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16. Abstract

Improvements of freeway-freeway interchanges in Texas have been initiated through re-design and re-construction. However, little documentation or definitive guidelines exist to assist in these efforts. Alternative designs need to be considered and evaluated for freeway-freeway interchanges. The operational impacts of alternative designs which will best accommodate current and future freeway demands must be evaluated. This research study focuses on the interaction between different freeway-freeway interchange designs and the resulting effects on operations and safety.

The methodology followed to evaluate the operational effects of freeway-freeway interchange design consisted of four steps: 1) Determination of operational effects as related to different interchange types; 2) Assessments of the level and patterns of interchange accidents; 3) Analysis of geometric elements of interchanges; and 4) Case study examination of specific alternative interchange designs and improvements.

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OPERATIONAL EVALUATION OF EFFECTS RESULTING FROM FREEWAY - FREEWAY INTERCHANGE GEOMETRICS

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Research Report 1232

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February 1992

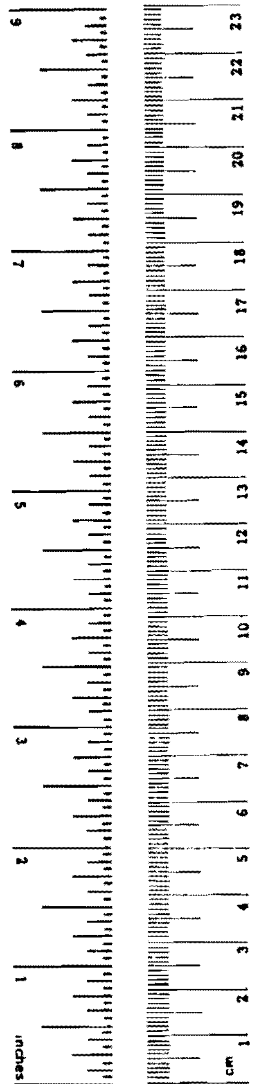
METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	2.54	millimetres	mm
ft	feet	0.3048	metres	m
yd	yards	0.914	metres	m
mi	miles	1.61	kilometres	km
	AREA			
in^2	square inches	645.2	millimetres squared	mm^2
ft^2	square feet	0.0929	metres squared	m^2
yd^2	square yards	0.836	metres squared	m^2
mi^2	square miles	2.59	kilometres squared	km^2
ac	acres	0.395	hectares	ha
	MASS (weight)			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
	VOLUME			
fl oz	fluid ounces	29.57	millilitres	mL
gal	gallons	3.785	litres	L
ft^3	cubic feet	0.0328	metres cubed	m^3
yd^3	cubic yards	0.0765	metres cubed	m^3

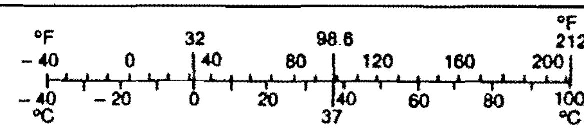
NOTE: Volumes greater than 1000 L shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C



APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimetres	0.039	inches	in
m	metres	3.28	feet	ft
m	metres	1.09	yards	yd
km	kilometres	0.621	miles	mi
	AREA			
mm^2	millimetres squared	0.0016	square inches	in^2
m^2	metres squared	10.764	square feet	ft^2
km^2	kilometres squared	0.39	square miles	mi^2
ha	hectores (10 000 m^2)	2.53	acres	ac
	MASS (weight)			
g	grams	0.0353	ounces	oz
kg	kilograms	2.205	pounds	lb
Mg	megagrams (1 000 kg)	1.103	short tons	T
	VOLUME			
mL	millilitres	0.034	fluid ounces	fl oz
L	litres	0.264	gallons	gal
m^3	metres cubed	35.315	cubic feet	ft^3
m^3	metres cubed	1.308	cubic yards	yd^3
	TEMPERATURE (exact)			
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



These factors conform to the requirement of FHWA Order 5190.1A.

* SI is the symbol for the International System of Measurements

EXECUTIVE SUMMARY

Since 1928, engineers have developed and used numerous schemes and techniques to effectively and safely manage conflicting traffic movements within interchanges. The safety and operational limitations of many freeway-freeway interchange designs were not immediately realized when these designs were initially implemented twenty to forty years ago. Specific safety and operational problems manifested as the annual daily traffic per lane volumes in these interchanges approached capacity. As the capacity was reached, these interchange designs exhibited both operational and safety characteristics which were undesirable.

The methodology followed to evaluate the operational effects of freeway-freeway interchange design consisted of four steps: 1) Determination of operational effects as related to different interchange types; 2) Assessments of the level and patterns of interchange accidents; 3) Analysis of geometric elements of interchanges; and 4) Case study examination of specific alternative interchange designs and improvements.

Several case studies were selected for detailed examination. Various alternative designs were developed for each site and the operation evaluated. The alternative designs range from the placement of additional travel lanes to implementing separate "by-pass" facilities that remove the through traffic from the interchange. Design elements specifically analyzed included: 1) Elimination of left-hand entrances and exits; 2) Elimination of interior

lane drops or "chicken" merges; and 3) Ramp spacing and location within the interchange. Computer modelling techniques were used on the alternative designs to quantify the improvement or deterioration of interchange operation. These simulations and comparisons were then employed to allow development of guidelines for the implementation of various freeway-freeway interchange designs.

ABSTRACT

Improvements of freeway-freeway interchanges in Texas have been initiated through re-design and re-construction. However, little documentation or definitive guidelines exist to assist in these efforts. Alternative designs need to be considered and evaluated for freeway-freeway interchanges. The operational impacts of alternative designs which will best accommodate current and future freeway demands must be evaluated. This research study focuses on the interaction between different freeway-freeway interchange designs and the resulting effects on operations and safety.

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DISCLAIMER

The contents of this report reflect the views of the authors who are responsible for the opinions, findings, and conclusions presented herein. The contents do not necessarily reflect the official views or policies of the Texas State Department of Highways and Public

Transportation, the U.S. Department of Transportation, or the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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INTRODUCTION

The first modern interchange in the U.S. was constructed and opened in 1928 at Woodbridge, N.J. (1). This grade-separated, cloverleaf interchange was implemented to permit all traffic movements, including left-turns to operate without interference from other movements within the intersection. Since 1928, engineers have developed and used numerous schemes and techniques to effectively and safely manage conflicting traffic movements within interchanges.

The majority of the existing freeway system in Texas is a part of the 42,000-mile National System of Interstate and Defense Highways approved by Congress in 1944. The first major funding directed toward improving urban system freeways through re-construction and rehabilitation was provided by the Federal Aid Highway Act of 1970. The construction and improvement of the freeway system necessitated the need for highly efficient and safe means of interaction between two or more freeways. To provide this interaction, several types of freeway-freeway interchanges were designed and implemented. The primary purpose of each interchange type was to establish a design which was most appropriate for the individual site conditions. These designs focused primarily on capacity, ease of operation, safety, uniformity and flexibility of operation, and coordination with the existing freeway system.

The safety and operational limitations of various freeway-freeway interchange designs were not immediately realized when these designs were initially implemented twenty to forty years ago. Safety and operational problems grew as the annual daily traffic per lane volumes in these interchanges approached capacity. As the capacity of these facilities was reached, both operational and safety characteristics were identified that suggested further evaluation of alternative freeway-freeway interchange designs was warranted.

Study Objectives

Major improvements of freeway-freeway interchanges in Texas have been initiated through re-design and re-construction. However, little documentation or definitive guidelines exist to assist in these efforts. Alternatives need to be evaluated for freeway-freeway interchanges. The operational impacts of alternative designs under varying traffic demands should be evaluated. This research study focuses on the interaction between alternative types of freeway-freeway interchange designs and the potential effects on operations and safety.

Evaluation Methodology

The methodology followed to evaluate the operational effects of freeway-freeway interchange design consisted of four steps: 1) Determination of operational effects as related to different interchange types; 2) Assessments of the level and patterns of interchange accidents; 3) Analysis of geometric elements of interchanges; and 4) Case study examination of specific alternative interchange designs and improvements.

The first step in the determination of the operational effects was to conduct an inventory of existing freeway-freeway interchange configurations in Texas. Once the various types of interchanges had been identified, geometric alignment and operational data were collected for each interchange.

Historical accident information was then obtained for each inventoried freeway-freeway interchange. The level and pattern of accidents were assessed relative to volume and interchange type. Accident rates were established for each interchange and then compared to overall freeway accident rates. Mainlane freeway accident rates were established independent of accidents occurring within the functional area of the interchanges.

The analysis of freeway-freeway interchange elements was conducted utilizing computer simulation techniques to evaluate the level-of-service, capacity, and delay for these elements. These factors were established for future comparisons between existing and alternative freeway-freeway interchange designs and resulting operations.

Finally, several case studies were selected from the interchanges on the inventoried list for detailed examination. Various alternative designs were developed for each site and the operation of each was evaluated. Design elements specifically analyzed included: 1) elimination of left-hand entrances and exits; 2) elimination of interior lane drops at merge terminals; and 3) ramp spacing and location within the interchange. Modelling techniques were used on the alternative designs to quantify the relative improvement or deterioration of interchange operation. These simulations and comparisons were then employed to allow development of guidelines for the implementation of various freeway-freeway interchange designs.

DATA DOCUMENTATION

Specific information was assimilated to determine the operational effects resulting from geometric features and elements associated with various types of freeway-freeway interchanges. The data collected were as follows:

- Inventory list by location and type of existing freeway-freeway interchange configurations;
- Interchange accident documentation;
- Geometric alignment documentation (cross-section, vertical, horizontal); and
- Traffic volumes and capacities on all approaches and connections.

These data were used to categorize the freeway-freeway interchanges and to determine operational characteristics for each interchange type. These data were also incorporated into subsequent accident assessment and geometric analyses.

Freeway-Freeway Interchange Inventory

The freeway-freeway interchanges studied are located in the four largest metropolitan areas in Texas -- Dallas, Fort Worth, Houston, and San Antonio. Fifty-four interchanges in these four metropolitan areas were identified for this study. A study team visited each site after reviewing aerial photographs to determine the existing configuration and interchange classification. The interchanges which were selected are listed in Table 1. The portion(s) of the study in which individual interchanges were used is also indicated.

Interchanges were divided into four-leg and three-leg facilities. These interchanges were then identified according to subcategories. Four-leg interchanges were divided into four subcategories: directional, semi-directional, cloverleaf, and partial cloverleaf interchanges (Figure 1). The four-leg directional interchanges were further categorized as having either one- or two-exit connector ramps. The three-leg interchanges were divided into two separate categories: directional and semi-directional (Figure 2). Both the operational characteristics and historical accident data were then used to evaluate each interchange.

