For instance, a traffic safety specialist might be called upon to predict accident reduction over the null condition or the construction section may supply construction cost estimates. The decision maker has the responsibility of finding the best expert judgment possible within the time and cost constraints of the decision.

Certain tools are available for making rational estimates of future performance. The tools 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. Each performance measure is best predicted with different devices, and it is the duty of the expert judge to apply the proper tool.

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, the tool mentioned above, 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 widths or ranges of these distributions increase with an increase in expert uncertainty. Construction costs may be predicted with little uncertainty due to the existence of good historical data, but accident predictions at a particular interchange may vary greatly.

Step 5: Obtain Worth Transformation Functions

Each alternative has now been rated on a set of performance measures either through a point estimate or a distribution of values. In order to combine these measures into a single over-all measure for the entire facility, the units must be transformed to worth or utility. Since the ranges of the performance measurements have been specified, the utility or worth functions can be assessed through the methods described earlier. Only one transformation function will exist for each performance measure, and its range will include the combined range of all the alternative designs.

Those performance measures that were estimated directly in a worth scale, of course, do not require transformation functions. However, the advantages of estimating performance measures and then transforming to worth units over direct worth estimation are obvious both from an operational standpoint and in the future defense of the decision.

Step 6: Generate a Number of Weighting Schemes

Different weighting schemes should be devised which reflect the diversity of opinion throughout the affected community. Examples of

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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.

One approach to devising the different weighting strategies might be to set one scheme which favors each of the second level goals in the hierarchy. Taking the goal structure in Figure E-7 as an example, this would mean that five separate schemes could be established. The weights across each level of the hierarchy should sum to unity, enabling individual weights to be computed by multiplying out along the branch.

To generate alternative weighting schemes one might consider different weights on the second-level goals only, leaving the lower-level weights constant, but effectively changing the final individual weights through multiplication.

Figure E-8 illustrates this method for using hierarchical structure to generate different weighting schemes. The numbers separated by slashes are three alternate 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

The three weighting systems illustrated in Figure E-8 can be more easily understood by considering the different weights on the five second-level goals rather than by trying to look only at the 24 lowest-level weights. Table E-1 gives the three weighting strategies in summary form. The first, Scheme A, gives more weight to user convenience and only slightly less to safety. Scheme B is primarily safety and

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 E-8. Sample Weighting Procedure

aesthetically-oriented, while Scheme C is intended to give most consideration to benefiting the community. All of these schemes appeal to at least one group of people within the affected society. In assigning prior probabilities the decision maker must decide how certain he is that an individual scheme represents the majority opinion.

TABLE E-1.

SAMPLE WEIGHTING SCHEMES

Weighting Scheme	Convenient Transport	Safe Transport	Aesthetically Pleasing	Beneficial to Community	Low Cost	Prior Probability
A	.4	.3	.1	.15	.05	.3
B	.1	.4	.3	.1	.1	.2
C	.2	.2	.2	.35	.05	.5

In Table E-1 the decision maker has determined that he is 50% certain that Scheme C, the community benefit scheme, is representative of community desires, 30% certain that Scheme A is realistic and 20% certain that Scheme B is preferred by the population. These prior probability assignments must be made based on public inputs to the designer through public hearings, local 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 worths 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 distribution and the weighting schemes can be applied.

Monte Carlo sampling can be accomplished by generating a random number between zero and 100 and entering on the probability side of the cumulative performance measure distribution curve. Where the random number crosses the cumulative curve fixes the value of the performance measure for that one sample. The technique is the basis for much computer simulation work and is fully explained in any operations research text. Such strategies have also been previously applied in decision-making under uncertainty as documented in a paper by Pouliquen written for the World Bank (Pouliquen, 1970).

This technique can be repeated to yield one performance measure for each of the attributes, which can subsequently be transformed to a worth value. If this procedure is followed 100 times we 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 distribution of payoffs would result for each alternative. These curves would form the basis for decision.

The ability to choose different weighting strategies enables the decision maker to test the sensitivity of his weights. In public investment decisions where good data on community preference is almost always lacking, this is very desirable. If the decision maker can defend his choice in a logical manner he can successfully "sell" his decision to many unwilling special interest groups.

Step 10: Produce Payoff Distributions

The final step before the decision is to graph the computed worths for each alternative in a cumulative distribution format. An example is given in Figure E-9 of three alternatives evaluated under a group of equally likely weighting schemes. The decision maker is presented with much more information than a simple mean payoff. (Although means could easily be computed and plotted.) The distribution of payoff is a much more meaningful device or tool for evaluating alternatives since it gives ranges and the shape of the entire function.

In the example, the decision maker sees that Alternative 2 has the highest payoff most of the time or is the best alternate in about 55% of the simulated cases. In addition, Alternative 2 is the worst of the three about 20% of the time. Alternative 1, however, is the best selection 45% of the time and is only slightly second best the remaining 55%. Also, Alternative 1 is always better than Alternative 3.

A further refinement of the payoff distribution might be to include the weighting schemes on the graph so that the decision maker could see the effect of different weighting schemes on the final payoff. Figure E-10 demonstrates this type of information by presenting two alternatives which are evaluated using three different weighting schemes. The bands represent the outer limits of each alternative as defined by the

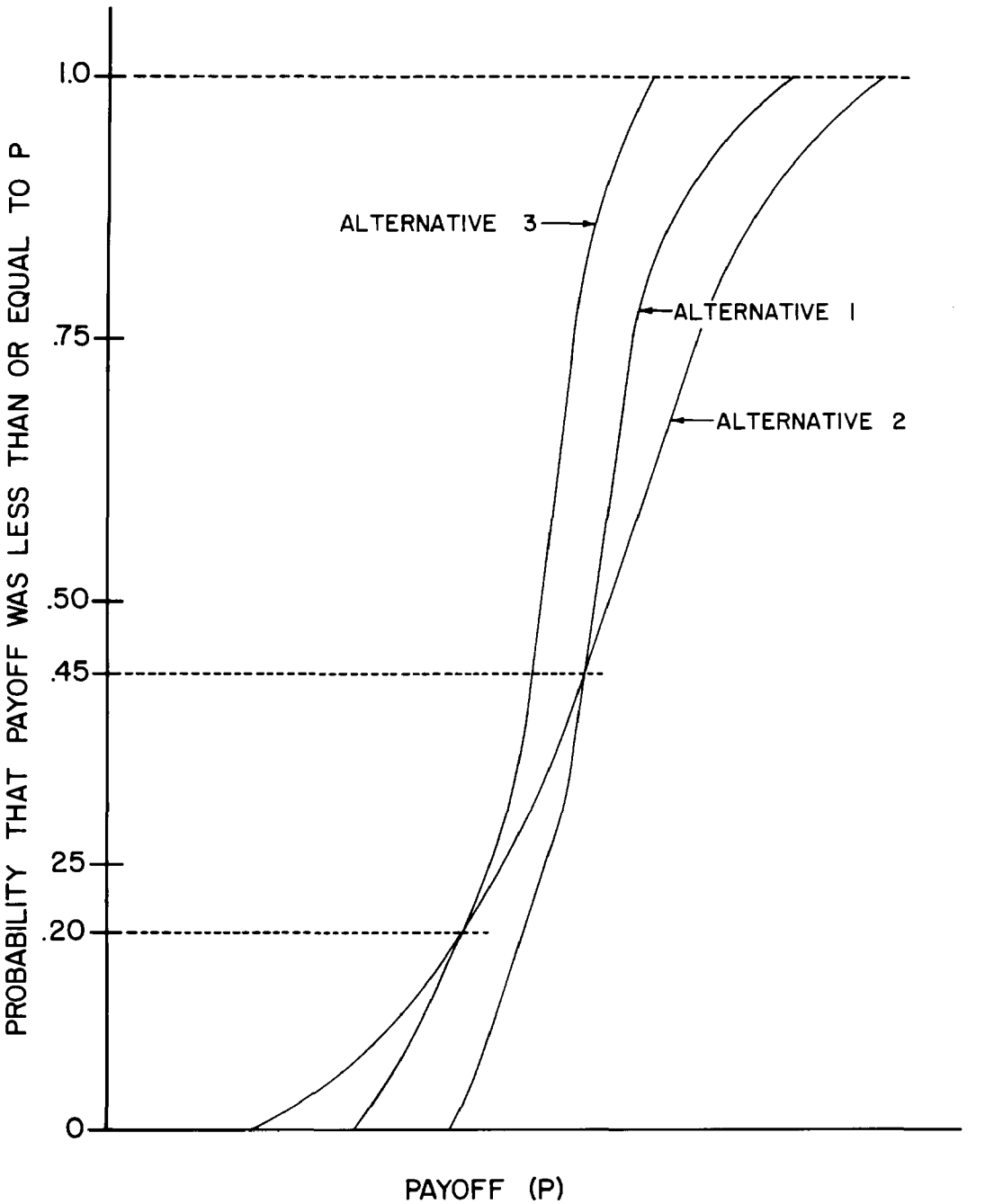


Figure E-9. Sample Payoff Distributions

individual weighting strategies. Figure E-10 shows that Alternative 2 is generally more sensitive to the different weighting schemes since the band widths of payoffs are larger.

Example Problem

One way to demonstrate the methodology proposed in this appendix is to work through an example problem. The example evaluation will be directed toward the choice between two hypothetical interchange configureations in a very simplified setting. Each alternative will be evaluated using three different weighting schemes and 24 separate performance measures. The example will follow the step-by-step procedure outlined in the previous section.

Step 1: Establish a Goal Hierarchy

For simplicity, the example goal hierarchy in Figure E-7 will be used.

Step 2: Establish Performance Measures

The 24 lowest-level goals and the attendant performance measures are shown in the left columns in Table E-2.

Step 3: Generate Alternative Designs

It will be assumed that both alternatives are feasible and representative of all reasonable solutions to the problem.

Step 4: Obtain Performance Distributions

It is assumed that the set of 24 performance distributions shown in Figure E-11 apply to both alternatives. No attempt was made to insure that the distributions are illustrative of two real world alternatives; rather, the distributions are intended to demonstrate the variety

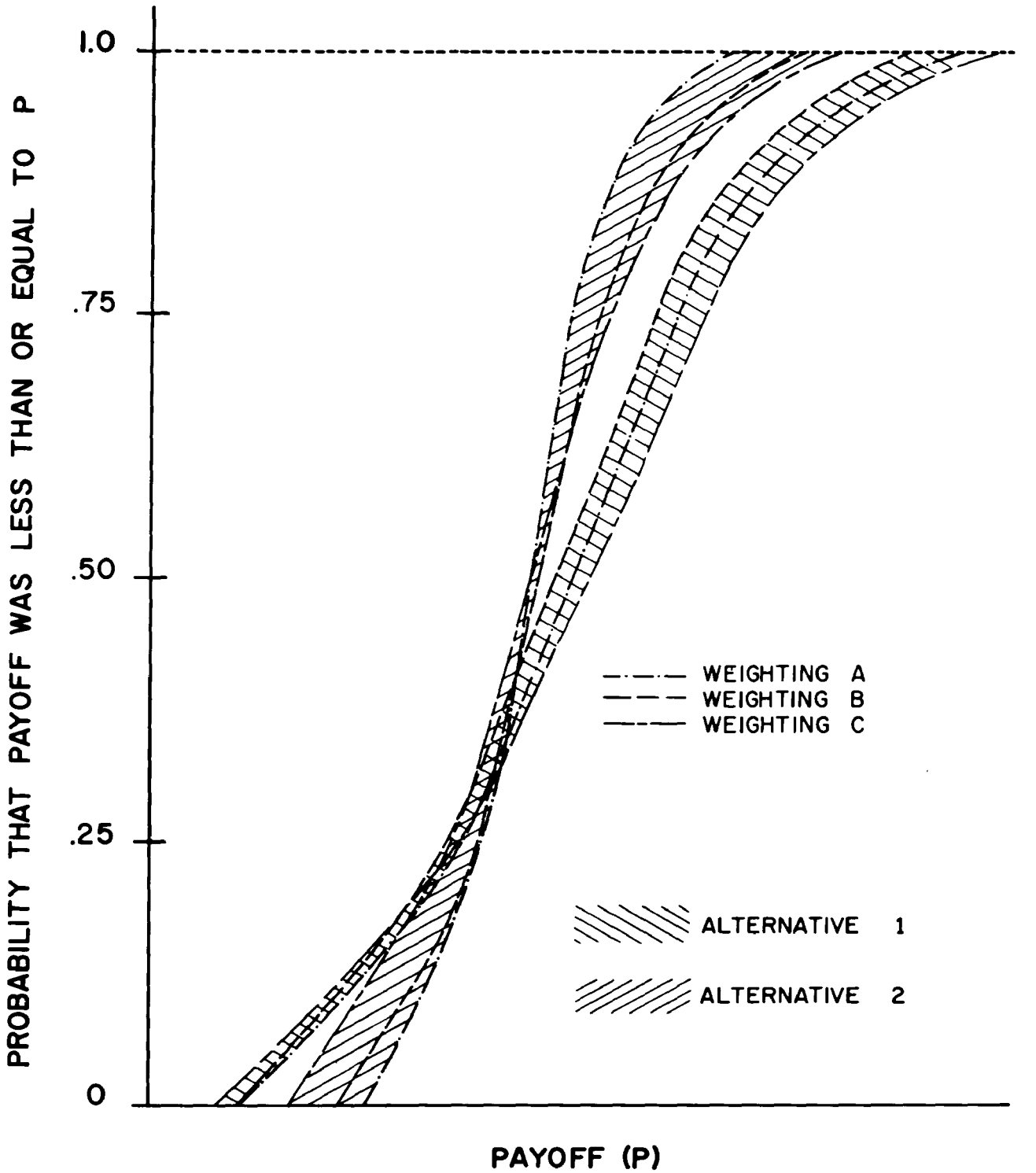


Figure E-10. Sample Payoff Distributions with Weighting Bands

TABLE E-2

EXAMPLE PROBLEM PERFORMANCE MEASURES AND WEIGHTING SCHEMES

	Goal	Performance Measure	Weighting Schemes		
			A Weights	B Weights	C Weights
1.	Low travel time	Reduced time (minutes)	.120	.030	.060
2.	Free flow	Level of service (A, B, C, or D)	.160	.040	.080
3.	Reduce user stress	Direct worth	.120	.030	.060
4.	Low fatalities	Reduced fatal accidents	.180	.240	.120
5.	Low inquiries	Reduced injury accidents	.090	.120	.060
6.	Low property damage	Reduced PDO accidents	.030	.040	.020
7.	Low noise (user)	Decibels at C/L	.012	.036	.024
8.	Visually pleasing (user)	Direct worth	.016	.048	.032
9.	Comfortable ride (user)	Roughness index (1-10)	.012	.036	.024
10.	Low noise (non-user)	Decibels at ROW line	.018	.054	.036
11.	Visually pleasing (non-user)	Direct worth	.018	.054	.036
12.	Low water pollution	% increased	.012	.036	.024
13.	Low air pollution	% increased	.012	.036	.024
14.	Increase industry	Increased payroll (%)	.0225	.015	.0525
15.	Decrease unemployment	Increased jobs	.030	.020	.070
16.	Increase tax base	Increased assessed value	.015	.010	.035
17.	Increase fire protection	Decreased avg time	.0075	.005	.0175
18.	Improve neighborhood	Direct worth	.015	.010	.035
19.	Improve poor & aged mobility	Direct worth	.015	.010	.035
20.	Improve recreation	Direct worth	.015	.010	.035
21.	Permit desired growth	Direct worth	.030	.020	.070
22.	Construction cost low	Dollars	.010	.020	.010
23.	User cost reduced	Dollars/year reduced	.024	.048	.024
24.	Facility cost reduced	Dollars/year reduced	.016	.032	.016
		Total	1.000	1.000	1.000