Interchange Accident Files

Historical accident data were obtained for each of the inventoried freeway-freeway interchanges. Three-year accident histories were obtained from the Texas Department of Public Safety (DPS) (2) for the years 1987-1989. The accident data identified the direction of travel, approximate location with respect to highway mile post, and the type of facility on which the accident(s) occurred. This information allowed accidents to be assigned to the various interchange elements (connector ramps, mainlanes, etc.) using the SDHPT's Roadway Inventory File (RI-1 logs). Schematic drawings of each interchange were

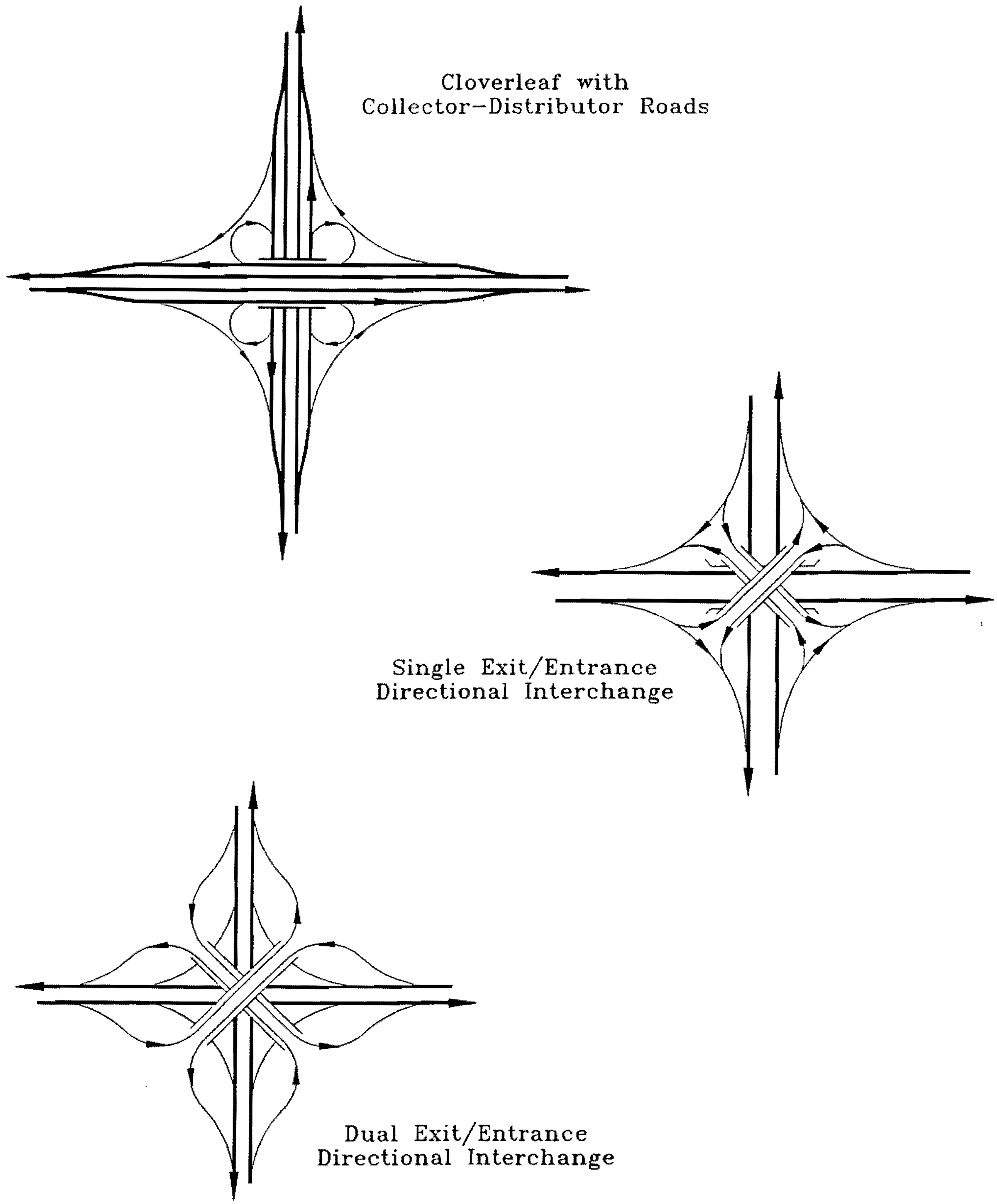
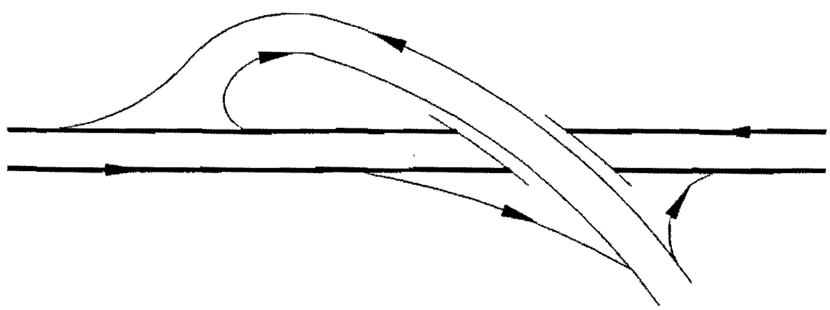
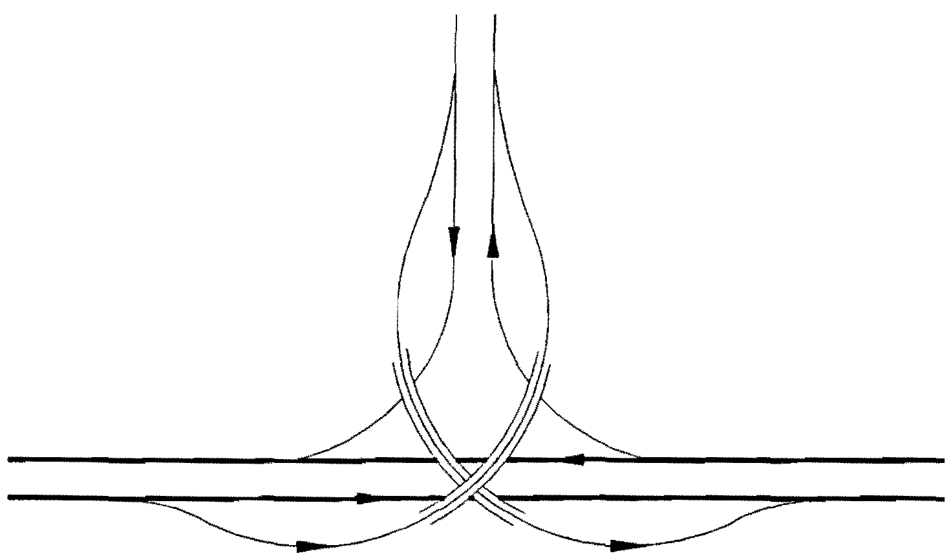


Figure 1. Typical Four-Leg Interchanges (7)



Trumpet



3-Leg Directional Interchange

Figure 2. Typical Three-Leg Interchanges (7)

Table 1. Freeway-Freeway Interchange Inventory

Location	Designation Code	Interchange Location	Interchange Type	Accident Experience Assessment	Operational/ Geometric Analysis	Case Studies
Dallas	D-1	I-20 & Spur 408	Three leg-T Directional	*	*	
Dallas	D-2	I-20 & US 67	Four leg-two ent/ex Directional	*	*	
Dallas	D-3	I-20/635 & I-35E	Four leg-two ent/ex Directional	*	*	
Dallas	D-4	I-20 & I-45	Four leg-two ent/ex Directional	*	*	
Dallas	D-5	I-20/635 & US 175	Four leg-two ent/ex Directional	*	*	
Dallas	D-6	I-635/I-20 Split	Three leg-Y Semi-Directional	*	*	
Dallas	D-7	I-30 & Loop 12(W)	Cloverleaf	*	*	
Dallas	D-8	I-30 & I-35E	Semi-directional		*	
Dallas	D-9	I-30 & I-45	Four leg-one ent/ex Directional	*	*	
Dallas	D-10	I-30/US 80 Split	Three leg-Y Directional		*	
Dallas	D-11	I-30 & I-635	Four leg two-two ent/ex Directional	*	*	
Dallas	D-12	I-35E & Loop 12(S)	Cloverleaf		*	
Dallas	D-13	I-35E & Spur 366	Three leg-trumpet Semi-Directional		*	
Dallas	D-14	US 175 & US 75	Three leg-Y Directional	*	*	
Dallas	D-15	I-635 & US 80	Four leg-two ent/ex Directional	*	*	
Dallas	D-16	I-35E & I-635	Partial Cloverleaf	*	*	
Dallas	D-17	US 72 & I-635	Partial Cloverleaf	*	*	
Dallas	D-18	US 75 & Loop 12(N)	Cloverleaf	*	*	
Dallas	D-19	US 75 & Spur 366	Three leg-trumpet Directional		*	
Dallas	D-20	Spur 482 & SH 183	Three leg-Y Directional		*	
Dallas	D-21	SH 114 & Spur 482	Partial Cloverleaf		*	
Dallas	D-22	US 80 & Loop 12(W)	Cloverleaf		*	
Dallas	D-23	US 67 & Loop 12(S)	Cloverleaf		*	
Dallas	D-24	Loop 12(S) & Spur 408	Three leg-Y Directional		*	
Dallas	D-25	Loop 12 & SH 183	Partial Cloverleaf	*	*	
Dallas	D-26	Loop 12 & SH 114	Diamond		*	
Dallas	D-27	SH 183 & SH 114	Three leg-Y		*	
Dallas	D-28	I-45 & Loop 12S	Cloverleaf	*	*	
Ft. Worth	FW-1	US 287 & I-820	Three leg-Y Directional		*	
Ft. Worth	FW-2	I-20/I-820 & US 287	Three leg-Y Directional		*	
Ft. Worth	FW-3	I-20 & US 287	Three leg-Y Directional		*	
Houston	H-1	I-45N & I-610 N	Four leg Directional	*	*	
Houston	H-2	I-10W & I-610	Four leg-one ent/ex Directional	*	*	
Houston	H-3	US 59 & I-610W	Four leg-one ent/ex Directional	*	*	*
Houston	H-4	SH 288 & I-610S	Four leg-one ent/ex Directional	*	*	
Houston	H-5	I-45S & I-610	Four leg Directional	*	*	*
Houston	H-6	SH 225 & I-610E	Four leg-two ent/ex Directional	*	*	
Houston	H-7	I-10E & US 59N	Four leg-one ent/ex Semi-Directional	*	*	*
Houston	H-8	I-45 & I-10	Four leg Directional	*	*	
Houston	H-9	I-45 & US 59	Four leg-two ent/ex Directional	*	*	
Houston	H-10	US 59 & SH 288	Three leg-Y Directional		*	
Houston	H-12	I-10E & I-610E	Four leg Directional	*	*	
Houston	H-13	US 290 & I-610	Three leg-Y Directional	*	*	*
Houston	H-14	US 59N & I-610	Four leg-two ent/ex Directional	*	*	
San Antonio	SA-1	I-410 & I-37/US 181	Four leg-one ent/ex Directional	*	*	
San Antonio	SA-2	US 281/I-37 & I-35/US 81	Four leg/Semi-Directional		*	
San Antonio	SA-3	Loop 1604 & I-10N	Cloverleaf	*	*	
San Antonio	SA-4	I-35 & US 90	Four leg-one ent/ex Directional	*	*	
San Antonio	SA-5	I-10 & I-410	Cloverleaf	*	*	
San Antonio	SA-6	I-410N & I-35	Three leg-Y		*	
San Antonio	SA-7	Loop 1604NE & I-35	Partial Cloverleaf	*	*	
San Antonio	SA-8	US 90 & I-410W	Cloverleaf	*	*	
San Antonio	SA-9	I-35/US 81 & I-410S/SH 16	Cloverleaf	*	*	
San Antonio	SA-10	I-140E & I-10	Cloverleaf	*	*	
San Antonio	SA-11	I-37 & I-10/US 90	Four leg-one ent/ex Directional	*	*	

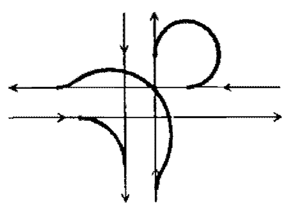
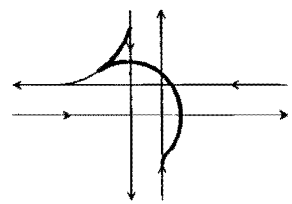
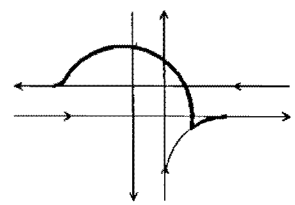
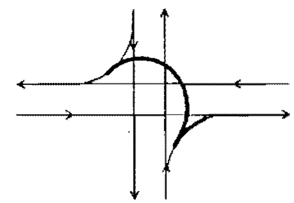
then prepared and coded so that the accidents could be located at specific locations on the different elements of the interchange.

DPS files from Houston provided the most complete accident data. San Antonio and Dallas were not able to provide direction of travel. Lack of this information inhibited the location of accidents on individual ramps or connectors. General associations, however, can be made relative to safety from this data. These evaluations are discussed in a later section of this report.

Geometric and Operational Data

Once the freeway-freeway interchanges were identified and categorized, geometric and operational information was obtained for each study site. Detailed data were available for 46 of the 54 interchanges selected.

Individual connection links within the freeway-freeway interchanges were also identified by three categories. These categories are: mainlanes, exit and entrance ramps, and freeway-freeway connector ramps. The classification scheme used to identify these facilities is shown in Table 2. Both the geometric and operational data were classified and placed in a database to evaluate differences in design, operations, and safety.

Freeway-to-Freeway Connectors	Origin	Freeway		Exit Ramp	
	Destination	Freeway	Entrance Ramp	Freeway	Entrance Ramp
	Example				

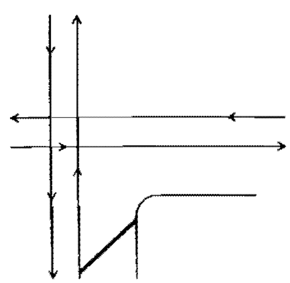
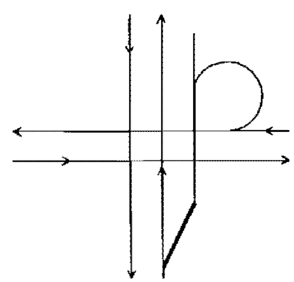
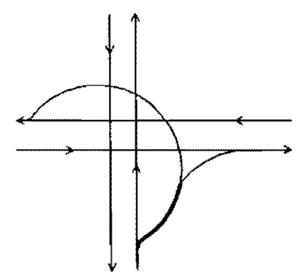
Ramps	Origin	Freeway		
	Destination	Frontage Road	CD Road	Direct Connector
	Example			

Table 2. Freeway-Freeway Interchange Ramp/Connector Classification

The detailed geometric data assimilated included: 1) vertical and horizontal alignments of ramps and mainlanes; 2) degree of curvature; 3) K factors for vertical curves; and 4) ramp configuration data. Descriptive geometry data were obtained from the State Department of Highways and Public Transportation (SDHPT) "as-built" plan sheets. The design speeds were estimated from the geometric specifications using current design criteria as per the 1984 American Association of State Highway and Transportation Officials (AASHTO) Manual A Policy on Geometric Design of Highways and Streets (3).

Geometric data were used to identify critical design elements within each individual freeway-freeway interchange. The geometric data were also used to identify limitations in sight distance caused by either horizontal or vertical alignments. These data were then used in the simulation modeling of interchange operations.

The detailed operational data obtained included volumes for freeway mainlanes, ramps, (including identification of left-hand ramps), direct connections, and configuration of ramp weaving areas. Ramp type and freeway-freeway connector descriptive data were taken from the "as-built" plan sets. This traffic volume data consists of average weekday (Monday through Friday), average daily, and peak-hour traffic volumes. Peak-hour volumes which were not available were estimated using peak-hour factors.

SIMULATION MODELLING TECHNIQUES

The operational analysis of various freeway-freeway geometries was conducted with the aid of computer simulation models. Computer simulation models can be used to evaluate conditions on urban streets, rural roads, and freeway systems. The computer models are valuable tools for simulating complex roadway situations where a number of roadway variables are influencing one another and for investigating traffic operations for various roadway conditions. These advanced computer analysis methods allow the freeway designer to evaluate a number of geometric alternatives. Since many calculations are necessary to predict the traffic operations for each configuration, employment of a computer model can prove to be the most cost effective means of evaluation.

INTRAS Model (4)

Two freeway simulation models were used for the operational analysis. The microscopic freeway simulation model INTRAS was used to simulate different freeway-ramp merge sections. Integrated Traffic Simulation (INTRAS) is a stochastic, microscopic model especially developed for studying freeway incidents. Stochastic models yield outcomes that are not completely predictable for a given set of inputs because they depend upon one or more random variables whose values vary among runs. Microscopic models treat each vehicle as a separate unit. INTRAS is a sophisticated, vehicle specific, time stepping simulation. INTRAS contains several algorithms which mathematically execute complex behavior including car-following, lane-changing, and crash avoidance maneuvers.

A detailed evaluation of complicated and unusual traffic operations of a freeway section or even an entire surrounding roadway network can be simulated. The INTRAS model may be used to simulate operations on basic freeway sections, freeway to freeway connectors, ramps, connecting surface streets or of an urban network. Possible simulation applications include: lane additions and removals, ramp reconfiguration, and curvature and grade changes. INTRAS provides the highest level of detail that can be achieved in simulating traffic behavior on the freeway at the present time. These higher levels of detail were needed for merging and weaving studies, where only microscopic models are capable of realistically modelling the vehicle interactions in these critical areas. For this study, INTRAS was used as a quantitative analysis tool to evaluate entrance and exit ramp configurations, ramp lengths, and ramp merge/diverge characteristics.

The INTRAS model produces many standard and optional outputs. The following lists a majority of the outputs. Summary tables of input parameters are provided for each simulation run. Freeway link statistics include: vehicles input and output, number of lane changes, current content, average content, vehicle miles, vehicle minutes, moving time, delay time, volume, speed and density. Since the introduction of the 1985 Highway Capacity Manual (5), freeway operations have been evaluated quantitatively by density and qualitatively by level of service. Density from the simulation may be converted directly to level of service by using Table 3-1 on page 3-8 of the 1985 HCM.

FREFLO Model (6)

The second phase of the analysis was to evaluate the operational effects that changes in geometry would have on a more global scale (i.e. entire interchanges). The existing conditions were modelled to evaluate the current level of operations associated with the existing geometry of the study interchanges. Simulations were then performed to evaluate the relative change in operations associated with various geometric improvements to these interchanges. Due to the amount of data and size of each network (interchange), the INTRAS model was not used. Instead, FREFLO was chosen to simulate the freeway-freeway interchanges.

FREFLO is a macroscopic, deterministic model that consists of a set of conservation and dynamic speed-density equations. The FREFLO model may be used to simulate uni-directional or bi-directional freeway sections, freeway to freeway connectors, interchanges or complete freeway networks. FREFLO may be used to evaluate freeway operations resulting from lane additions, lane blockages, alternative ramp configurations or changes in demand. For this study, FREFLO was used to model individual freeway to freeway interchanges. Alternative designs were developed and simulated with existing volumes. This procedure allowed a quantitative comparison between existing and proposed configurations.

FREFLO output provides speed, volume, and density information for autos/trucks, buses and carpools on each link. Cumulative freeway statistics for each link are also reported, including vehicle trips, vehicle miles, vehicle minutes, vehicle average speed, person trips, person miles, and person minutes.

ACCIDENT ASSESSMENT

The comparison of different interchanges with respect to safety involves the use of historical accident and volume data. These data can be used to develop accident rates for a given facility. Accident rates, rather than the total number of accidents on a facility, are typically used in safety comparisons. The accident rate associated with a given facility corresponds to the ratio obtained by dividing the total number of accidents at the facility by the total volume that uses the facility. Higher ratios correspond to the higher relative frequency of accidents for the respective traffic volume. This ratio can be used as a relative measure rather than an indicative measure, thereby allowing comparative observations to be made about the accident frequency at different interchanges.

State of the Art

The accident assessment methodology followed in this report was founded on a 1968 research study by Cirillo (7). Cirillo's research presented accident frequencies and rates occurring at specific locations within urban interchanges. Other studies evaluating freeway-freeway interchanges have also used these MOEs to evaluate safety (8, 9). These studies have shown that the highest accident rates occur at entrance and exit ramp terminals. The Lundy (9) research indicated that, of all the types of ramps evaluated, left-hand ramps (both entrance and exit) experienced higher accident rates than did right-hand ramps (Table 3).

Table 3. Accident Rates by Type of Freeway Ramp

	Ramp Type	Accident Rate[1] On Ramp	Off Ramp	On & Off
1.	Diamond ramps	0.40	0.67	0.53
2.	Cloverleaf ramps with collector-distributor roads[2]	0.45	0.62	0.61
3.	Direct connections	0.50	0.91	0.67
4.	Cloverleaf loops with collector-distributor roads[2]	0.38	0.40	0.69
5.	Buttonhook ramps	0.64	0.96	0.80
6.	Loops without collector-distributor roads	0.78	0.88	0.83
7.	Cloverleaf ramps without collector-distributor roads	0.72	0.95	0.84
8.	Trumpet ramps	0.84	0.85	0.85
9.	Scissors ramps	0.88	1.48	1.28
10.	Left-hand exits	0.93	2.19	1.91

[1]Accidents per million vehicles.
[2]Only the On & Off rate includes the accidents occurring on the collector distributor roads.

Source: Lundy, R.A. (9)

The previously referenced research (7, 8, and 9) indicates that entrance ramp terminals exhibit the highest accident rates compared to other interchange elements. This has been attributed largely to the need for the driver to adequately ascertain the availability of a gap, thereby increasing the possibility of a rear-end collision. Improperly designed merge facilities tend to intensify this problem.

Exit ramp terminals have been shown to exhibit the second highest accident rates within interchanges. The majority of accidents at these locations are believed to result when ramp facilities become congested, thereby creating a speed differential in the vicinity of the ramp terminal. The effectiveness of the signing layout in preparing drivers to make a desired movement prior to the exit terminal is an additional factor that contributes to the accident rate at exit terminals (9).

Accident Rate Comparison

The most common method for presenting accident rates uses the ratio of the total number of accidents within the interchange to the total vehicle-miles travelled within the interchange. The outer limits of a given interchange in the preliminary stages of this study were defined as the distance between the first ramp upstream and downstream of the facility. Examination of the accident rates that corresponded to this definition of the interchange limits indicated that accident rates at smaller, more compact interchanges were consistently higher than those associated with larger interchanges. This bias occurred because the ramp-to-ramp length used in the denominator drastically influences accident rates.

The interchange limits were standardized in order to eliminate bias caused by the variability in the size of interchanges. One and one-half mile and two-mile sections centered around the interchange were used. The number of additional ramps located within the corresponding section were recorded in order to ascertain the significance of accidents that occurred outside the limits of the interchange. Additional ramps were defined as those that provide access and egress to the freeway, i.e. entrance and exit ramps, but not part of the basic freeway-freeway interchange. Comparison of these two standardized lengths indicated that the accident rates corresponding to two-mile segments were frequently excessive compared to those associated with the one and one-half mile segments. Further examination of the accident data indicated that the additional length associated with the two mile sections allowed accidents outside the functional area of the smaller, more compact interchanges to drastically affect the accident rates of these interchanges. One and one-half mile segments appeared to more consistently represent the accident rate at all interchanges.