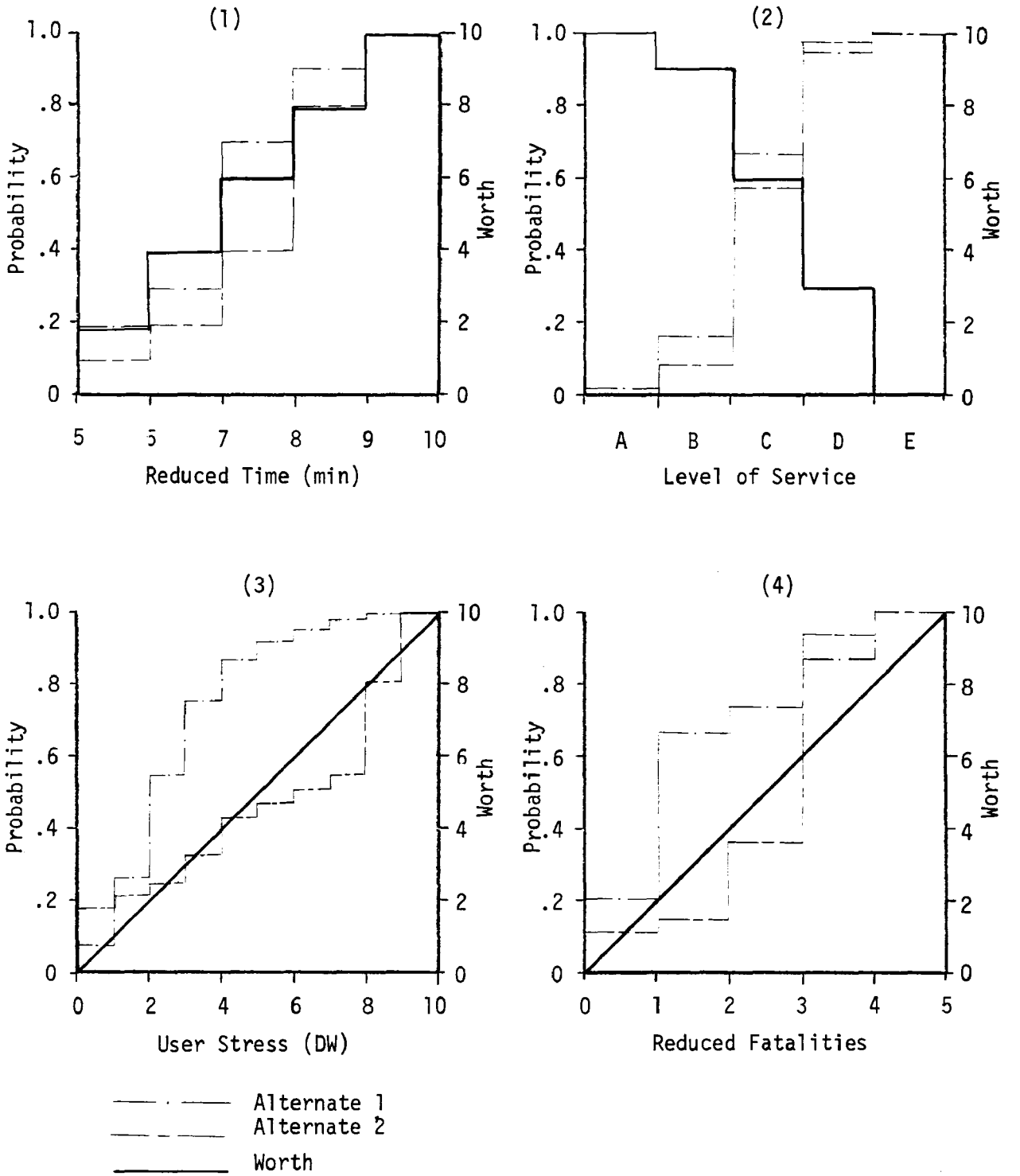


Figure E-11. Performance and Worth Distributions

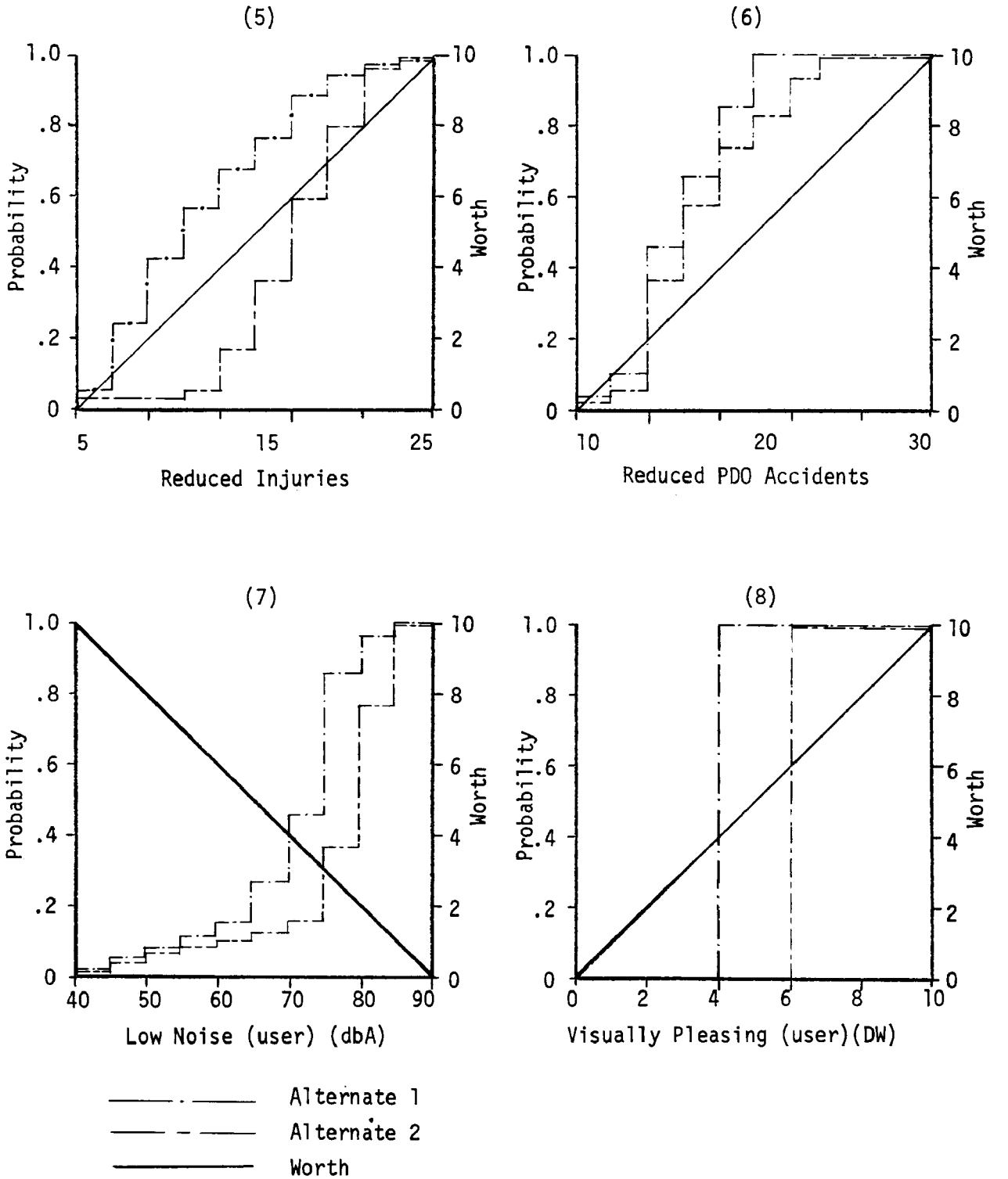
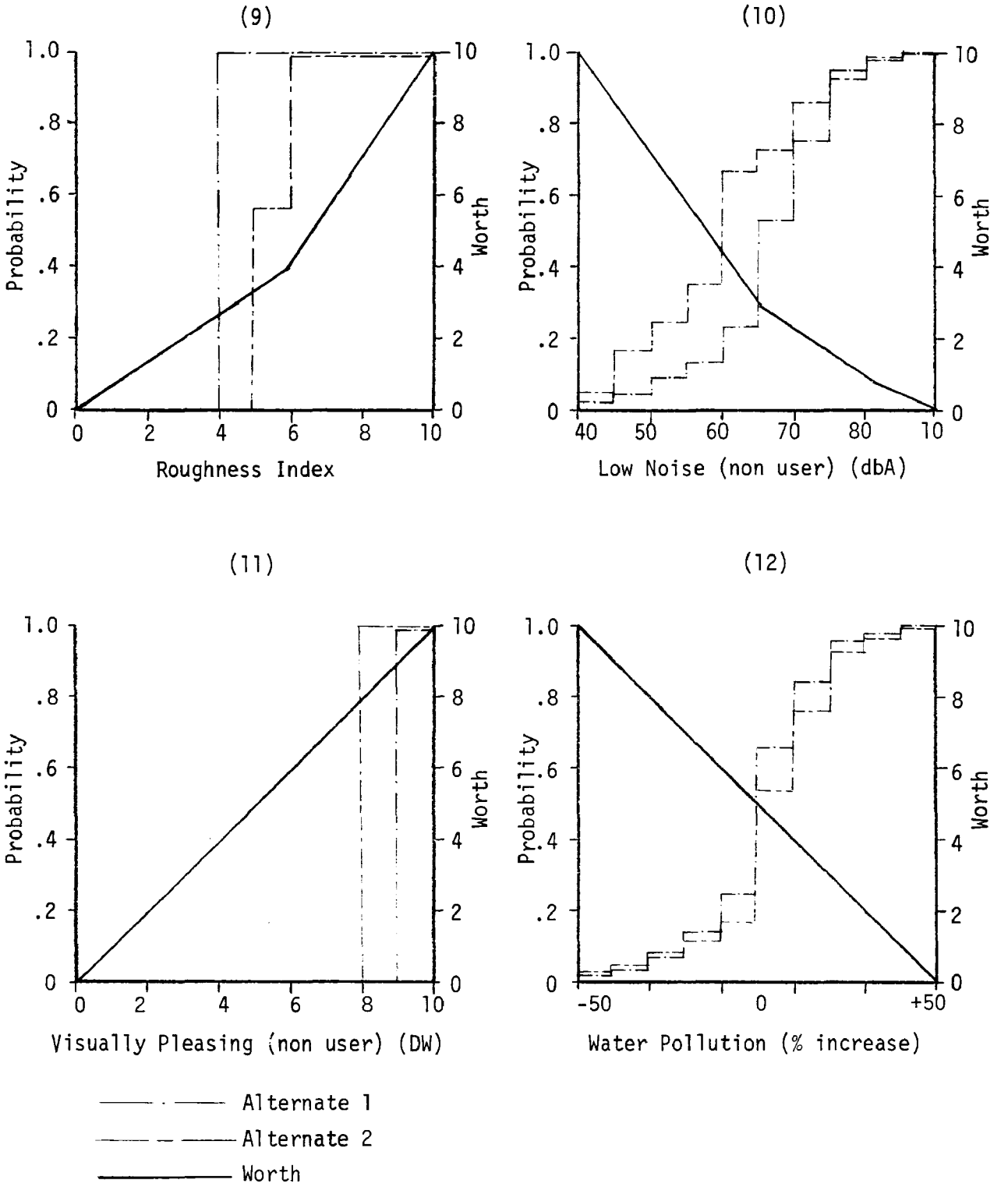


Figure E-11 (cont.)



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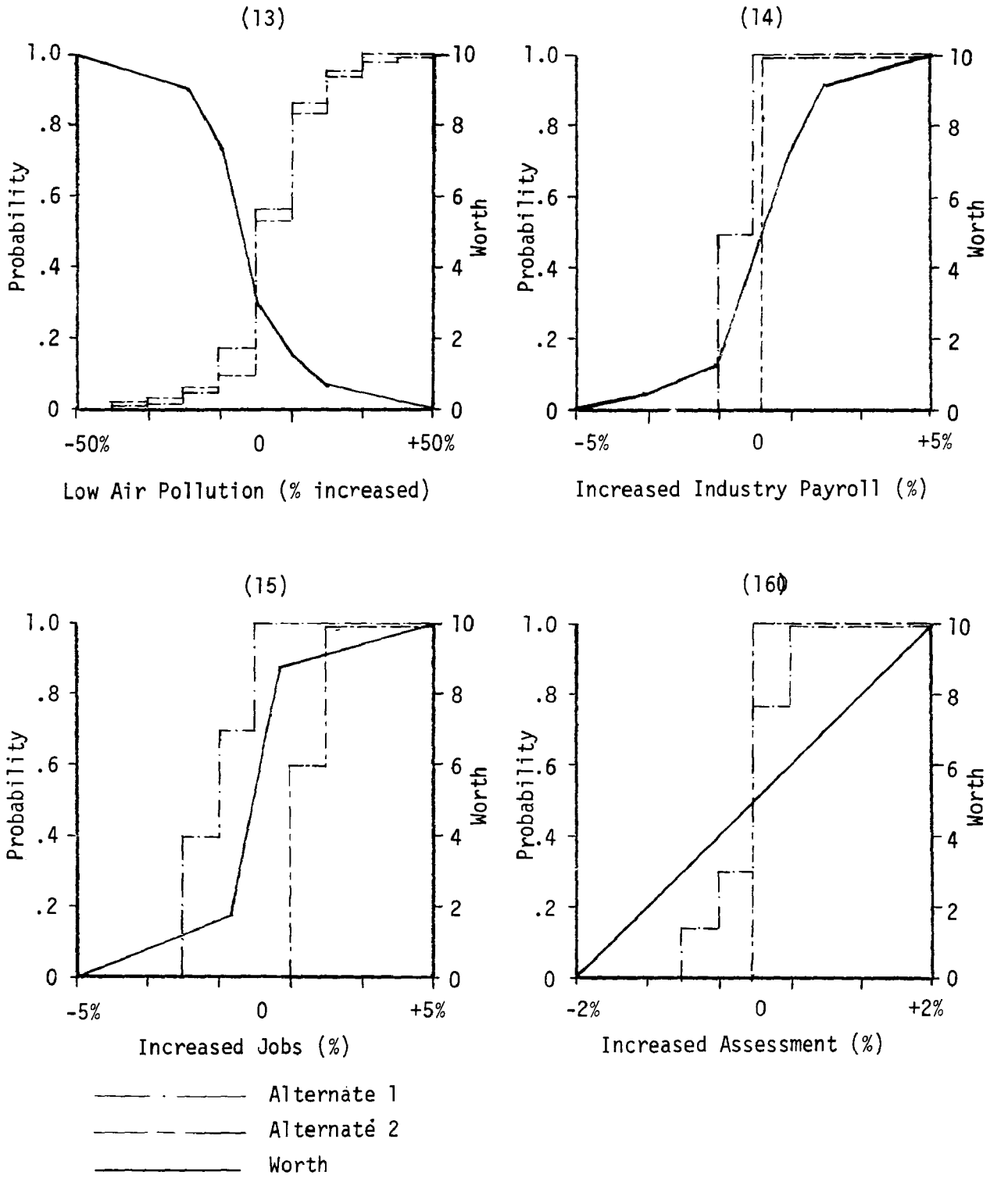

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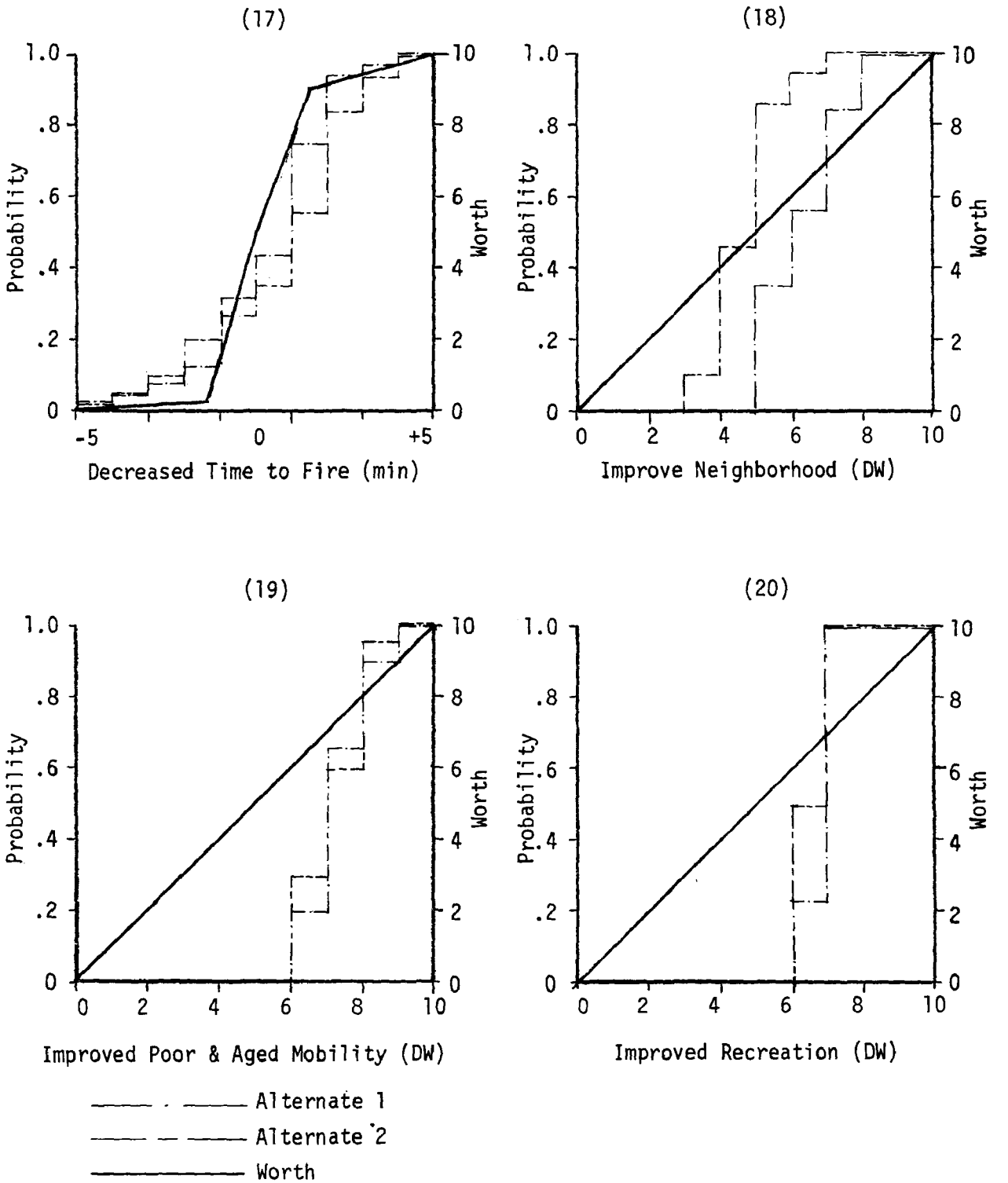

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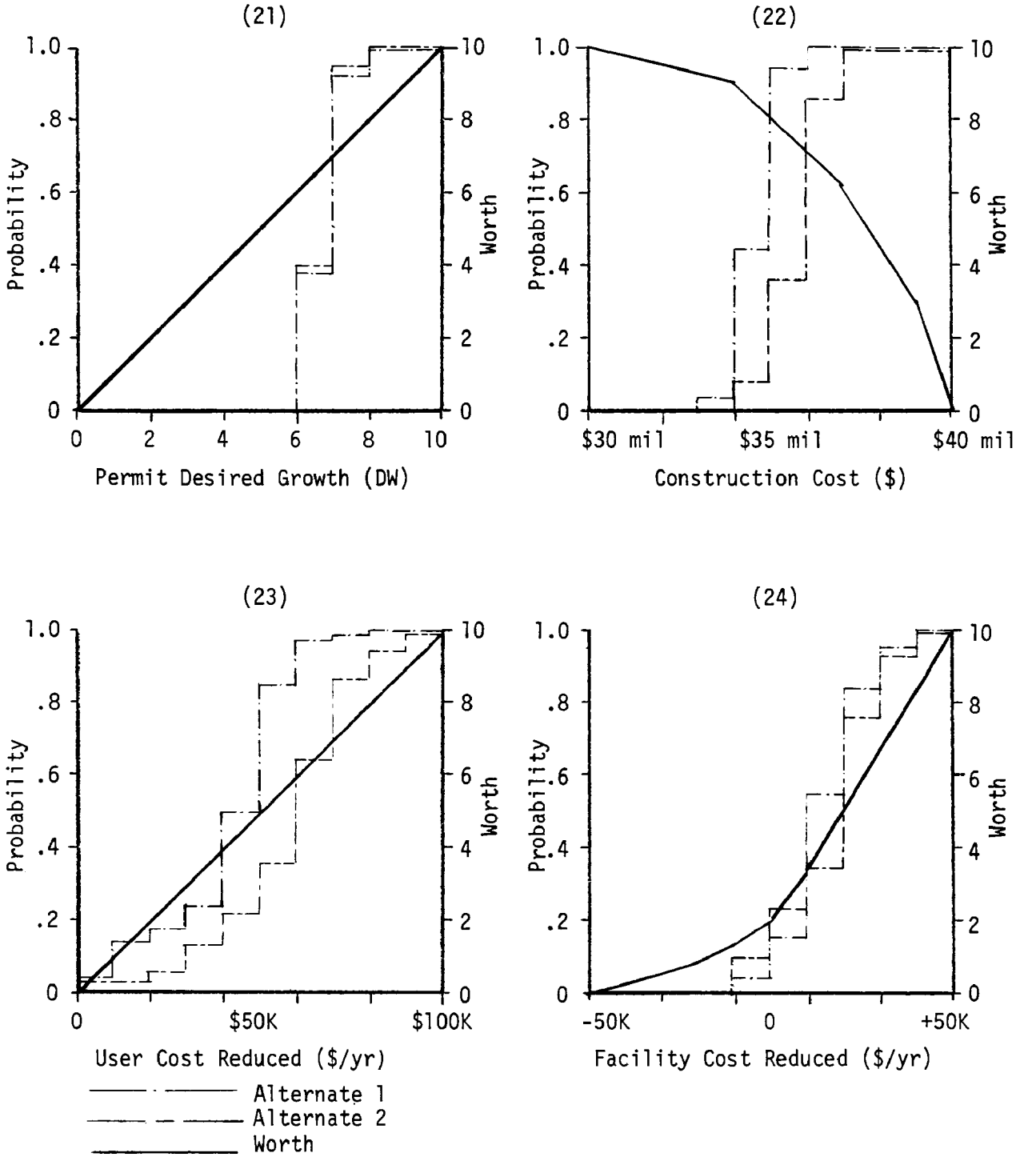

Figure E-11 (cont.)

of shapes which might be encountered in a typical analysis. Certain measures, such as visual pleasure, may be represented as point values, while others, such as reduced injury accidents, are shown to have a nearly uniform distribution. The decision maker can represent his estimate of uncertainty through the shape of his performance measure distributions.

Step 5: Obtain Worth Transformation Functions

The worth transformations for the example are also shown in graphical form in Figure E-11. As was the case with the performance measures, the worth transformations were selected to demonstrate the variety of functions available rather than to represent some particular decision-maker's actual feeling. Worth transforms can be step functions, or continuous forms, either sloping positively or negatively. Direct worth transforms are simply one-to-one transformations, shown in Figure E-11 as a positively sloping 45° line.

Step 6: Generate a Number of Weighting Schemes

Three systems for weighting the performance measures are shown in Table E-2. These were taken from the goal hierarchy example in Figure E-8 and are intended to demonstrate a user convenience oriented system (Scheme A), a safety oriented system (Scheme B), and community-benefits-oriented strategy (Scheme C).

Step 7: Assign Prior Probabilities to the Weighting Schemes

For the example problem it is assumed that the probability of Scheme A being representative of community desires is .3; Scheme B is .2; and Scheme C is .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.

Step 8: Monte Carlo Sample the Performance Measure Distributions

A random number between 1 and 100 for each of the 24 performance measures was determined by consulting a table of random numbers. The number was used to enter the performance distributions and therefore set a performance measure for that particular sample. This measure was transformed to worth by using the transformation function for the performance measure. The procedure was repeated until a set of worth measures for each of the 24 variables were produced for all of the 25 samples of each alternative. One sample set for each alternative is shown below in Table E-3.

Step 9: Monte Carlo Sample from the Weighting Distribution

A random number between 1 and 100 was generated for each of the 50 samples (25 for Alternative 1 and 25 for Alternative 2). A random number less than or equal to 30 indicated weighting Scheme A, 31 through 50 indicated Scheme B, and greater than 50 fixed the weights as C. For Alternative 1 there were seven A weights, only two B weights and 16 C weights. Of the 25 samples of Alternative 2, five were weighted according to Scheme A, five with Scheme B, and 15 with Scheme C.

As the sample sizes are increased the numbers of each weighting scheme would approach the expected number, computed by multiplying the prior probability by the sample size. For example, the expected number of B weighted samples for Alternatives 1 and 2 is .2 times 25 or 5. Only two actually appeared in Alternative 1 because of the small sample size and the randomized process of selection. For ease of hand computation the sample sizes were kept to a minimum, but if the procedure were automated, much larger samples (200-300) would be appropriate.

TABLE E-3.

EXAMPLE PROBLEM SAMPLE VALUES FOR ONE SAMPLE

	Performance Measure	Alternative 1		Alternative 2	
		Random No.	Worth	Random No.	Worth
1.	Reduced time (minutes)	21	4	25	6
2.	Level of Service (A, B, C, or D)	84	3	40	6
3.	Direct worth	78	5	50	7
4.	Reduced fatal accidents	38	3	77	7
5.	Reduced injury accidents	18	2	48	7
6.	Reduced PDO accidents	33	3	23	3
7.	Decibels at C/L	63	3	96	1
8.	Direct worth	17	4	71	6
9.	Roughness index (1-10)	68	3	22	4
10.	Decibels at ROW line	53	3	92	1
11.	Direct worth	12	8	26	9
12.	% increase	64	5	87	3
13.	% decrease	25	3	55	3
14.	Increased payroll (%)	14	3	52	6
15.	Increased jobs	64	2	98	9
16.	Increased assessed value	94	7	26	6
17.	Decreased avg time	67	9	15	0
18.	Direct worth	65	8	74	6
19.	Direct worth	76	9	19	7
20.	Direct worth	16	7	47	7
21.	Direct worth	55	8	78	8
22.	Dollars	93	8	99	6
23.	Dollars/year reduced	80	6	1	2
24.	Dollars/year reduced	57	6	98	9

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Step 10: Produce Payoff Distributions

The final step in the evaluation is to multiply the appropriate weighting strategy by the worth values and sum the score to get a total worth. The total worth has limiting values of 0 and 10, with the higher scores being preferable. The distribution of total worth scores for each alternative is tabulated in Table E-4. From this ranking the cumulative distribution of total worth was plotted as Figure E-12. The heavy lines are the worth distributions for each alternative taken over all three weighting schemes.

In addition, Figure E-12 shows the effect of different weighting schemes on the total worth of each alternative. These form bands which enable the decision maker to evaluate each alternative under the best or worst weighting condition.