Texas Department of Public Safety data (2) indicates that in 1989 the statewide average accident rate was 1.86 accidents per million vehicle-miles. Analysis of randomly selected freeway sections in major Texas urban areas resulted in ranges with values exceeding the statewide average (see Table 4).

Table 5 illustrates the 1987, 1988, and 1989 accident rates observed at 36 of the inventoried freeway-freeway interchanges where complete file data were available. These accident rates reflect the 1.5 mile interchange length selected for this study. Of the 36 interchanges included in Table 5, eleven have 3-year averages exceeding the statewide average of 1.86 accidents per million vehicle-miles. Comparison of interchange accident rates (Table 5) to urban area average accident rates (Table 4) indicates the following:

- Four interchanges in San Antonio exceed the area average;
- Two interchanges in Dallas exceed the area average; and
- Three interchanges in Houston exceed the area average.

The two highest interchange accident rates for the interchanges studied correspond to the SA-5 and H-3 interchanges. Interchange SA-5 is a partial cloverleaf with two left-hand connector ramps. Interchange H-3 is a four-leg fully directional interchange. These

Table 4. Ranges of Accident Rates in Major Texas Urban Areas (2).

1987 Accident Figures

Urban Area Location	Number of Accidents per year	Primary Length (miles)	Primary Volume (AADT)	Accident Rate	Range Of Rate
San Antonio	25	3.0	18,500	1.23	0.58
San Antonio	337	3.0	117,000	2.63	
San Antonio	16	2.0	37,800	0.58	to
San Antonio	268	3.5	144,000	1.46	
San Antonio	382	3.0	111,000	3.14	3.14
Dallas	323	3.0	159,000	1.86	1.47
Dallas	194	2.0	101,000	2.63	to
Dallas	217	4.0	101,000	1.47	2.63
Houston	932	7.5	168,000	2.03	1.57
Houston	445	3.0	145,000	2.80	to
Houston	1104	6.0	130,000	3.88	3.88
Houston	373	4.5	145,000	1.57	

1988 Accident Figures

Urban Area Location	Number of Accidents per year	Primary Length (miles)	Primary Volume (AADT)	Accident Rate	Range Of Rate
San Antonio	30	3.0	18,500	1.48	0.45
San Antonio	323	3.0	115,300	2.56	
San Antonio	13	2.0	39,750	0.45	to
San Antonio	310	3.5	151,000	1.61	
San Antonio	361	3.0	122,500	2.69	2.69
Dallas	331	3.0	163,500	1.85	1.85
Dallas	182	2.0	48,500	5.15	to
Dallas	255	4.0	48,500	3.61	5.15
Houston	888	7.5	181,000	1.79	1.68
Houston	496	3.0	144,000	3.17	to
Houston	1183	6.0	140,000	3.86	3.86
Houston	395	4.5	143,000	1.68	

1989 Accident Figures

Urban Area Location	Number of Accidents per year	Primary Length (miles)	Primary Volume (AADT)	Accident Rate	Range OF Rate
San Antonio	22	3.0	17,750	1.13	0.55
San Antonio	373	3.0	120,000	3.84	
San Antonio	16	2.0	39,500	0.55	to
San Antonio	373	3.5	156,000	1.87	
San Antonio	393	3.0	120,000	2.99	3.84
Dallas	351	3.0	167,000	1.92	1.53
Dallas	176	2.0	100,500	2.40	to
Dallas	225	4.0	100,500	1.53	2.40
Houston	886	7.5	179,500	1.80	1.80
Houston	474	3.0	145,500	2.98	to
Houston	1152	6.0	149,500	3.52	3.52
Houston	513	4.5	145,500	2.15	

Source: DPS Accident Files (2).

Table 5. Interchange Accident Rates in Accidents per Million Vehicle-Miles.

Urban Area Location	Interchange Description Code	1987 Accident Rate	1988 Accident Rate	1989 Accident Rate	Average 3-year Rate
San Antonio	SA-1	1.80	2.17	2.17	2.05
San Antonio	SA-3	1.08	0.81	0.84	0.91
San Antonio	SA-4	5.10	3.83	2.54	3.82
San Antonio	SA-5	4.62	3.75	4.33	4.23
San Antonio	SA-7	1.36	1.18	1.13	1.22
San Antonio	SA-8	1.63	1.61	1.13	1.46
San Antonio	SA-9	2.14	2.10	1.68	1.97
San Antonio	SA-10	1.07	0.98	1.50	1.18
San Antonio	SA-11	2.21	1.79	1.75	1.92
Dallas	D-1	0.24	0.35	0.41	0.33
Dallas	D-2	0.92	0.70	0.67	0.76
Dallas	D-3	0.59	0.64	0.63	0.62
Dallas	D-4	0.47	0.48	0.51	0.49
Dallas	D-5	0.50	0.31	0.58	0.46
Dallas	D-7	0.24	0.31	0.28	0.28
Dallas	D-9	1.65	1.35	1.29	1.43
Dallas	D-11	1.56	1.21	1.10	1.29
Dallas	D-14	2.12	1.92	2.15	2.06
Dallas	D-15	1.32	1.47	0.80	1.20
Dallas	D-16	0.73	1.07	1.02	0.94
Dallas	D-17	1.06	1.14	1.05	1.08
Dallas	D-18	3.12	2.89	2.84	2.95
Dallas	D-25	1.97	1.99	1.54	1.83
Dallas	D-28	1.00	1.09	0.93	1.01
Houston	H-1	2.22	3.32	2.87	2.80
Houston	H-2	1.27	1.44	1.83	1.51
Houston	H-3	3.13	3.42	3.23	3.26
Houston	H-4	0.86	1.12	1.39	1.12
Houston	H-5	2.40	2.86	2.86	2.71
Houston	H-6	1.57	1.59	1.79	1.65
Houston	H-7	1.96	2.08	1.76	1.93
Houston	H-8	1.38	1.60	1.48	1.49
Houston	H-9	2.00	2.14	1.79	1.98
Houston	H-12	0.89	1.08	1.06	1.02
Houston	H-14	1.78	1.56	1.54	1.63

Source: DPS Accident Files (2)

interchanges have the highest and second highest accident rates, respectively, of the interchanges used in this study. Comparing the accident rates (Table 5) indicates that SA-5 has a relatively higher accident rate (approximately 25% higher) than does the H-3 interchange.

The effects of interchange configuration and geometrics can be fully appreciated when the interchange volumes are incorporated. Interchange H-3 handles almost twice the average daily traffic (ADT) of SA-5 with a lower accident rate (Table 6). It appears that the interchange type and configuration may greatly influence interchange accident rates and that these interchange characteristics become increasingly important as traffic volumes increase. In support of this observation, it should also be noted that from 1987 through 1989, the range of accident rates in the Houston urban area has been considerably higher than the range of accident rates in the San Antonio urban area (Table 4).

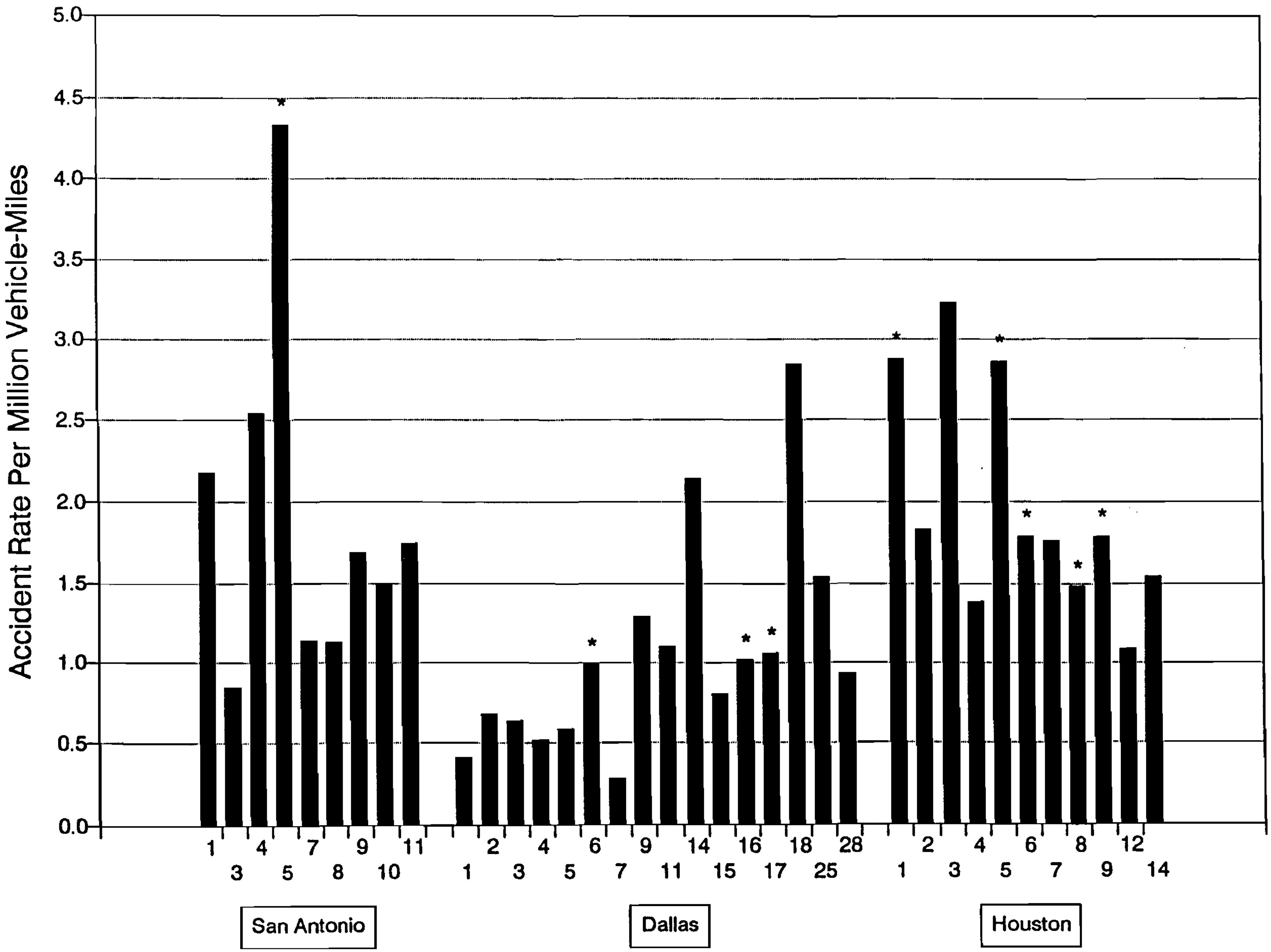
Figure 3 graphically illustrates the relationship between the number of accidents and million vehicle-miles. The SA-5 and H-3 interchanges experience the highest accident rate for the 36 interchanges studied. These interchanges reinforce the relationship between interchange type, configuration, and accident rates. Interchange H-3, a fully directional interchange with fairly good geometric design, serves a much larger ADT volume with a relatively small increase in accidents per ADT. Other interchanges with high rates include:

- D-18 - (Cloverleaf) With total ADT of 175,500 and 2.84 accidents per million vehicle miles

Table 6. 1989 Interchange Average Daily Traffic Volumes and Accidents.