In comparing Alternatives 1 and 2, it is obvious that the second alternative is preferable. It gives a higher expected utility (total worth score) under many sets of circumstances, and in no case does it give a lower one. Only in one instance is there a tie, and that occurs when weighting scheme A is the true preference of the public. Now, in the "unluckiest" 20% of the cases, Alternative 2 gives a total worth of only 4.8. However, we see that under the same condition (weighting scheme A), Alternative 1 may also be that "unlucky" 20% of the time. Furthermore, we also see that in all outcomes other than the "unluckiest" 20%, Alternative 2 does better than Alternative 1--even in the least favorable case (of weighting scheme A. In an average, 50th percentile outcome, it yields a total worth of 5.5 as opposed to 5.2. Under both other weighting schemes, the superiority of Alternative 2 is even more definitive.

The foregoing relationship might be referred to as "distribution dominance." That is, the cumulative distribution of worth scores of Alternative 2 dominates Alternative 1 at all probability levels

This is not the same as saying that Alternative 2 actually will perform better than Alternative 1, however.

TABLE E-4

EXAMPLE PROBLEM TOTAL WORTH RANKINGS

	Alternative 1			Alternative 2		
Rank	Sample Number	Total Worth	Weight	Sample Number	Total Worth	Weight
1	21	3,3090	B	15	4.8585	A
2	8	3.9750	B	7	5.0350	B
3	11	4.2155	C	25	5.4965	A
4	16	4.4370	C	3	5.5730	A
5	1	4.6000	C	5	5.6505	C
6	17	4.8585	C	9	5.8170	C
7	2	4.8625	A	22	5.8725	C
8	12	4.8875	A	17	6.0285	C
9	24	4.8880	C	1	6.0650	C
10	10	4.9205	C	16	6.1015	C
11	9	4.9495	C	14	6.1395	C
12	13	4.9865	A	21	6.2070	B
13	15	5.0420	C	24	6.3510	B
14	25	5.2015	C	2	6.5040	C
15	7	5.2180	C	13	6.5785	C
16	4	5.3385	A	8	6.6045	C
17	22	5.3790	C	12	6.6210	C
18	5	5.3815	C	23	6.6290	C
19	3	5.4735	C	20	6.6800	A
20	23	5.5575	A	18	6.7275	A
21	20	5.5630	C	4	6.9635	C
22	18	5.9650	A	11	6.9845	C
23	6	6.0680	C	6	7.0965	C
24	19	6.1285	A	19	7.3210	B
25	14	6.3520	C	10	7.5250	B

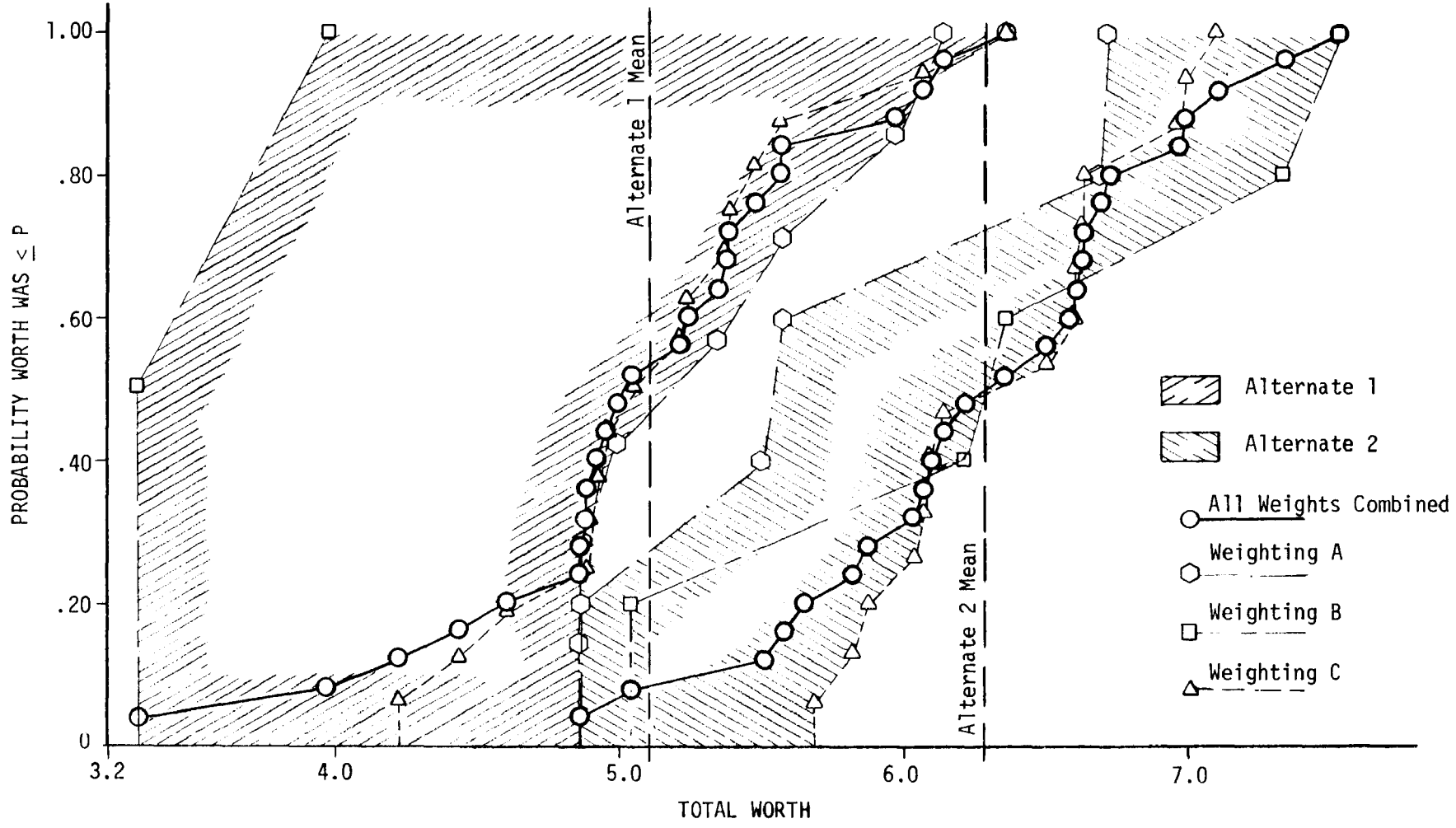

Figure E-12. **Example Total Worth Distributions**

There may occur individual outcomes in which, due to chance factors, Alternative 1 would actually achieve a higher score. What is meant here by "chance" factors are (as always) really causal elements, but those which must be treated as random variables due to our lack of knowledge of how they operate. Clearly, the more that such relationships become known and incorporated into the model, the fewer erroneous decisions will be generated. Meanwhile, it is clearly desirable to recognize explicitly such "chance" sources of error. One can thereby not only take the best of calculated risks, but also known how likely he is to make a mistake.

Implementation Difficulties

It is not likely that the methodology for interchange evaluation presented above will be given instantaneous approval by the design community. The difficulties which might be expected in implementing a decision theory approach can be grouped into user-oriented and organization-oriented problems.

Organizational Difficulties

An evaluation technique sophisticated enough to be useful in the design of major interchanges must rely heavily on a computer. This tie to a computer facility, in most cases, means that there must be a "middle man" between the designer and the decision. The decision-maker must submit his judgment on the various evaluative categories to the computer in some format recognizable by each. This coding procedure requires transformation of the abstract judgment of a designer to a stark numerical form acceptable to the computer. Since most designers are not overly familiar with the computer, nor do most computer systems

personnel have an adequate understanding of the design process, a certain amount of organizational difficulty is to be expected.

Highway departments generally lack the organizational structure necessary to gather the type of public opinion data necessary to devise representative weighting schemes. The public hearing and governmental review procedures currently in use are not intended to quantify public opinion, nor are the procedures employed really suitable for such data collection. The existing process enables those most closely affected to voice their dissatisfaction with a proposed design and relies on representative government to reflect the wants and needs of the rest of the public. Neither of these sources are quantified to the point required in a decision theory evaluation. The highway departments, therefore, must actively solicit public opinion in a quantifiable manner acceptable to the methodological framework.

A final organizational impediment to implementing a decision theory evaluation in major interchange decisions is that such a technique must be integrated within the over-all planning process. As was indicated in Chapter Two, the interchange design process cannot be separated from the over-all transport system planning procedure. Therefore, an interchange evaluation method cannot be applied without integrating it into the larger process. Decision theory evaluative techniques can be applied throughout the planning program with only refinements in the evaluative categories or subgoals necessary. In addition, such an integration would enable the data collection tasks for transport planning to be more comprehensive and coordinated.

User Resistance

The organizational impediments can be overcome relatively simply through the application of additional manpower and financial resources

and some organizational revision. The resistance to a decision theory approach by the prospective users cannot be so easily circumvented.

The designers who make the decisions are by-and-large not familiar with the language or concepts of Bayesian decision theory. Rather than regarding the techniques as aids to decision-making, they tend to see decision theory approaches as "numbers games" aimed at taking the decision out of their hands. The prospective users must be educated to the advantages of the theory and be persuaded that it doesn't replace judgment but rather focuses it.

This focusing of informal judgment is itself a difficulty in user acceptance. To force oneself to quantify judgment could be a painful exercise which would deter the user from adopting the evaluative technique. This discomfort "cost" is magnified when one is uncertain about the benefits of such a technique. The decision-maker can never really "know" if the application of the methodology presented above will lead him to a sufficiently better decision to justify the real and discomfort costs.

Trade-Off Analyses Extension

Introduction

The largest scale problem in interchange evaluation is the evaluation of alternative configurations. The smaller scale problem -- the level of investment in the individual components -- may be analyzed through a scaled down version of the methodology presented above. When the problem becomes one of the choice between alternative lengths of deceleration lanes, for example, the goal hierarchy and weighting schemes can be reduced to a very simplified evaluation system. In

fact, evaluative attributes such as safety, operations and cost may be considered as the only attributes with any weight at all in the analysis.

The problem, then, is reduced to a trade-off analysis where costs are balanced against one other attribute, operational and safety benefits. Therefore, the weighting strategy which could be used to combine these two attributes is superfluous to the analysis -- direct comparisons would be just as meaningful.

The uncertainty involved in the cost and operational and safety performance levels may also be reduced to a point estimate. The cost of added pavement in a longer deceleration lane can be accurately estimated, for example, making point value analyses acceptable. It further simplifies the methodology, promoting increased useage.

This problem approach was discussed at the workshops to obtain the designers' views on such a scheme. It should be noted that at the time of the workshop the decision theory approach had not been devised so that some of the terminology in the trade-off presentation was not consistent with the previous section. The most obvious difference is between the "Level of Merit" concept introduced in the trade-off section and the worth or utility scores discussed in the initial part of this appendix. The concepts are essentially the same.

Workshop Questionnaire

After some introductory remarks on the need for trade-off analysis, a set of discussion questions were posed to the workshop participants. These were followed by a period of open discussion and distribution of a questionnaire. The participants were asked to complete and return the questionnaire the next day -- thereby giving them an opportunity to discuss the subject further among themselves and to consolidate their

thinking. In general, the questions were nearly the same as those presented for discussion. The questions, with answers received, are given in Table E-5.

As can be seen from Question #1, the interest in and feeling of necessity for economic analyses decreases somewhat as the design decision becomes more and more specific. This is logical in that the alternative costs become relatively smaller and the over-all project constraints are pretty well set by the time the design details are selected. A number indicated that more economic analyses would be desirable but that appropriate methodology was not available. However, there is no clear mandate for the development of this methodology.

Answers to Question #2 indicate that "engineering judgment" is the most used decision-making procedure on including "desirable features." It is perhaps surprising, and certainly encouraging, that only about a third of the respondents indicated their organization had adopted the policy of simply meeting certain minimums.

Again, in Question #4, it is apparent that experience is the prime input to the design decision process, although considerable attention is being paid to accident record analyses and pertinent research results.

Level of Merit Concept

Cost and some measure of operations and safety are two major trade-off factors receiving consideration in the selection of alternative component configurations (such as left vs. right ramps, single vs. double exits, etc.) and in the specification of design dimensions (design speed for a given ramp, length of acceleration lane in a given situation, etc.). In development of a final interchange design, a number of these trade-off decisions are made (although, perhaps, not "consciously").

TABLE E-5.

RESULTS FROM QUESTIONNAIRE ON TRADE-OFF ANALYSIS

Questions and Answer Choices	No. of Participants Selecting Given Answer
1. Economic analyses (cost/benefit ratios, rate-of-return methods, etc.) as applied to major interchange design. Please circle the statement you feel most appropriate:	
a. Economic analyses in comparing alternative interchange configurations as a whole	
i. Common practice	5
ii. Desirable and feasible, but not usually carried out	5
iii. Desirable, but not feasible -- appropriate methodology not available	3
iv. Of little practical value; other considerations are determining factors	11
v. Other _____	6
b. Economic analyses in selection of alternative components -- (loop ramp vs. direct connection; collector-distributor roadway vs. double exit, etc.)	
i. Common practice	5
ii. Desirable and feasible, but not usually carried out	6
iii. Desirable, but not feasible -- appropriate methodology not available	2
iv. Of little practical value; other considerations are determining factors	12
v. Other _____	5

Table E-5. (Continued)

Questions and Answer Choices	No. of Participants Selecting Given Answer
c. Economic analyses in specification of design dimensions -- (length of acceleration lane, radius of curvature of loop ramp, etc.)	
i. Common practice	2
ii. Desirable and feasible, but not usually carried out	5
iii. Desirable, but not feasible -- appropriate methodology not available	4
iv. Of little practical value; other considerations are determining factors	17
v. Other_____	2
2. How do you reach decisions on "desirable features," such as exclusion of left-hand exits, good visibility of the exit area, uniformity of exiting maneuvers, etc.? (Circle one)	
a. Decision to meet AASHO Blue Book minimums at all costs	7
b. Decision not to incorporate (or exclude) certain features at all costs	2
c. Attempt benefit/cost (or similar) analysis for individual situations	5
d. Engineering judgment -- i.e., no formal analysis of cost factors as such	15
e. Other_____	1

Table E-5. (Continued)

Questions and Answer Choices	No. of Participants Selecting Given Answer
3. Can meaningful cost data be obtained for individual components (ramp configurations, length of deceleration lane, etc.)?	
a. Yes; Comment_____	19
b. No; Comment_____	9
4. How do you assess "benefits" to justify extra expenditures for improving on "minimum" design standards? (Circle any appropriate answers)	
a. Accident record analyses of similar situations	15
b. Experience in observing similar situations, and relating this to extra costs involved	19
c. Study of research results in these areas	12
d. Consensus of personnel in your design department	12
e. Usually use minimum values	0
f. Other_____	7

If design engineers are asked: -- "Is it more desirable, from an operations and safety viewpoint, to provide a single exit (with subsequent branching for left and right movements) or two individual exits?" -- The answer is almost unanimously: -- "Single." However, if then asked which configuration should be established as a design standard to be rigidly adhered to, the answer becomes somewhat less definite, and "hedging" will be noted. Obviously, the hedging comes about because designers feel there are "situations" in which the single exit should not be selected; and this is often because, in that situation, the double exit could be achieved at considerably less cost.

The same types of questions and answers can be applied to other design features, such as right vs. left ramp, length of acceleration lane, etc. In other words, there are known desirable features, but something less is often used because of some cost factor. Designers claim it is impossible to give a set answer to any of these types of questions which will hold across all situations. A major reason for this is that they are trying to assess cost and merit (worth or utility) measures at the same time, and, as the combinations are nearly infinite, so are the "correct answers."

It appears, then, that since no definite universal answers can be had when the two factors are considered together, it would be helpful to decision-makers if they could assess the two factors (cost, and operations and safety merit) individually and then make their decision on the basis of relative costs and relative merits.

Assessing relative costs will usually be possible, though sometimes with considerable difficulty if the alternatives are such that a major portion of the interchange design is involved (such as a decision on a right-hand or a left-hand exit). In the case of designating

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the length of an acceleration lane, the cost analysis may be very simple (if only a little change in earthwork quantities and pavement length is required) or somewhat more difficult if the longer lane will also interfere with downstream features, require a larger grade separation structure, etc.

The problem, then, will be to assess the relative level of merit provided by the alternative configurations, or the alternative design dimensions, and then to choose among the alternative levels of performance and the corresponding costs.

Assuming, for the moment, that the specification of alternative merits is possible, the designer is then in a much better position to select the final design. This will still be a highly subjective process, dependent largely on the designer's engineering experience and judgment -- a "benefit-cost analysis" is not being suggested.

An example will illustrate the concept. Assume the following conditions:

Configuration	Merit Rating	Additional Cost
Single Exit (on right)	10	$3,000,000
Double Exit (both right)	8	$2,000,000
Double Exit (right & left)	3	0

- If the total interchange cost (with double exit, right and left) is estimated at $40,000,000, which configuration should be selected?
- If the total interchange cost (with double exit, right and left) is estimated at $7,000,000, which configuration should be selected?
- Now assume the ratings are changed to 10, 8, 6: -- Which configuration should be selected?

The fact that different configurations might be chosen under these differing conditions points up the problem of setting definitive configuration selection criteria. Even in this simple example (in practice, other considerations, such as maintenance costs, road user costs, etc., would also enter the decision-making process) it is not possible to select a single "always correct" answer.

The merit ratings give some insight to the question of "How much better?" It is agreed that a single exit is better than one incorporating a right-hand and left-hand exit, and therefore using a design incorporating a single exit justifies a higher cost -- but how much higher? First, one must determine how much "better" one configuration is than another. The merit ratings, if available, could provide some "feel" for these qualitative comparisons.

Each time a decision has been made in the past, the designer did go through some similar assessment of the relative merits and costs. The merit ratings, if they can be developed in a credible and acceptable manner, will provide some basis for a rational choice. They would provide a means by which the decisions could be made more consistently by each designer, and more consistent designs could be obtained from various designers.

As another example, assume a speed change lane (acceleration) from a turning roadway with a design speed of 40 mph to a through roadway with a design speed of 70 mph must be designed. The "Blue Book" suggests this acceleration lane should be 1,000 ft. long. Suppose, due to situational considerations, a speed change lane 800 ft. long would be $500,000 less expensive than one 1,000 ft. long; which should be selected?

Obviously, a judgment on the importance of that missing 200 ft. is required. This assessment is usually made on the judgment of the design engineer. Suppose, however, that credible "merit" ratings are available -- "8" for 1,000 ft., and "7.5" for 800 ft. Wouldn't this affect the decision in a different manner than if the two ratings were "8" and "4"? Wouldn't this degree of specificity help the designer in making this decision?

Illustrative Rating Questionnaire

At this point in the workshop discussions, the participants were asked to complete the questionnaire shown in this section, to illustrate the feasibility, and problems, of deriving consensus "expert" judgmental evaluations.

"The procedure in filling out this questionnaire is quite simple; set a value of '10' for the most desirable alternative presented, and then rate the others against that one on the basis of operations and safety -- keeping in mind that '0' designates totally unacceptable. Costs will be considered later in the decision-making process, and are not to be a factor here."

1. Various possible exit ramp configurations for an approach to a major interchange at the crossing (roughly perpendicular) of two freeways are shown in Figure E-13. Assume single-lane turning roadways, four-lane freeways and that the DHV for each turning movement is 1,000 vph.

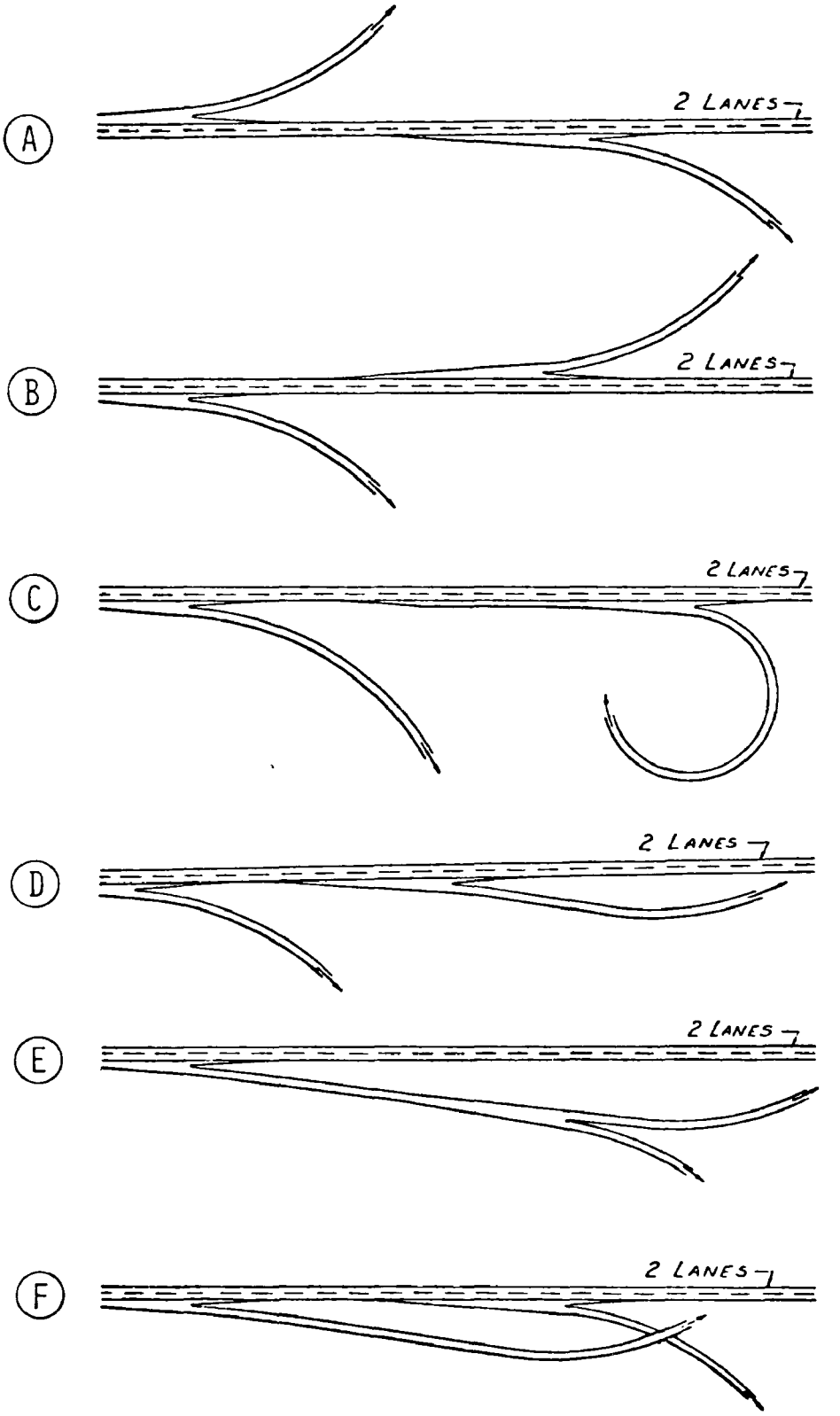

Figure E-13. Alternative Exit Ramp Configurations

Figure	Rating (Operations and Safety)
A	_____
B	_____
C	_____
D	_____
E	_____
F	_____

2. Alternative lengths for an acceleration lane of a major interchange. Turning roadway (single lane) design speed is 40 mph, through roadway is 70 mph. (Blue Book value is 1,000 ft.) Assume DHV of 1,000 vph.

Length (ft.)	Rating (Operations and Safety)
1400	_____
1200	_____
1000	_____
800	_____
600	_____
400	_____
200	_____
0	_____

Results from Illustrative Rating Questionnaire

A tabulation of the ratings given to the laternative ramp configurations is given in Table E-6. In tabulating the results, the participants were "categorized" into three groups -- Design Engineers, Traffic Operations Specialists, and Academic and Research -- so that any differences of opinion among these three areas of expertise could be noted. The number of participants in each group returning the questionnaire is indicated at the bottom of the Table.

It is interesting to note that all three groups select configuration E as the "best," and consider the left-hand exit designs the least

TABLE E-6

MEDIAN MERIT RATINGS FOR EXIT RAMP CONFIGURATIONS

	Merit Ratings (Median of those responding)			
Figure	Design Engineers	Traffic Operations Specialists	Academic & Research	All Groups
A	1	1	3	1
B	2	2	3	2
C	6	4	4	5
D	8	8	6	8
E	10	10	10	10
F	6	6	6	6
Number Responding	18	6	7	31

desirable. The Traffic Operations Specialists gave slightly lower ratings to the loop ramp configuration (C) than did the Design Engineers. Although the sample is small, the results tend to indicate that those who work with the "product" on a day-to-day basis feel even more effort (and money) should be expended to eliminate "second-choice" design features.

In general, those categorized as Academic and Research were not quite as critical of the left-hand exit designs as the other groups. A possible interpretation is that the Academic and Research group base their opinions primarily on conceptual principles and that, in fact, actual operations and safety at left-hand exit ramps are even poorer than might be anticipated.

The results of the ratings of the alternative lengths of acceleration lanes are shown in Table E-7.

Again, it can be noted that the three groups are essentially in agreement, with the Design Engineers being slightly less critical of "sub-standard" design.

It is also interesting to note that the Blue Book value has a median rating of "9" -- indicating that the participants believe this value to be adequate. A slightly higher value is reported for 1200 ft., but then it tends to drop off again as the length is extended further. From comments, it would seem this dropping off is due to concern for the excessively long merging area which might result, or the possibility that drivers might temporarily believe the lane was not going to be dropped.

TABLE E-7

MEDIAN MERIT RATINGS FOR ACCELERATION LANE LEHGHTS

	Median Ratings (Average of those responding)			
Length (ft.)	Design Engineers	Traffic Operations Specialists	Academic and Research	All Groups
1400	10	9	9	10
1200	10	10	10	10
1000	9	9	8	9
800	7	6	4	6
600	3	1	0	1
400	0	0	0	0
200	0	0	0	0
0	0	0	0	0
Number Responding	18	6	7	31

The use of group medians in Tables E-6 and E-7 masks the rather wide range of individual ratings, as the "outliers" are lost in this process. As examples, the ratings for configuration A in Figure 1 ranged from "0" to "7"; configuration D from "3" to "10"; and configuration E from "7" to "10." These large discrepancies may indicate an interpretation problem on the part of some of the respondents, or differences in past experiences with the various designers. Hence, the use of the "Delphi Method," as described by Dalkey and Helmer in an article entitled "An Experimental Application of the Delphi Method to the Use of Experts" (Management Science, vol. 9, 1963), or some similar technique for arriving at concensus opinion is suggested for future studies of this type.

Further Introductory Remarks

Before beginning the open discussion in the workshop, it was further pointed out that if these merit ratings can be set for alternative configuration choices and for design dimensions, the possibility for specifying different "levels of merit" (or total worth) for entire interchanges exists. For example, for a major interchange, the designer could specify that all configurations and design dimensions must have merit ratings of "9" or better; while for a less important interchange, configurations and dimensions with ratings of "7" might be acceptable.

Hence, these merit ratings could be used to select individual design features through comparison of relative merits and relative costs, or as a means to assure design features consistent with the "importance" of the interchange, and, if desirable, consistent within a given interchange.

This last statement leads to another question: -- "Is it ever desirable to purposely degrade a design feature so that the 'level of design' will appear to be consistent to the driver?" In other words, is it better if the driver encounters marginal quality throughout the interchange than if he observes high quality in all places in the interchange except at one critical site? Will he be deceived into thinking he is on a better grade facility than he is, in fact?

Questionnaire Results

In addition to the illustrative rating questionnaire handed out during the introductory remarks, a session questionnaire was given to

TABLE E-8

RESULTS FROM QUESTIONNAIRE ON LEVEL-OF-MERIT DESIGN CONCEPT

Questions and Answer Choices	No. of Participants Selecting Given Answers
1. Do you feel it is possible to derive meaningful ratings for alternative general configurations (as in the example of the various exit ramp configurations)?	
a. Yes; Comment_____	18
b. No; Comment_____	12
2. Do you feel it is possible to derive meaningful ratings for alternative design dimensions (as in the example of the acceleration lane lengths)?	
a. Yes; Comment_____	19
b. No; Comment_____	11
3. How should the merit ratings be developed -- utilizing which inputs? (Circle all you feel apply.)	
a. Physical analyses (acceleration potentials, friction factors, reaction times, etc.)	20
b. Accident data across alternatives	20
c. Research studies on driver behavior and preferences	21
d. Judgment of highway designers and operations specialists	17
e. Others_____	8
4. Were you "comfortable" making the ratings requested in the earlier examples?	
a. Yes; Comment_____	17
b. No.	12
If no, what additional information would have been helpful?_____	

Table E-8. (Continued)

Questions and Answer Choices	No. of Participants Selecting Given Answers
5. Do you feel the concept of using level-of-merit ratings in interchange design is:	
a. feasible? Yes No Comment_____	Yes - 14 No - 6
b. practical? Yes No Comment_____	Yes - 5 No - 11
c. deserving of more investigation, better definition, more trial, etc.?	
Yes No Comment_____	Yes - 17 No - 4
6. Is consistency in interchange "quality" important? Should some elements purposely be degraded to make them compatible with the lower standard design-controlling elements?	
a. Yes, usually. Comment_____	5
b. Yes, sometimes. Comment_____	9
c. No. Comment_____	13

the participants at the end of the discussion, and they were asked to complete it and return it the following day. As in the case of the questionnaire on Trade-Off Analyses, the questions generally paralleled those employed to structure the discussion. The questions, with tabulations of the answers, are given in Table E-8.