Urban Area Location	Interchange Designation Code	Number of Accidents per year	Primary Volume (AADT)	Secondary Volume (AADT)	Total Volume (AADT)
Houston	H-3	722	216,000	192,500	408,500
Houston	H-2	397	179,500	217,000	396,500
Dallas	D-17	186	125,000	197,500	322,500
Dallas	D-16	177	165,500	153,000	318,500
Houston	H-9	310	166,500	149,500	316,000
Houston	H-1	496	177,500	138,000	315,500
Dallas	D-9	206	140,000	151,000	291,000
Houston	H-5	451	176,500	112,000	288,500
Houston	H-8	231	162,000	123,000	285,000
San Antonio	SA-5	655	156,000	120,000	276,000
Houston	H-7	236	114,000	130,500	244,500
Dallas	D-25	201	99,000	139,000	238,000
Houston	H-4	172	81,000	145,500	226,500
Houston	H-12	132	130,000	92,500	222,500
Houston	H-14	182	122,000	94,000	216,000
Dallas	D-11	117	77,500	117,500	195,000
San Antonio	SA-4	252	94,500	86,500	181,000
Dallas	D-18	273	125,000	50,500	175,500
Dallas	D-7	26	81,000	88,500	169,500
San Antonio	SA-11	156	83,500	79,500	163,000
Houston	H-6	157	61,600	98,400	160,000
Dallas	D-15	65	100,500	48,500	149,000
Dallas	D-2	47	69,500	58,000	127,500
Dallas	D-3	43	75,900	48,100	124,000
Dallas	D-1	25	72,500	40,000	112,500
San Antonio	SA-8	65	40,000	65,500	105,500
Dallas	D-5	30	53,500	41,000	94,500
Dallas	D-6	47	67,500	19,100	86,600
San Antonio	SA-10	71	47,500	39,000	86,500
Dallas	D-4	24	46,500	39,500	86,000
Dallas	D-14	91	46,000	31,450	77,450
San Antonio	SA-7	45	17,750	55,000	72,750
Dallas	D-28	35	45,500	23,500	69,000
San Antonio	SA-3	27	19,250	39,500	58,750
San Antonio	SA-1	48	17,850	22,500	40,350
San Antonio	SA-9	33	18,700	17,100	35,800

Source: DPS Accident Files (2) and SDHPT ATR Data.



Note: Asterick (*) above bar denotes interchange with left-hand ramps or connections.

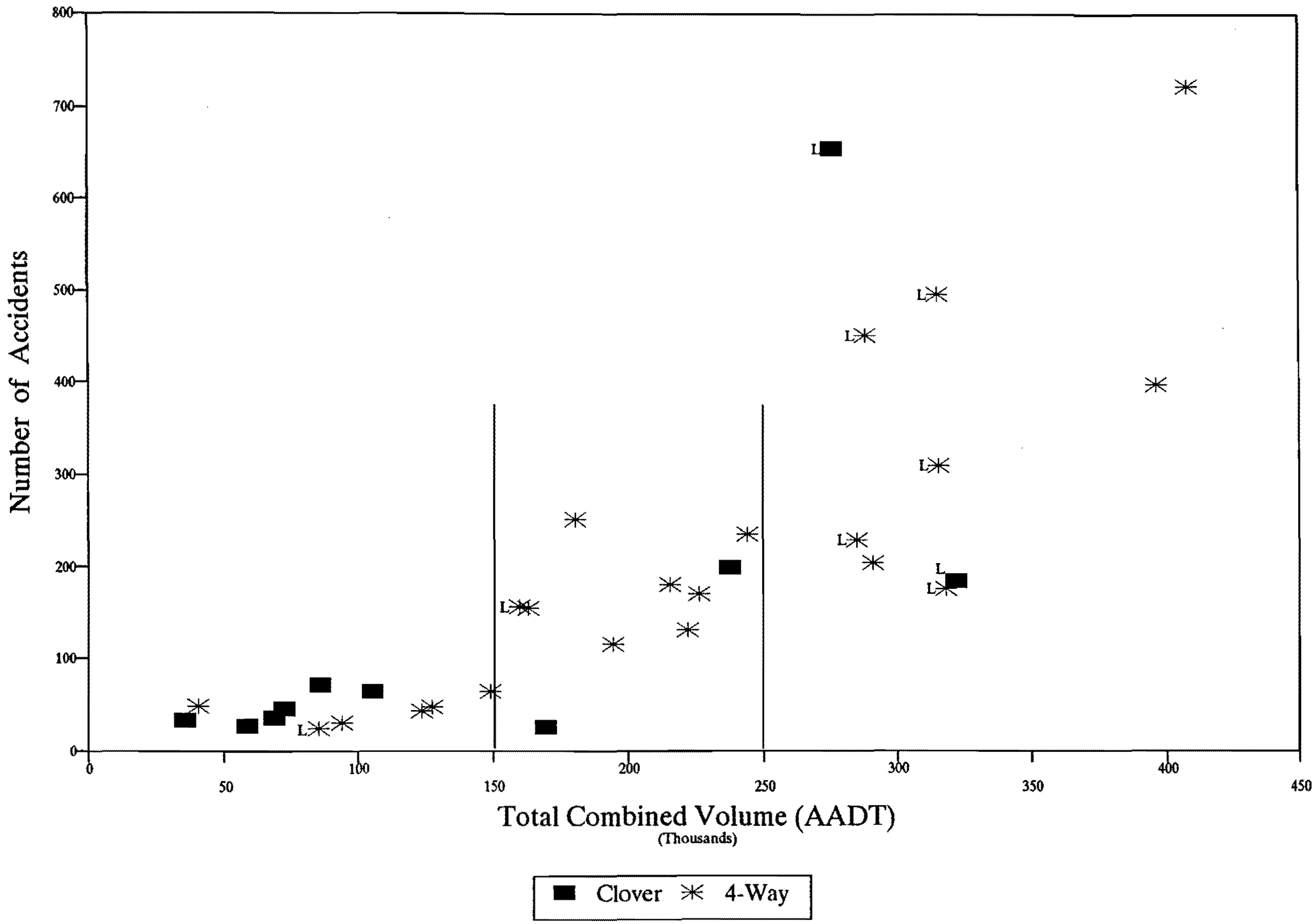
Figure 3. 1989 Accident Rates for Interchanges in Three Major Texas Urban Areas

- H-1 - (Four-leg fully directional with left-hand connector ramps) With total ADT of 315,500 and 2.87 accidents per million vehicle-miles, and
- H-5 - (Four-leg fully directional with left-hand connector ramps) With total ADT of 288,500 and 2.86 accidents per million vehicle-miles.

These interchanges have similar accident rates. The fully directional interchanges within this list, however, carry substantially more traffic than does the cloverleaf interchange. These interchanges reinforce the relationship between interchange type, configuration, and accident rates.

Figure 4 illustrates the relationship between total interchange ADT and number of accidents occurring within the interchange. In interchanges with a total ADT below 150,000, the number of accidents were fairly constant. It appears that interchange type and configuration probably have a minimal effect on accidents below this level of ADT.

Once interchange ADT volumes exceed 250,000, however, there is a high degree of variability in the number of accidents. Once ADT levels reach this range, interchange type and configuration may be critically important and have at least as much influence on accident rates as do volumes. Interchanges with ADTs ranging from 150,000 to 250,000 showed some variability in the number of accidents. In this range, interchange type and configuration probably are not significant. Careful consideration should be given, however, to specific geometric characteristics if future ADT estimates approach 250,000.



Note: "L" to the left of datapoint denotes left-hand ramp or connector facility.

Figure 4. 1989 Interchange Accidents in Relation to Interchange Volume

Reviewing Figure 4, six of the interchanges containing left-hand ramps are operating with total interchange volumes exceeding 250,000 ADT. The variance in accident rates illustrated in interchanges operating within this ADT range does not indicate a clear relationship between the presence of left-hand ramps within the interchange and the number of interchange accidents.

Summary

The inaccuracy of accident locations and incomplete accident files limited the type and scope of accident assessment relative to freeway-freeway interchanges. Examination of the accident rates associated with the study interchanges suggest, however, that a relationship appears to exist between accident rates and freeway-freeway interchange type and configuration. This relationship results in interchange type and geometric configuration becoming increasingly important as freeway-freeway interchange ADT levels increase (Figure 4). With interchange ADT volumes below 150,000, there is little variance between interchange type and configuration. However, when ADT volumes exceed 250,000, desirable geometric characteristics should be emphasized.

OVERVIEW OF INTERCHANGE DESIGN PRINCIPLES

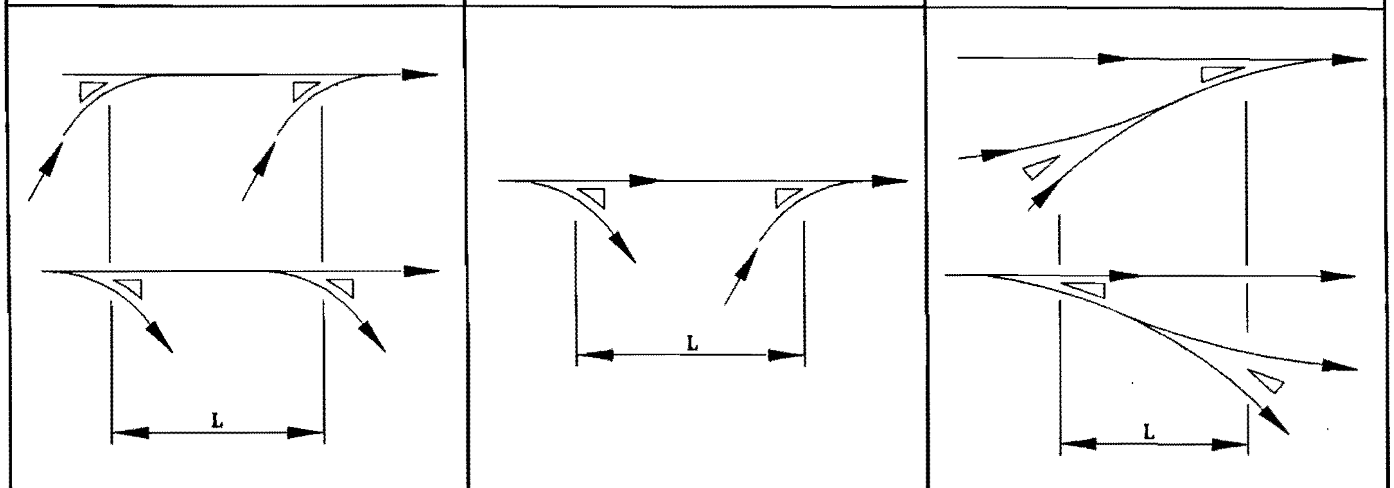
The primary concern in the design of an interchange should be the movement of drivers through the facility in the safest, most efficient manner possible. The ability of an interchange to accommodate drivers in this manner is closely related to the efficiency with which the information is provided to the driver and with the degree to which driver expectancy is re-enforced at the interchange.

Driver expectancy corresponds to the readiness of a driver to react to events, situations, or the presentation of information (10). Driver experience is the single most important factor that contributes to the level of driver expectancy. Other factors that contribute to the level of driver expectancy are signing and positive guidance. Signing transfers primary information to drivers concerning routes, safe operating speeds, etc. Positive guidance corresponds to secondary information that drivers obtain from the freeway and its surroundings. Edge-lines, centerline markings, placement of luminaries, etc., all contribute to positive guidance. Providing consistency in the application of signing and consideration of the secondary information presented to the driver through the design of interchange elements should improve safe and efficient travel through interchanges.