The answers to Questions #1 and #2 indicate somewhat more than half the participants believe it is possible to derive meaningful merit ratings. The Design Engineer group was about evenly split, while the other two groups were considerably more optimistic.

The results of Question #3 are not very informative, in that virtually everyone felt that all possible inputs should be utilized in developing these merit ratings (assuming they should be developed)

In Question #4, a number of the participants indicated they were "not comfortable" making the ratings, but they provided little information as to what would have been helpful. (The signing and lighting conditions were mentioned as other possible information inputs.)

From the results of Question #5, it can be seen that the participants generally felt the level-of-merit design worthy of more investigation and trial, but were not optimistic about obtaining a practical design tool.

No clear-cut conclusion can be drawn from the answers to Question #6. This is perhaps due to the wording of the question -- the comments accompanying the answers indicated that the participants were interpreting this question in a variety of ways.

Conclusions

The value of the evaluative methodology presented above measured in terms of improved decisions is unknown. Due to the complexity and magnitude of the problem it is infeasible to design an experiment which could establish the benefits expected from the use of a decision theory approach. The best argument for its adoption is simply that similar techniques are being used currently by large corporations with apparent success and the perceived trend is toward expansion of their use.

There will be problems in implementing the technique within the highway decision framework -- both with organizational structure and individual resistance. Highway departments are generally not organized as a corporation in that some of the staff services groups required for data collection and decision theory analysis do not exist. The decision-makers themselves are not generally acquainted with decision theory concepts nor are new engineers being trained in this particular field as business school graduates are. Remedial education is needed at the decision-making level and program revision required in current engineering training before the method will be applied on any significant scale.

If the decision theory approach is adopted and executed properly it will force informal evaluation to be better focused, which will lead to a better understanding of the reasoning process. It makes the hidden assumptions inherent in the decision explicit and, therefore, subject to scrutiny. Both of these characteristics of decision theory approaches are desirable if one accepts the notion that the more we know about a process, the better the process will work.

Finally, decision theory analysis can be an effective communication vehicle for conveying the analysis underlying the choice of a particular interchange design. This is necessary to convince both the public and design reviewers of the appropriateness of a particular design. Such a presentation vehicle would fill an existing void.

APPENDIX F

ACCIDENT ANALYSIS OF MAJOR INTERCHANGES

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APPENDIX F: ACCIDENT ANALYSIS OF MAJOR INTERCHANGES

Introduction

This limited analysis of major interchange accident characteristics attempts to answer three basic questions regarding interchange operation.

(1) How do the accident patterns at major interchanges differ from those at minor interchanges?

(2) Where on major interchanges do accidents most frequently occur?

(3) What types of connections are the least hazardous; loop, direct or semi-direct, or outer connection?

The data base used for the accident analyses is an automated accident record system maintained in the FHWA Interstate System Accident Research Program. The information system allows the user to obtain tabulations of various response variables, crossed with numerous geometric, traffic operation, and environmental characteristics. The information reported below was supplied to this research agency (via the Contract Manager) by Ms. Julie Fee, Head of the Interstate Accident Study group.

As with many studies where the original data has been collected for another purpose by another agency, many of the specific data requests could not be fulfilled. Thus the data shown here represent a compromise between what was requested and what could be supplied. Consequently, this discussion does not constitute a comprehensive study of major interchange accident characteristics.

The data which were provided represent accident histories for 1,688 minor and 37 major interchanges. The imbalance between the two types of interchanges implies a great deal about the confidence one may place in the resulting figures. By virtue of the significantly larger numbers,

the statistics on minor interchanges are necessarily more reliable, and therefore can be more easily generalized. With only 37 major interchanges, representing a wide range of geometrics, within the data base, one must be considerably more cautious in generalizing from the resulting average figures.

Major vs. Minor Interchanges

The initial question posed dealt with a comparison of the general safety characteristics of major versus minor interchanges. Table F-1 presents the accident, injury and fatality rates (per 100 million vehicle miles) by rural and urban types of major and minor interchanges.

TABLE F-1

ACCIDENT CHARACTERISTICS OF MAJOR AND MINOR INTERCHANGES

	Interchange Type			
	Minor		Major	
	Urban	Rural	Urban	Rural
Accident Rate (Accidents/100 M veh-mi)	240	122	174	225
Injury Rate (Injuries/100 M veh-mi)	155	81	112	149
Fatality Rate (Fatalities/100 M veh-mi)	3.1	4.3	2.9	4.1
Number of Interchanges	529	1059	15	22

Accident rates for major and minor interchanges have an irregular pattern when stratified by urban and rural locations. That is, accident rates at major interchanges are lower in urban areas than in rural areas, whereas the opposite is true for minor interchanges.

One might assume that since turning volumes on major interchanges are usually much heavier than on minor ones, the major interchanges would be more hazardous due to the many merges and diverges. The results support this assumption except for the urban-minor case, where the rate is the highest of all four interchange classes. The explanation for this high rate may lie in the types of ramp connections with the cross streets. These would typically be diamond connections with high volume, at-grade junctions controlled by signals or stop signs. Since accidents in this area may have been classified as "interchange" accidents, this one feature inflates the accident rate artifically for the urban-minor interchange category. Extremely low cross street volumes would decrease the hazard at rural-minor locations.

The injury rate statistics reflect the same rankings as the accident rate figures. A measure of the severity of the accidents which occur on each type of interchange would be the injuries per accident, which can be computed by dividing the injury rate by the accident rate. The results of this calculation are shown in Table F-2. It is obvious that the severity of accidents, as measured by injuries per accident, are essentially the same for all interchange types.

TABLE F-2

INJURIES PER ACCIDENT FOR INTERCHANGE TYPES

	Interchange Type			
	Minor		Major	
	Urban	Rural	Urban	Rural
Injuries/Accident	.65	.66	.65	.66

Accident Locations on Major Interchanges

The second question posed is where on major interchanges do accidents occur and where are they most severe. Table F-3 shows accidents per deceleration area and per acceleration area. On major interchanges it appears that deceleration areas are more hazardous than acceleration areas. The term "area" is used here rather than lane because the data for some sites include the appropriate half of a combined accel-decel lane.

The number of units shown in Table F-4 and subsequent tables does not refer to the actual number of sites in the data base. It refers, instead, to the number of location entries in the data base. Therefore, some double counting occurs when the same location is entered into the file for two separate time periods. This accounting method requires that extreme caution be used in interpreting these data.

Table F-4 is refined from the previous table in that left- and right-side subdivisions have been added. The first thing to note in Table F-4 is that only three left-side acceleration areas are represented in the sample, rendering the per unit statistics virtually useless. There are 26 left-side deceleration areas which, while far from the 330 right-side total, are enough to make some limited inferences on the safety of left-side terminals. The accident rate per unit is nearly twice as high for left deceleration areas as for the right side.

Table F-5, showing accidents by terminal type and location, is the most finely divided. It shows deceleration and acceleration lanes separated from combined accel-decel lanes and includes left- and right-side breakdowns. The table indicates that combined lanes operate more safely than separate acceleration or deceleration lanes.

TABLE F-3

ACCELERATION VS. DECELERATION AREA ACCIDENTS ON MAJOR INTERCHANGES

Unit	Number of Units	Number of Accidents	Accidents/ Unit
Deceleration Area	356	431	1.21
Acceleration Area	323	271	.84

TABLE F-4

LEFT-SIDE VS. RIGHT-SIDE ACCELERATION AND DECELERATION AREA ACCIDENTS ON MAJOR INTERCHANGES

Unit	Number of Units	Number of Accidents	Accidents/ Unit
Deceleration			
Right Side	330	376	1.14
Left Side	26	55	2.12
Acceleration			
Right Side	320	243	.76
Left Side	3	28	9.33

TABLE F-5

ACCIDENTS BY TYPE OF TERMINAL ON MAJOR INTERCHANGES

Unit	Number of Units	Number of Accidents	Accidents/ Unit
Deceleration Lane			
a. Right Side	242	311	1.29
b. Left Side	23	50	2.16
c. Combined	265	361	1.36
Acceleration Lane			
a. Right Side	235	185	.79
b. Left Side	3	28	9.33
c. Combined	238	213	.89
Deceleration Half of Combined			
a. Right Side	88	65	.74
b. Left Side	3	5	1.67
c. Combined	91	70	.77
Acceleration Half of Combined			
a. Right Side	85	58	.68
b. Left Side	0	0	None
c. Combined	85	58	.68

Accidents for Different Connections on Major Interchanges

The final question considered in this analysis is that of the relative safety of different types of connections; specifically loop ramps, outer connections, and direct or semi-direct turning ramps. Table F-6 contains accident rates expressed as the number of occurrences per 100 million vehicle-miles for the three types of connections, for both one- and two-lane widths.

TABLE F-6

ACCIDENTS BY TYPE OF CONNECTION ON MAJOR INTERCHANGES

Unit	Number of Units	Accidents/ 100 M veh.-mi.
Loop Ramp		
a. One Lane	42	355
b. Two Lanes	40	485
Outer Connector		
a. One Lane	22	361
b. Two Lanes	66	171
Direct or Semi-Direct Ramp		
a. One Lane	11	98
b. Two Lanes	21	164

The rates shown in Figure F-6 should be interpreted with caution. The number of units for any category is relatively small, particularly in view of the possible double counting involved in the unit tabulation. The small number of units for each category probably explains the apparently irrational relationships among the accident rates.

For example, one would not expect that one-lane outer connections would be much more hazardous than one-lane direct or semi-direct ramps; yet the rates indicate that the outer connection ramps are four times more dangerous. Further, there is no apparent explanation for the indication that one-lane outer connections are twice as dangerous as two-lane outer connections.

For these reasons, no specific conclusions are drawn from the data in Table F-6.

APPENDIX G

EXIT TERMINAL CASE STUDY

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APPENDIX G: EXIT TERMINAL CASE STUDY

The following critical analysis of the exit terminals of an existing major interchange provides an example of how the design recommendations and conditions of the previous chapters can be applied to analyze or critique an existing design.

The data used here were collected during visits to the interchange site, the state highway district office, and the office of the consulting engineers who designed the interchange.

Site Description

The major interchange chosen for analysis is the interchange between Interstate 80 (Legislative Route 1009) and Interstate 81 (Legislative Route 1005) lying in a rural area of Luzerne County, Pennsylvania. Interstate 80 is the major east-west route across northern Pennsylvania, and Interstate 81 is a major north-south route in the eastern half of the state. The major destinations from the interchange are Scranton and Wilkes Barre to the north, Harrisburg to the south, Bloomsburg to the west, and Stroudsburg to the east.

A schematic of the interchange is shown in Figure G-1. The interchange is relatively new, having been designed in 1963. Basically, the interchange can be described as being a modified cloverleaf. Three of the four left turn movements are accommodated by loop ramps. The remaining left turn, from east to north, is served by a left-hand exit. The design speed for both freeways is 70 miles per hour, and the posted speed limits are 65 miles per hour. Both freeways are four-lane facilities, and all ramps are one lane wide.

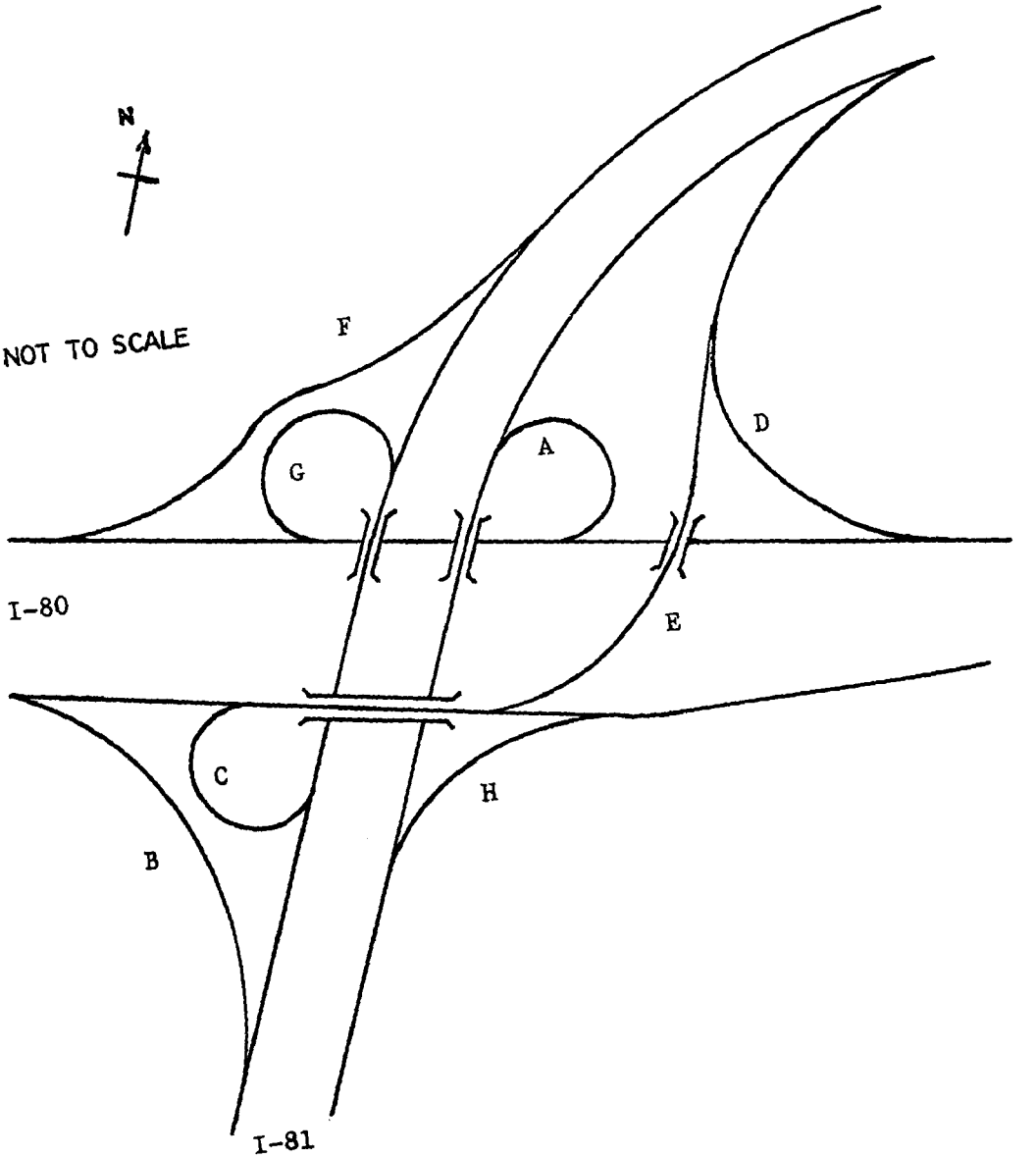

Figure G-1. Schematic of the Interstate 80 and 81 Interchange

The 1975 projected ADT figures which were used to design the interchange are shown in Figure G-2. Using the appropriate methods contained in the Highway Capacity Manual (1965), the thirtieth highest hourly volume was estimated for each traffic movement. These estimates are the numbers in parentheses in Figure G-2.

Critical Analysis of the Existing Design Features

Deceleration Lane Shape

Either type of deceleration lane, parallel or tapered, will work adequately if designed properly. However, with normal freeway volumes and geometry, a tapered deceleration lane is regarded as the optimal design because drivers will utilize the lane more effectively since the taper conforms to the path that they desire to follow. Therefore, the amount of unused pavement area is minimized. Parallel deceleration lanes are recommended where volumes are high or where the geometrics of the exit are less than ideal. The primary advantage of a parallel lane is the target value the "stub" provides.

Table G-1 presents data on the length and shape of each deceleration lane for the interchange under study. Both types of deceleration lane have been used in the interchange.

A parallel deceleration lane should be used where sight distance to the exit gore is restricted by either horizontal or vertical curvature. The sight distances to the exit gores of this interchange are all adequate except on the approach to Ramp H. Here, as is shown in Figure G-3, the mainline roadway is curving to the right such that the exit ramp gore is hidden from the view of the approaching driver by a side slope. Good usage has been made of the target value provided by a parallel lane. Despite

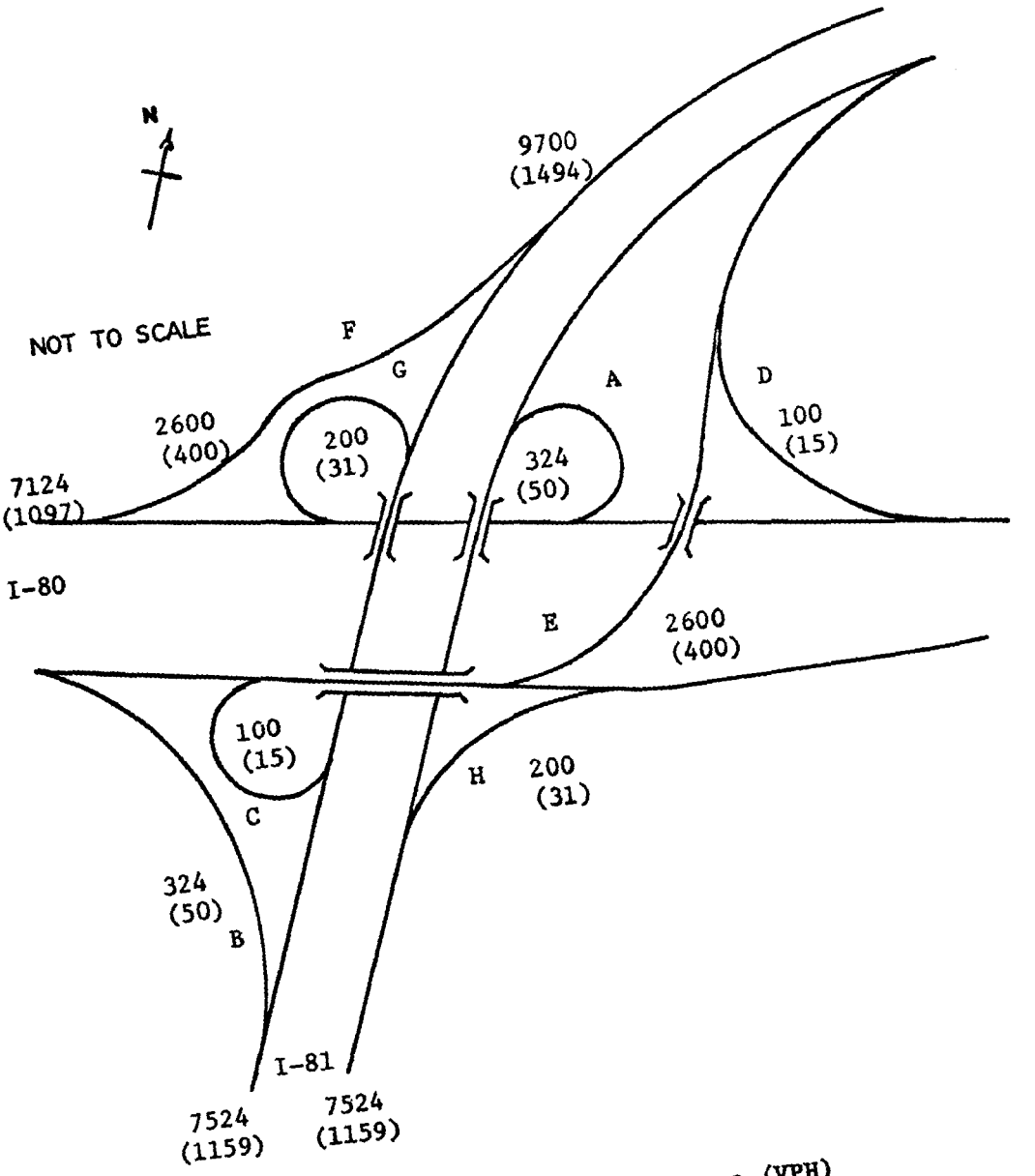

Figure G-2. Projected Traffic Volumes (VPH)

TABLE G-1

DECELERATION LANE SHAPE AND LENGTH

Exit Ramp Terminal	Deceleration Lane Shape	Ramp Design Speed (mph)	Posted Ramp Speed (mph)	Deceleration Lane Length (ft.)	AASHO Recommended Minimum Length (ft.)
A	Tapered	32	25	413	550
B	Tapered	50	40	486	425
C	Parallel auxiliary lane	30	25	1250	550
D	Tapered	30	25	501	550
E	Parallel	60	40	1045	425
F	Parallel	50	40	1045	425
G	Parallel auxiliary lane	25	25	785	550
H	Parallel	45	40	1052	425

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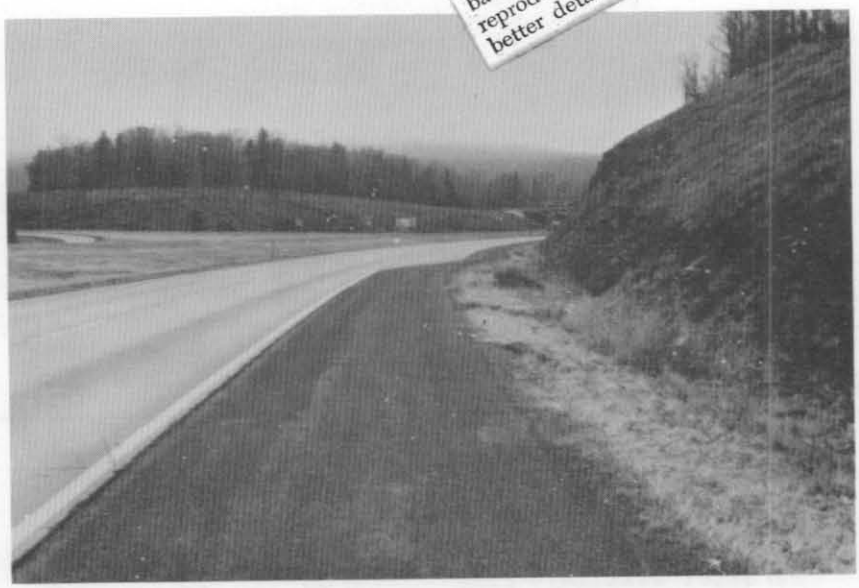

Figure G-3. Northbound Approach to Exit Ramp H

the fact that the exit gore cannot be seen, the abrupt beginning of the parallel lane gives the approaching motorist a definite cue of the impending exit.

A parallel deceleration lane should also be employed if an exit departs tangentially from the mainline. As is shown in Figure G-4, Ramp F, although not tangential, departs on the outside of Interstate 81 which curves to the left in the vicinity of the exit terminal. Here a full width parallel deceleration lane has been provided to better define the downstream roadway geometry and give the driver an unmistakable cue that an exit ramp is ahead. In this situation, a parallel speed-change lane is more effective in guiding drivers along their proper course than a tapered design.

A parallel auxiliary lane has also been rightly provided adjacent to Interstate 81 between the entrance terminal of Ramp G and the exit terminal of Ramp C, and between the entrance terminal of Ramp A and the exit terminal of Ramp G on Interstate 80. The additional lane improves weaving operations between the terminals of the two loop ramps without the addition of collector-distributor roads by providing extra pavement width in which to accomplish the necessary lane changes.

The recommendations in the main report suggest that a parallel deceleration lane should be used if ramp curvature is such that the off-ramp cannot be safely negotiated at a speed of 40 miles per hour or more. With a tapered type of design a driver does not have an auxiliary full width lane for the entire deceleration distance. Consequently, it is more likely that some braking will occur while a portion of the exiting vehicle is still in the freeway mainline.

Figure G-4. Southbound Approach to Ramp F

The speed at which each ramp can be safely negotiated as well as the deceleration lane shape and length for each exit terminal are summarized in Table G-1. An examination of the Table indicates that Ramps A and D, as shown in Figures G-5 and G-6 respectively, are the only terminals which employ a tapered deceleration lane for ramp speeds of less than 40 miles per hour. In addition, the Table indicates that both of these deceleration lanes have shorter lengths than the minimums recommended by the AASHO "Blue Book" (1965) and "Red Book" (1957). Therefore, it is theorized that these terminals will probably experience operational problems. Indeed, the run-over post delineators shown in the lower portion of Figure G-6 indicate that some difficulties have already arisen at the exit terminal for Ramp D. Therefore, it is recommended that the deceleration lanes of Ramps A and D should be lengthened minimums of 137 and 49 feet respectively. Furthermore, along with the length extensions, the shape of the speed-change lanes should be altered to provide a parallel-type design.

Left-Hand Exit

As is seen in Figure G-2, the interchange under study incorporates a left-hand exit from the eastbound roadway of Interstate 80 to serve traffic which desires to go northbound on Interstate 81.

In general, the design community is strongly opposed to the use of left-hand exits since operational problems are unavoidable even with well-designed left-hand facilities. Nevertheless, left-hand exits cannot be excluded from the realm of possible design alternatives because in some circumstances a left-hand exit may be the least objectionable design.

At this interchange, the primary consideration which justifies the use of a left-hand exit is the relatively large left turn volume.

Figure G-5. Existing Deceleration Lane for Ramp A

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better detail.

Figure G-6. Existing Deceleration Lane for Ramp D

Figure G-2 shows that approximately 36% of the total volume approaching the interchange from the west desires to turn left. In the preceding chapters, it is reported that approximately 75% of the design engineers would "occasionally" use a left-hand diverging configuration if the left turn volume is 50% of the total approach volume. If the left turn volume is 30% of the approach volume, only one-third of the engineers surveyed would possibly employ a divergence from the left side of the freeway. Most of the engineers indicated they would design the left-turning facility as a major fork.

Considering absolute volume alone, a loop ramp could have been used to accommodate the necessary left turn. Apparently the designers of this interchange believed that the relatively high left turning volume should be handled in a directional manner. A semi-directional ramp departing from the right-hand side of the freeway could adequately handle the projected volume without the inherent difficulties of a left-side facility. But with the three other quadrants of the interchange having a cloverleaf configuration, the use of a semi-directional ramp requires that many additional structures be built. Therefore, only a loop ramp or a left-hand facility could be economically justified.

With a left-hand exit, diverging drivers are required to change lanes to the left in order to position their vehicles in the left-hand, high-speed freeway lane in advance of the exit gore. For safe freeway operations the gradient of the freeway in advance of the exit gore should be such that diverging trucks can make the required lane changes at a high speed. Furthermore, since the exiting trucks will be traveling in the left-hand freeway lane, the approach grades should not be such that these trucks would be required to travel at speeds lower than those of passenger vehicles.

Figure G-7 shows the horizontal and vertical alignment of Interstate 80 in the vicinity of the left-hand exit. The gore of the left-hand exit is preceded by 1,594 feet of -1.3% grade. In advance of this, a 5,800 foot, + 2.0% grade exists. The first directional sign which notifies motorists of the left-hand exit is 1.4 miles upstream of the left-hand exit. Therefore, the majority of the required lane changes will take place on the +2.0% grade. The "Blue Book" (1965) specifies that the critical length of +2.0% grade for a 15 mile per hour speed reduction is 3,000 feet. Since the length of +2.0% grade in advance of the exit is greater than 3,000 feet, trucks will be traveling at speeds significantly lower than passenger vehicles as they approach the left-hand exit. Therefore, the possibility of rear-end collisions is great. Recent accident records do not indicate that this approach is hazardous, but this may be due to the low volumes which are presently using the facility. As volumes increase, problems may very well arise.

Adequate sight distance is essential if a left-hand exit is to operate safely. The workshop discussions and a review of research literature indicate that an approaching driver should be able to clearly see the left-hand off-ramp geometry from a point at least one-quarter mile upstream from the beginning of the deceleration lane. When the driver is even with the beginning of the deceleration lane, researchers at Northwestern University (1969) believe that he should be able to see the mainline pavement for a distance of 1,000 feet beyond the gore and the ramp pavement for a distance of 500 feet beyond the gore.