Spacing of exit and entrance ramps is an important design consideration that is necessary to insure the safe and efficient operation of freeways. Spacing of ramps within interchanges is important because of the time needed by drivers to evaluate their position in the traffic stream. Drivers must also ascertain how their position relates to the traffic stream with which they intend to merge. Adequate distance must be provided to allow weaving movements between ramps. Spacing between ramps must also be sufficient to provide adequate space for signing (3). By providing adequate weaving distance between successive ramps, the freeway operations between ramps can be improved (5). Minimum ramp spacings recommended by AASHTO (3) are summarized in Figure 5.

Uniform placement of ramps is also a desirable aspect of an interchange. Right-hand ramp facilities are considered to be superior to left-hand facilities, primarily because their prevalence has caused many drivers to develop an "inherent expectancy" that ramps are right-hand facilities (3). Left-hand facilities present problems with the development of signing layouts for freeways sections. The potential exists, therefore, for operational and safety problems resulting from left-hand ramps to reduce the overall effectiveness of the interchange.

Considerations associated with the design of merge terminals are similar to those associated with the proper spacing of freeway ramps. Merge terminals should be designed such that drivers are able to evaluate their position on the ramp relative to the traffic on the facility with which they will merge, make a decision, then react to this decision. Failure to design merge terminals with this process in mind can cause safety problems and create freeway and ramp bottlenecks that limit the overall capacity of these facilities.



Entrance-Entrance or Exit-Exit		Exit-Entrance		Roadways
Full Freeway	C-D Road or Freeway Distributor	Full Freeway	C-D Road or Freeway Distributor	System Interchange
1000	800	500	400	800
All Minimum Lengths are Measured from Physical Nose to Physical Nose				

Note: These recommendations are based on operational experience and the need for flexibility and adequate signing. They should be checked in accordance with the procedure outlined in the 1965 Highway Capacity Manual. The larger of the two values obtained from this publication should be used.

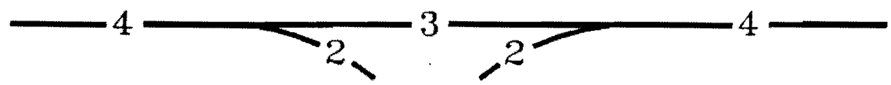
Figure 5. AASHTO Minimum Ramp Spacing Recommendations (2)

The selection of the type of freeway-freeway interchange is also an important consideration. The operational characteristics associated with merging and diverging activity vary with interchange type. The use of different interchange types, therefore, causes drivers to have different expectations from facility to facility. Differences in driver expectations between interchange types can be illustrated via an example. Single-ramp directional interchanges use a single freeway exit ramp to collect traffic from the freeway (Figure 3). This traffic can then be split such that those vehicles in the leftmost and/or middle lanes make a left-hand maneuver while those in the rightmost and/or middle lane make a right-hand maneuver. Dual-ramp directional interchanges (Figure 3) use two ramps that attach directly to the freeway to accomplish this split. The first ramp typically serves left-hand maneuvers and the second ramp serves right-hand maneuvers. Single-ramp directional interchanges are designed such that freeway weaving occurs prior to the exit ramp, while weaving associated with a dual-ramp facility normally occurs prior to and between the ramps. Providing regional consistency in the selection of interchange type should re-enforce driver experience with respect to merging and diverging maneuvers at interchanges within the region. This would in turn provide an improved level of driver expectancy.

The principles of route continuity, lane balance, and the provision of a basic number of lanes are interrelated. These principles must be adhered to in order for an interchange to function as part of a system and to maintain a high level of driver expectancy. A basic understanding of these principles helps to simplify the driving task by reducing the number of lane changes, simplifying the layout of signing, providing proper delineation of the route, and by reducing the driver's search for directional signing as they pass through the interchange (3).

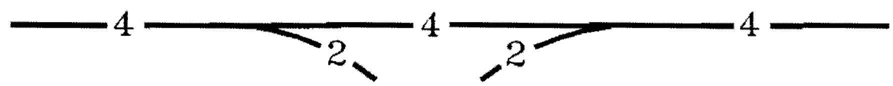
Route continuity refers to providing "...a directional path along and throughout the length of a designated route (3)." The route should be continuous through all structures such that a minimum number of lanes is designated that will exist along a significant length of the route, regardless of traffic demand or lane balance requirements (3). Lane balance refers to balancing the number of lanes that exist on the freeway mainlanes and on the ramps of an interchange or roadway. The relationship between the basic number of lanes and lane balance is illustrated in Figure 6.

Design considerations for interchanges must incorporate a number of basic principles in order for these facilities to function properly. The design of interchanges should provide a system of freeway mainlanes and connections that satisfy and re-enforce driver expectancy through the interchange. Signing and positive guidance must both be considered to insure that desired information is transferred to the driver properly and in sufficient time to allow the driver to assimilate the information presented. The proper design of entrance and exit ramps is necessary to insure that merging traffic streams do not abruptly come together and create a bottleneck situation. The placement and spacing of ramps is important to obtain high speed, efficient merge and diverge activity. The selection of the type of freeway-freeway interchange is also an important consideration in maintaining a high level of driver expectancy. Route continuity, lane balance, and the provision of a basic number of lanes



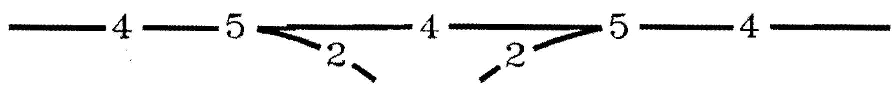
Lane Balance but No Compliance with Basic Number of Lanes

–A–



No Lane Balance but Compliance with Basic Number of Lanes

–B–



Compliance with Both Lane Balance and Basic Number of Lanes

–C–

Figure 6. Coordination of Lane Balance and Basic Number of Lanes (2)

throughout a system also contribute to improved efficiency and driver expectancy at interchanges.

ANALYSIS OF INTERCHANGE GEOMETRICS AND OPERATIONS

Geometric analysis of the interchanges focused on evaluating specific areas of freeway-freeway interchanges with respect to six different geometric considerations. These considerations were:

- The design of merge and diverge terminals within an interchange
- The adequacy of weaving areas, i.e., spacing of ramps within an interchange
- The effects of left-hand entrances and exits with respect to operations and safety
- The provision of route continuity through an interchange
- The provision of lane balance for the major routes through an interchange
- The horizontal and vertical alignment of individual elements within an interchange

The design of merge terminals were evaluated via the use of INTRAS, a computer simulation model designed to simulate freeway operations at a microscopic level. Three different merge configurations were evaluated at various conditions. The measure of effectiveness used to evaluate these configurations was the average lane density within the simulated merge section.

The remaining design considerations were evaluated through the use of case studies and the macroscopic freeway simulation model FREFLO. On-site inspections, as-built plan-and-profile sheets, and historical accident data were used to develop case studies that illustrated minimum design guidelines needed to provide acceptable levels of safety and operations. FREFLO was used to evaluate changes in operations that corresponded to geometric improvements for the study interchanges.

Procedure for Merge Terminal Evaluations

Three different merge terminal configurations were analyzed in this study. These configurations included: 1) interior taper merge, 2) exterior taper merge, and 3) exterior parallel merge designs. These different configurations function differently under similar traffic demands and geometric conditions. Empirically quantifying the magnitude of this difference would require extensive studies and costs that are well beyond the scope of this project. An understanding of the relative operational benefits with respect to traffic demand and geometry can be ascertained through the use of computer simulation models. The relative benefits of one configuration over another can, therefore, be used to provide guidance in the design of merge terminal facilities.

Figure 7 shows the base freeway segment used to simulate the three merge terminal configurations. Section 1 consists of the freeway and ramp lanes prior to the physical nose that is formed by the merge of these facilities. Section 2 extends from the physical nose of the merge to the point after the merge where freeway mainlanes are effectively reduced from a 5-lane to a 4-lane cross-section. This point was defined as the location where the merge lane was reduced to a width of nine feet. Beyond this distance driver work load quickly increases and can reach the point where the driver finds it necessary to stop and/or use the shoulder of the roadway to complete the desired maneuver. The freeway mainlanes beyond this point, therefore, were assumed to function as a 4-lane freeway. Section 3 extended from the downstream end of Section 2 to the outer limits of the modelled network.

The base freeway segment shown in Figure 7 was used so that comparisons could be made concerning the relative benefits of lane configurations within the merge area (Section 2) of the different merge terminal designs. The total flow volumes needed to obtain average level-of-service B, C, D, and E in Section 4 were calculated. These volumes were used as input volumes to the modelled network. The input volumes were assigned to the freeway and ramp, respectively, using 70/30, 60/40, and 50/50 traffic splits. By specifying volume levels that would not exceed the capacity in Section 3, downstream bottlenecks did not affect operations in Section 2. Taper rates of 30:1, 50:1, and 70:1 were used to vary the length over which the merge lane was eliminated for both the interior and exterior taper merge terminal designs. Auxiliary lane lengths for exterior parallel merge terminal designs were varied from 1000 to 2000 feet. Density and overall delay within the merge section were used as operational MOEs.

Results of Alternative Merge Terminal Design Evaluations

Figure 8 illustrates an interior taper merge terminal that was modelled using the operational and geometric conditions previously described. AASHTO recognizes this type of merge design as one means of tapering out a lane. The inherent problem that exists with this type of merge design is that in congested conditions drivers in the taper lane can be trapped if they are not able to find a gap in which they can enter the major stream of traffic. This creates an unexpected situation for a driver and has the potential to cause operational problems. When drivers become trapped in the merging lane, they may be forced to travel

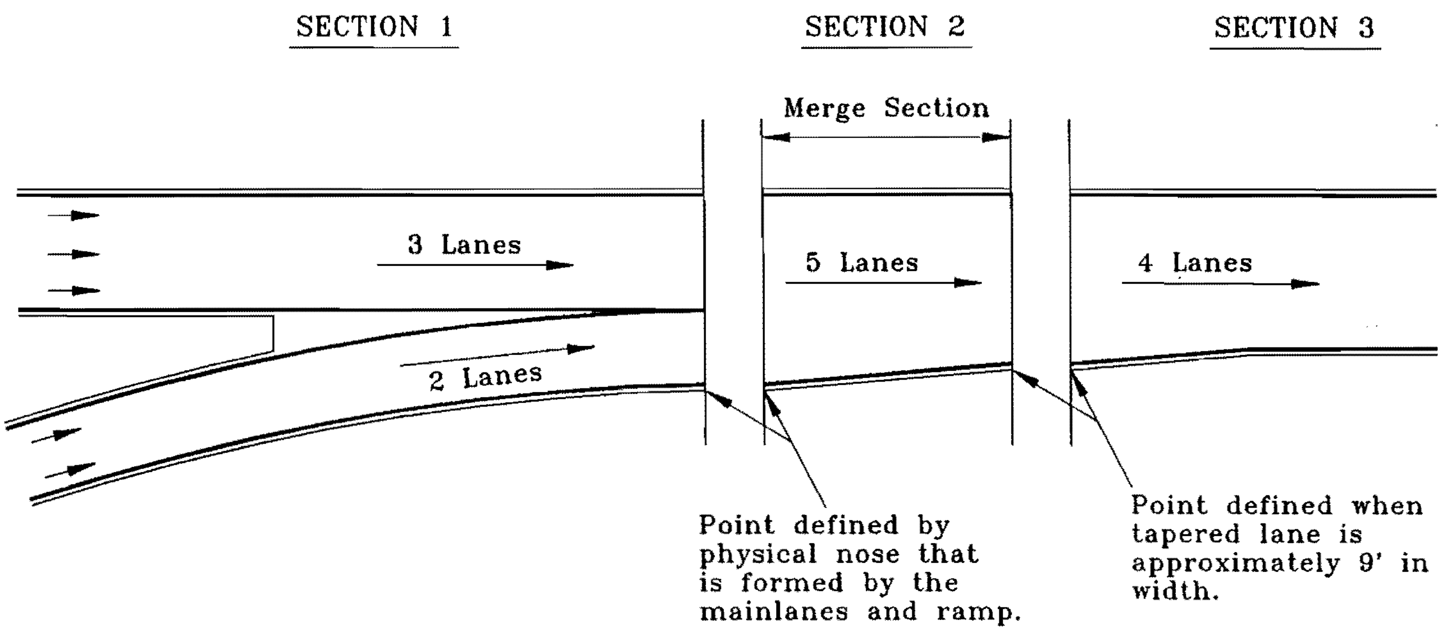


Figure 7. Base Freeway Section Used for Modelling Taper Merge Terminal Designs

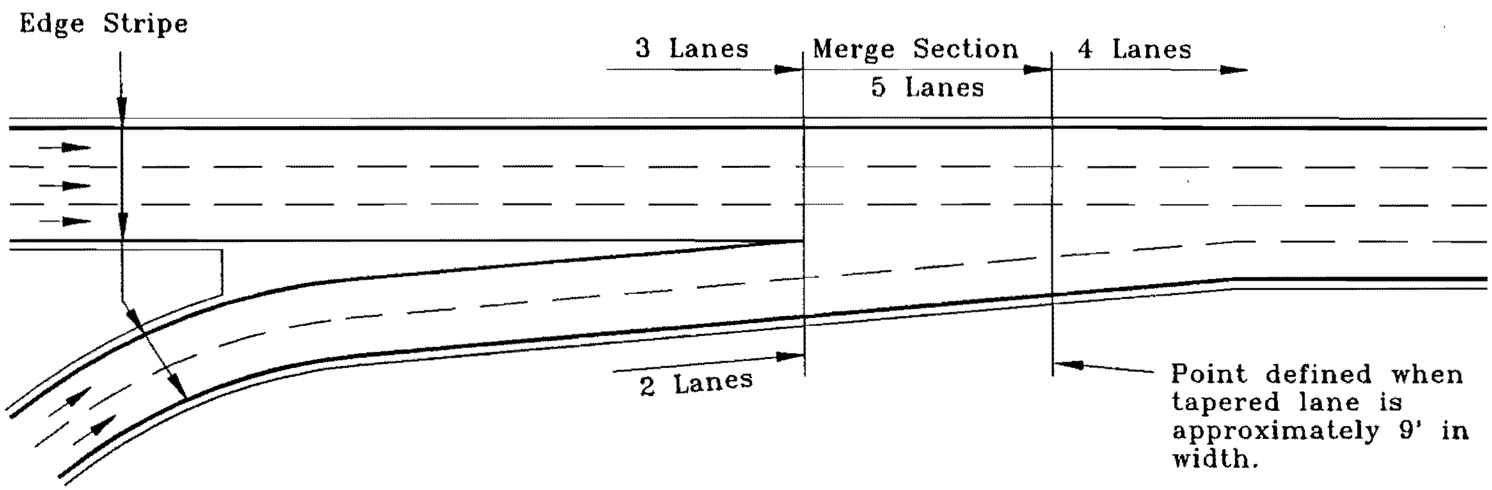


Figure 8. Illustration of Interior Taper Merge Configuration

at a very low rate of speed or stop altogether. Both rear-end and side-swipe accidents can be expected in these situations. This type of design also reduces the overall capacity of the merge terminal, since merging activity takes place in two relatively high speed lanes as compared to merging activities that occur in the outermost freeway lane.

Figures 9, 10, and 11 illustrate the results of these simulations for freeway/ramp traffic splits of 70/30, 60/40, and 50/50, respectively. These figures graph the density within the merge section against the total traffic demand on the merge section. These figures indicate that, while longer tapers improve the LOS in the merge section, acceptable levels of operation (i.e., LOS C or better) were obtained only at very low traffic demands. As the ratio of freeway to ramp traffic was varied from 70/30 to 50/50, operational conditions were found to deteriorate more rapidly as volumes increased.

Figure 12 illustrates an alternative to the interior taper design. This alternative provides for the outer lane to be tapered out beginning immediately after the physical nose formed by the merge. The input volumes, taper rates, and mainlane/ramp traffic splits used to simulate the operations in this type of design are the same as those used to evaluate the interior taper merge configuration. Figures 13, 14, and 15 indicate that at low volumes acceptable LOS could be expected with virtually all taper rates. Higher volume conditions, however, required higher taper rates to accommodate weaving movements within the merge section.