An investigation of Figure G-7 reveals that, by these standards, sight distance is clearly inadequate at this left-hand exit. Although a driver can see the mainline pavement for 1,000 feet beyond the gore and the ramp pavement for 500 feet beyond the gore when at the beginning

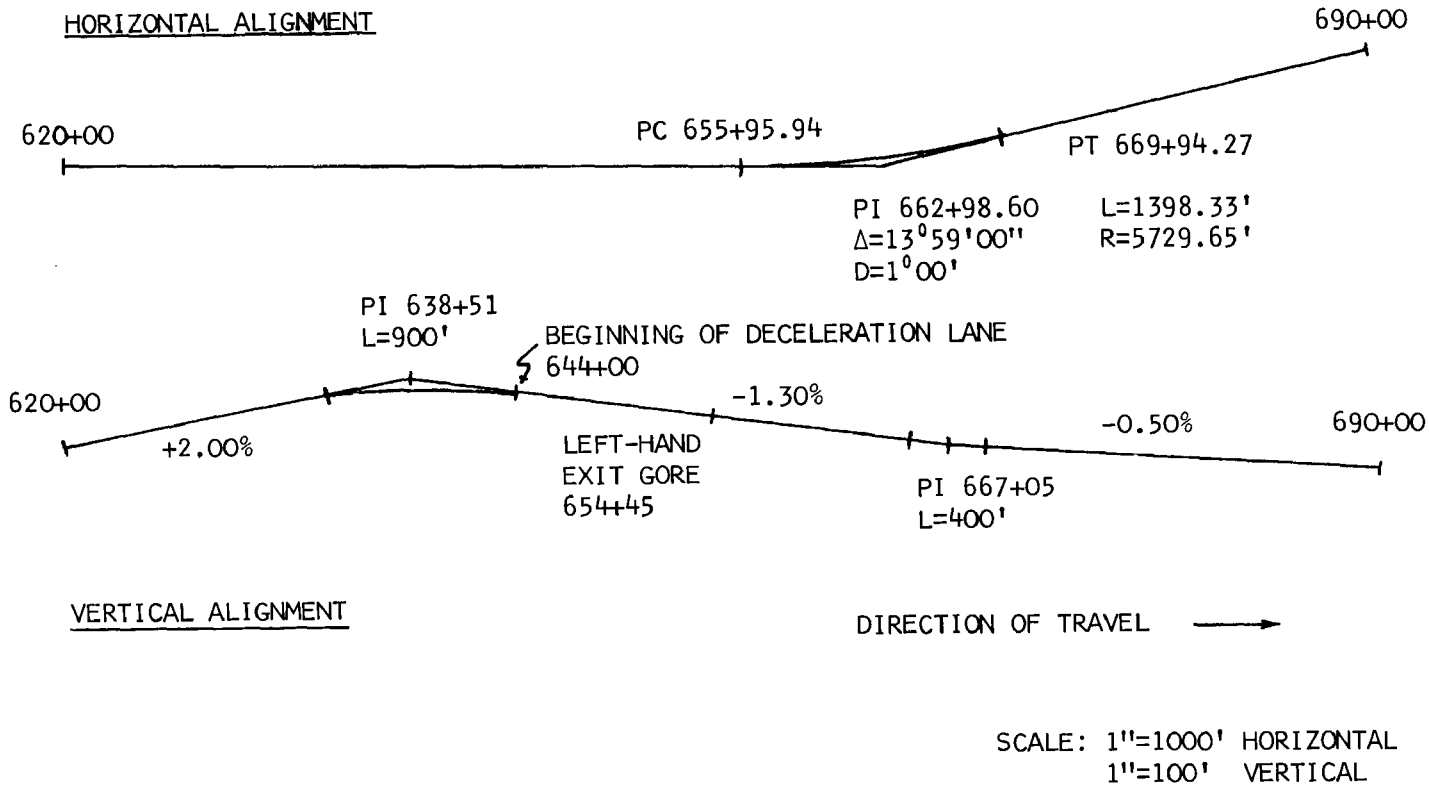

Figure G-7. Alignment for I80 Eastbound

of the deceleration lane, the requirement that he be able to see the exit gore from a distance of one-quarter mile upstream of the beginning of the deceleration lane is not satisfied. As a result of the 900 foot crest vertical curve in advance of the speed-change lane, an approaching driver can see the ramp gore from a distance of only 550 feet in advance of the start of the deceleration lane.

Signing is the most critical factor influencing left-hand exit ramp operations. At this interchange signing is even more important since the sight distance to the ramp in question does not satisfy the minimum standards recommended in the research literature.

The present signing sequence for the left-hand exit is shown in Figure G-8. Figure G-9 shows the signing plan that has been recommended for left-hand exits by Northwestern University (1969). The existing and the recommended signing plans generally conform up to the one-quarter mile point. Here the researchers at Northwestern recommended a sign which gives the drivers a further indication of the left-hand exit and also informs them of the speed at which they should exit onto the deceleration lane. The exiting speed indicated on the sign is approximately the same as the posted speed of the highway. This sign is provided in order to encourage the exiting drivers to do all of their decelerating while in the deceleration lane and not while in the high-speed, left-hand freeway lane. After the driver is in the deceleration lane, the advisory signs at the ramp gore will inform him of the deceleration which is required to safely negotiate the ramp curve. Therefore, it is recommended that the signs shown in Figure G-10 be provided along the left-hand side of Interstate 80 one-quarter mile upstream of the off-ramp nose. These signs will give drivers an additional indication of the left-hand exit and also will encourage high-speed exits.

A. 2.4 miles from the left-hand exit gore

Figure G-8. The Existing Signing for the Left-Hand Exit

B. 1.4 miles from the left-hand exit gore

C. At Ramp B or 0.4 mile from the left-hand exit gore

Figure G-8. (Continued)

D. Left-hand exit gore

Figure G-8. (Continued)

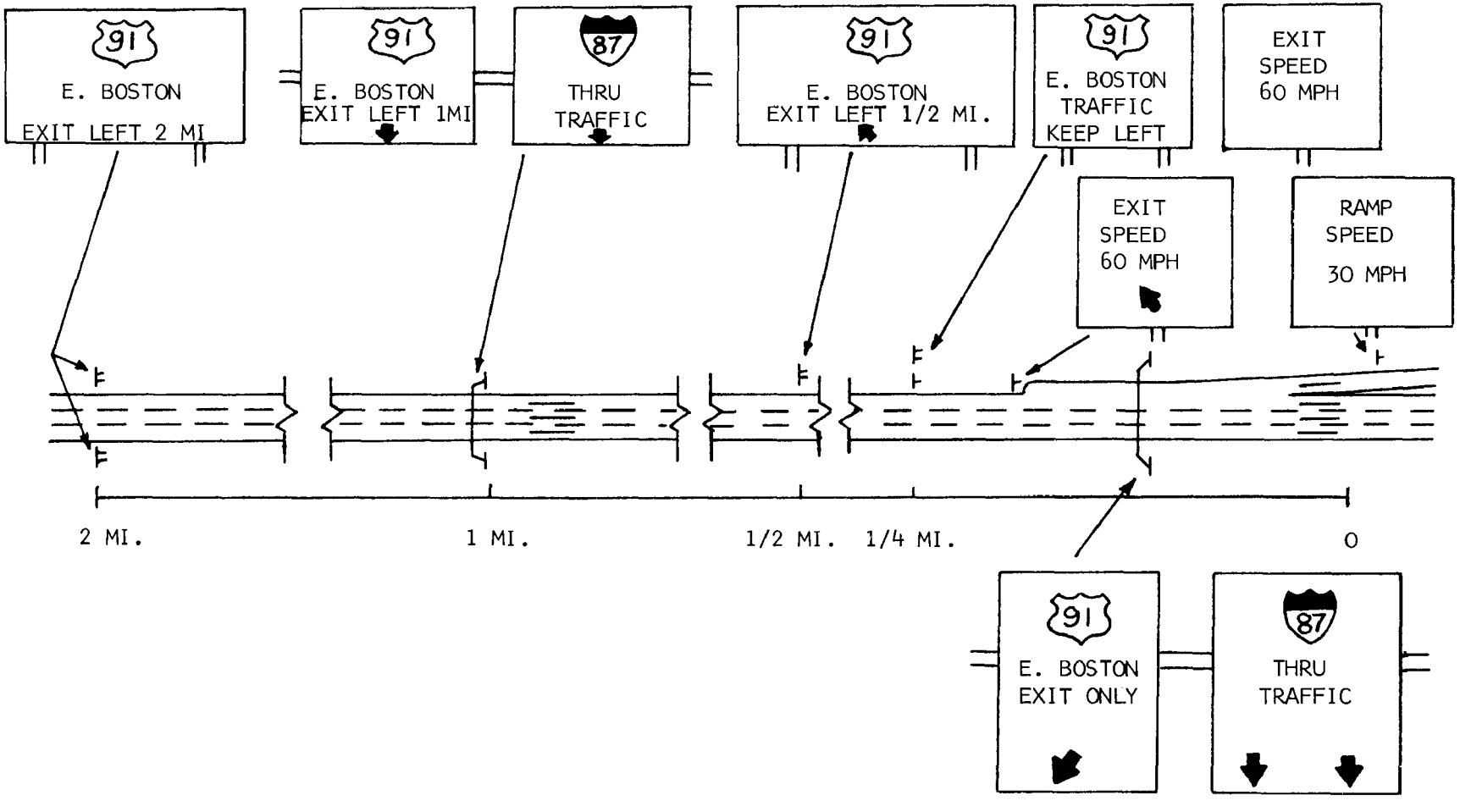

Figure G-9. Suggested Signing Plan for Left-Hand Exits

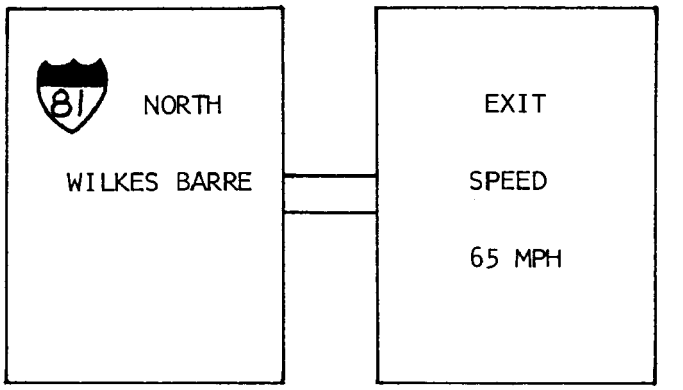

Figure G-10. Recommended Supplementary Signs

Furthermore, it is recommended that yellow "EXIT ONLY" or "THIS LANE MUST EXIT" tabs be placed on the overhead sign at the exit gore so that drivers will not mistake the relatively long, parallel deceleration lane which has been provided in advance of the left-hand exit for an added through lane.

The left-hand exit ramp terminal itself is relatively well-designed. The vast majority of design engineers believe that a parallel deceleration lane should be provided at left-hand exits because of the target value it provides. The abrupt full lane width will alert drivers of the impending exit and will inform them of the parallel lane which has been provided for their use in diverging. Such a deceleration lane configuration has been provided at this left-hand exit.

The delineation of both the ramp and the mainline roadway is excellent. Currently, the terminal area is not lighted. In the future, lighting should be considered if operations in the vicinity of the exit terminal become hazardous.

The recovery taper at the left-hand exit ramp nose is 125 feet long. All other exit terminals in the interchange have 150 foot tapers. It appears somewhat illogical to provide a shorter recovery area at the gore of the left-hand exit since this is probably the one location where the most driver confusion will result. Using the general procedure outlined in the AASHO "Blue Book" (1965), it was found that a 150 foot taper is recommended for any exit ramp departing from a 70 mile per hour approach roadway with a 10 foot nose offset. Northwestern University (1969) has suggested that a minimum taper length of 250 feet should be used for left-hand exits. Therefore, it is evident that the present recovery area is inadequate, and thus it should be extended to provide a minimum taper length of 250 feet.

Single Versus Double Exit Design

Research literature and the results of the workshop sessions indicate that the majority of design engineers believe that single exit interchange should be used as often as possible for freeway-to-freeway interchanges. Since the majority of interchanges along a route are likely to be single exit, diamond-type designs, a single exit interchange generally assures uniform exiting patterns. With a single exit pattern signing is simplified since points of decision are separated. Furthermore, with cloverleaf-type designs, the provision of single exits with the concomitant collector-distributor roadways also removes weaving areas from the mainline roadway.

Despite the operational advantages which can be gained by a single exit design, experienced designers recognize that such configurations cannot always be economically justified due to the additional costs which would be required in order to furnish the collector-distributor roads and the wider and/or additional structures they would require. Designs with two exits on an approach can operate adequately. Therefore, the provision of a single exit can be viewed as a desirable, but not essential, operational refinement.

The interchange under study is a conventional cloverleaf with the exception of a left-hand exit on one approach. As such, each approach roadway has two exits from the mainline. Almost all of the interchanges located along each approach to the interchange under study possess single exits from the mainline. This suggests that the Interstate 80 and 81 interchange should likewise be of the single exit type so that the design would conform to the pattern that drivers would expect to encounter.

Nevertheless, the interchange was designed as a two-exit conventional cloverleaf with a left-hand exit on one approach. With the left-hand exit,

single exits could be provided only on the southbound and westbound approaches. Considering the very low projected traffic volumes and the rural location, a basic cloverleaf configuration is the least expensive configuration which could be employed. As a result of the low volumes, each of the exit terminals of the present two-exit design is operating at level of service A. Thus, the author believes that the designers were justified in employing successive exits on each approach.

If successive exits are used, a driver must be given adequate distance to make decisions and maneuver between the two exit ramp gores. Table G-2 compares the distances between the exit gores in the interchange with those which have been recommended as a result of this study. The recommended distances in Table G-2 were chosen because at least 50% of the design experts at the workshops endorsed a distance which fell within the specified range. The Table indicates that the distances between the two exits on each approach to the interchange far exceed those recommended.

Suggested Design Improvements

A review of relevant research literature and the results of a study of the opinions of a representative sample of design experts indicate that the following improvements should be made to improve the safety and operations at the exit terminals of this major interchange:

1. The deceleration lanes for Ramps A and D should be lengthened minimums of 137 and 49 feet respectively and should be changed to a parallel-type design.
2. A supplementary sign, with the message shown in Figure G-10, should be provided along the left-hand side of the roadway one-quarter mile in advance of the left-hand exit gore.

TABLE G-2

A COMPARISON OF THE EXISTING AND RECOMMENDED

DISTANCES BETWEEN SUCCESSIVE EXITS

Direction	Exit Terminal	Existing Distance (ft.)	Recommended Distance (ft.)	
			Minimum	Desirable
Eastbound	B-E	2,095	500-800	1,000-1,200
Westbound	D-G	2,708	800-1,000	1,000-1,500
Northbound	H-A	2,242	800-1,000	1,000-1,500
Southbound	F-C	2,260	800-1,000	1,000-1,500

3. Yellow "EXIT ONLY" or "THIS LANE MUST EXIT" tabs should be added to the overhead sign at the left-hand exit gore.
4. The tapered recovery area at the left-hand exit gore should be lengthened to a minimum of 250 feet.