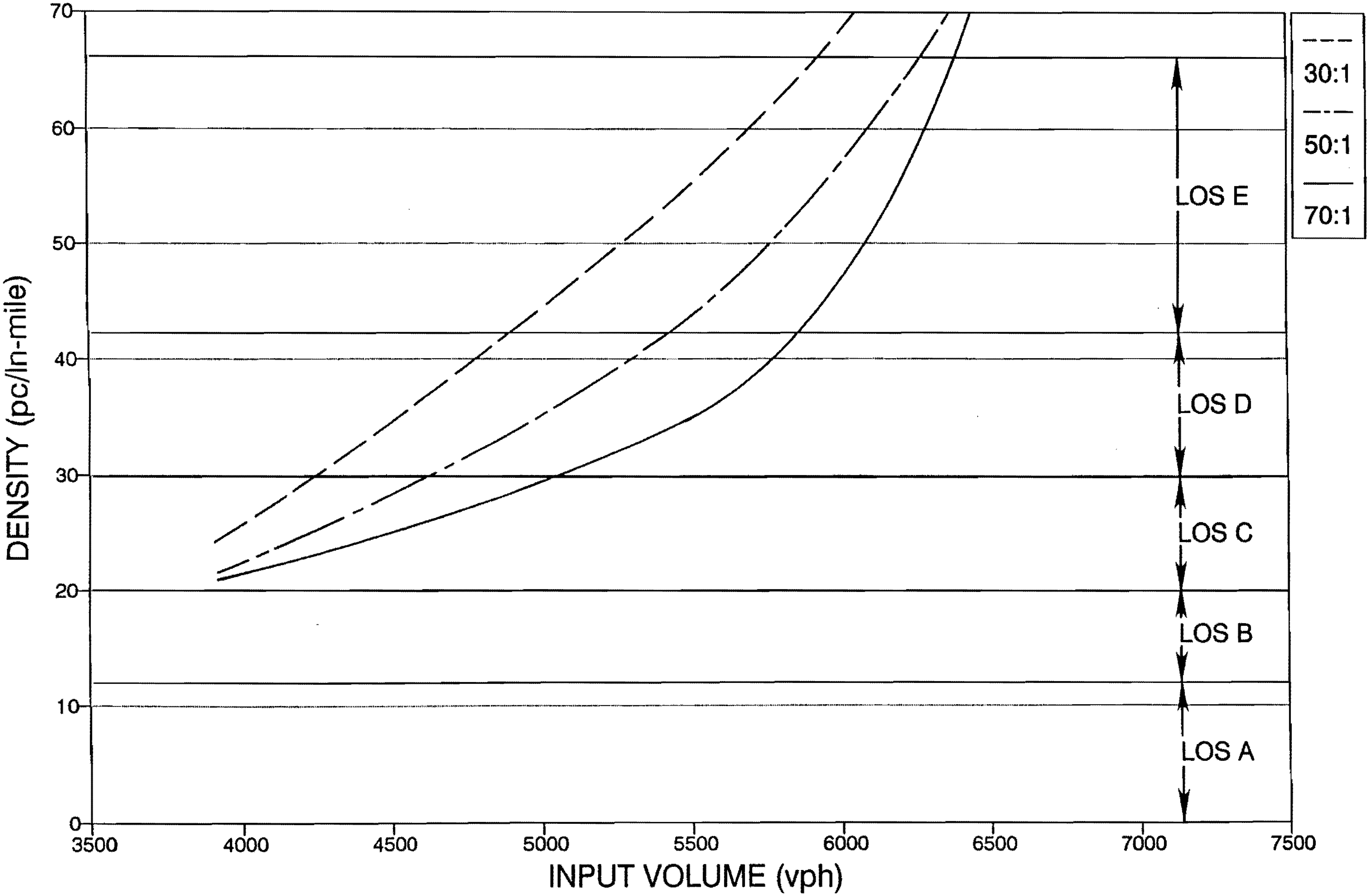


Figure 9. Interior Merge Configuration, 70/30 Traffic Split

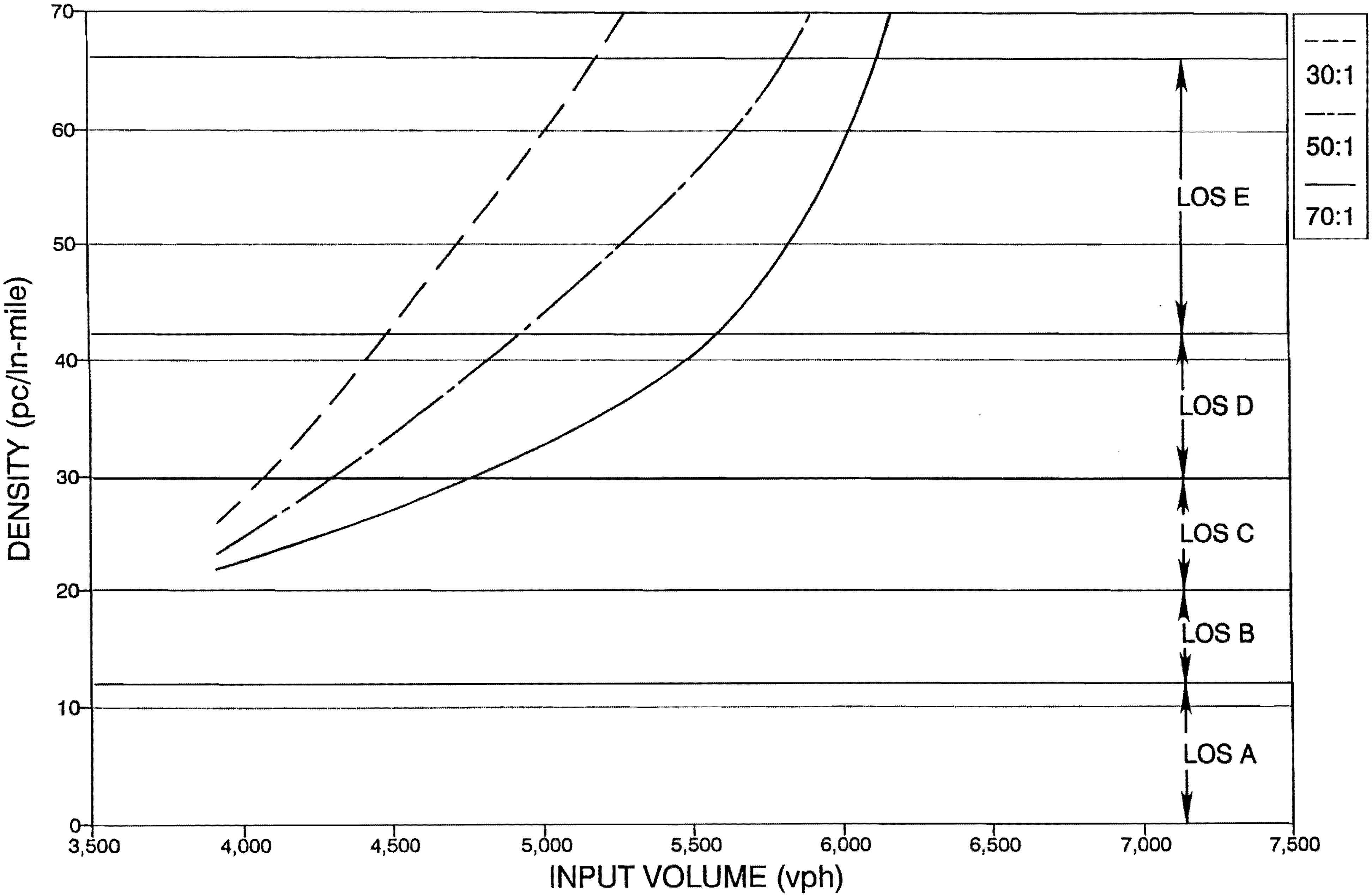


Figure 10. Ingterior Taper Merge Configuration, 60/40 Traffic Split

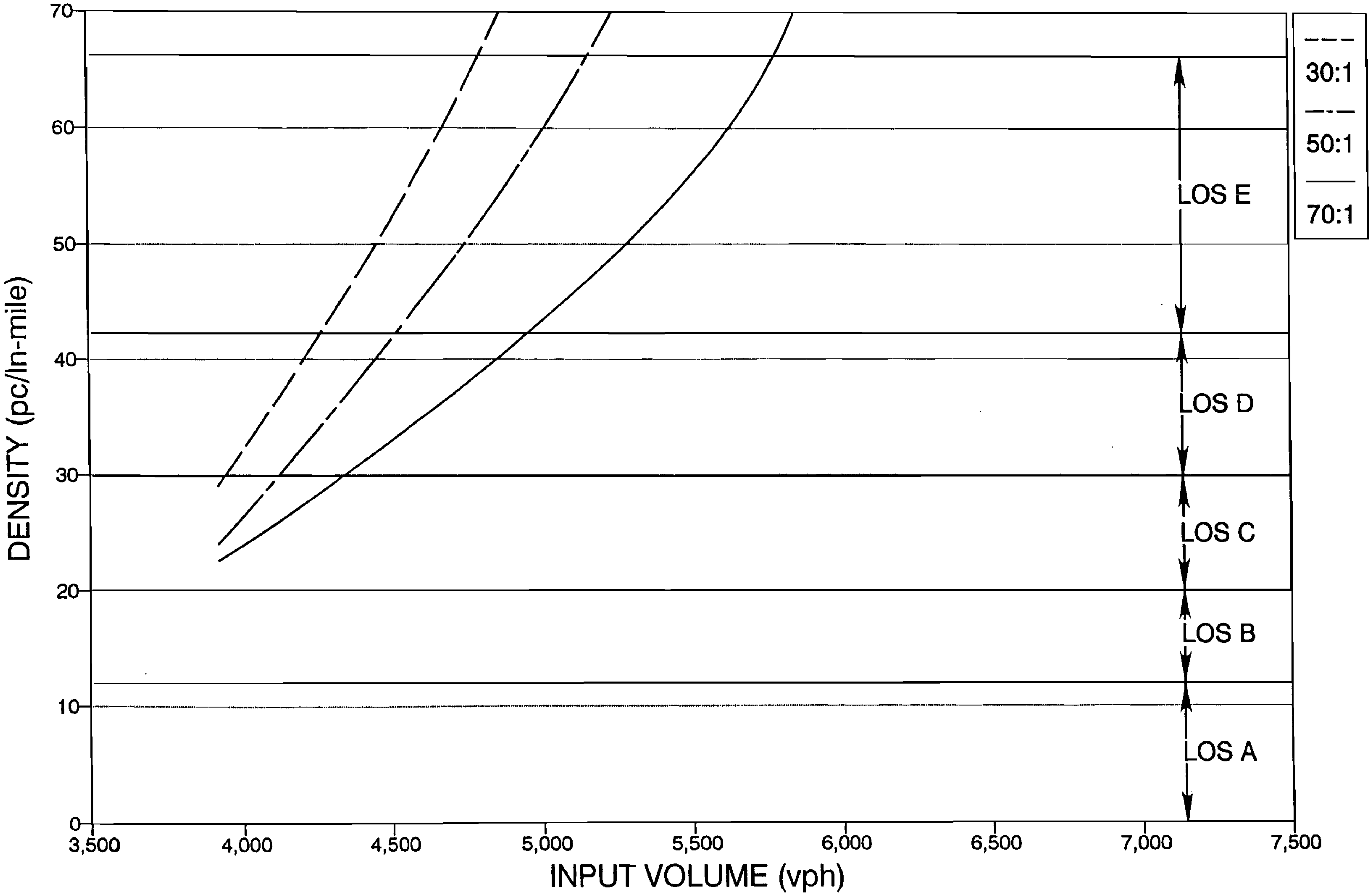


Figure 11. Interior Taper Merge Configuration, 50/50 Traffic Split

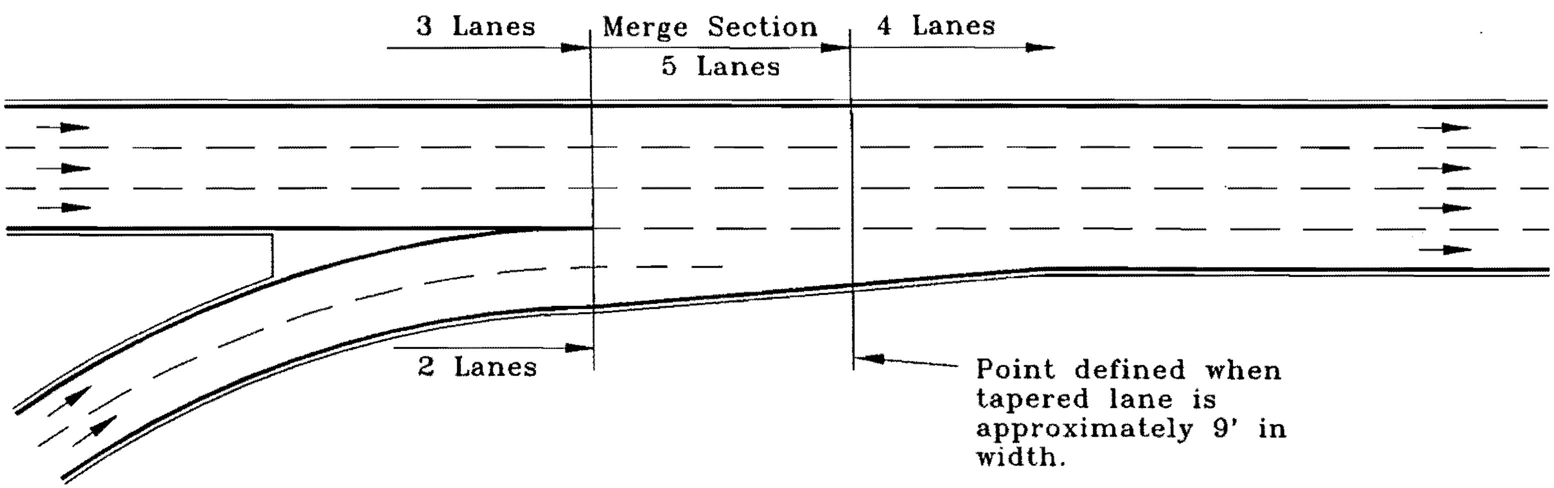


Figure 12. Illustration of Exterior Taper Merge Terminal Configuration

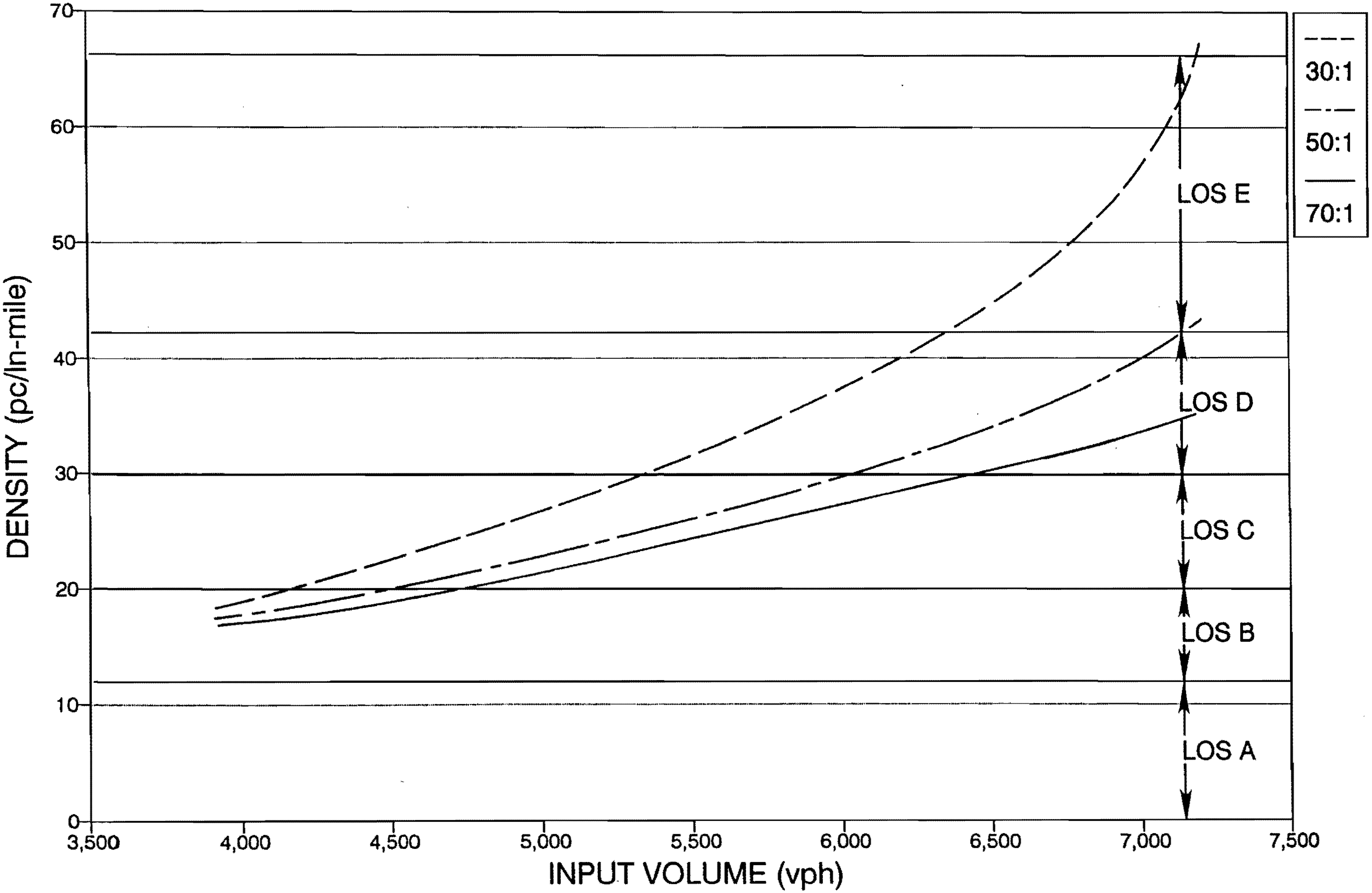


Figure 13. Exterior Taper Merge Configuration, 70/30 Traffic Split

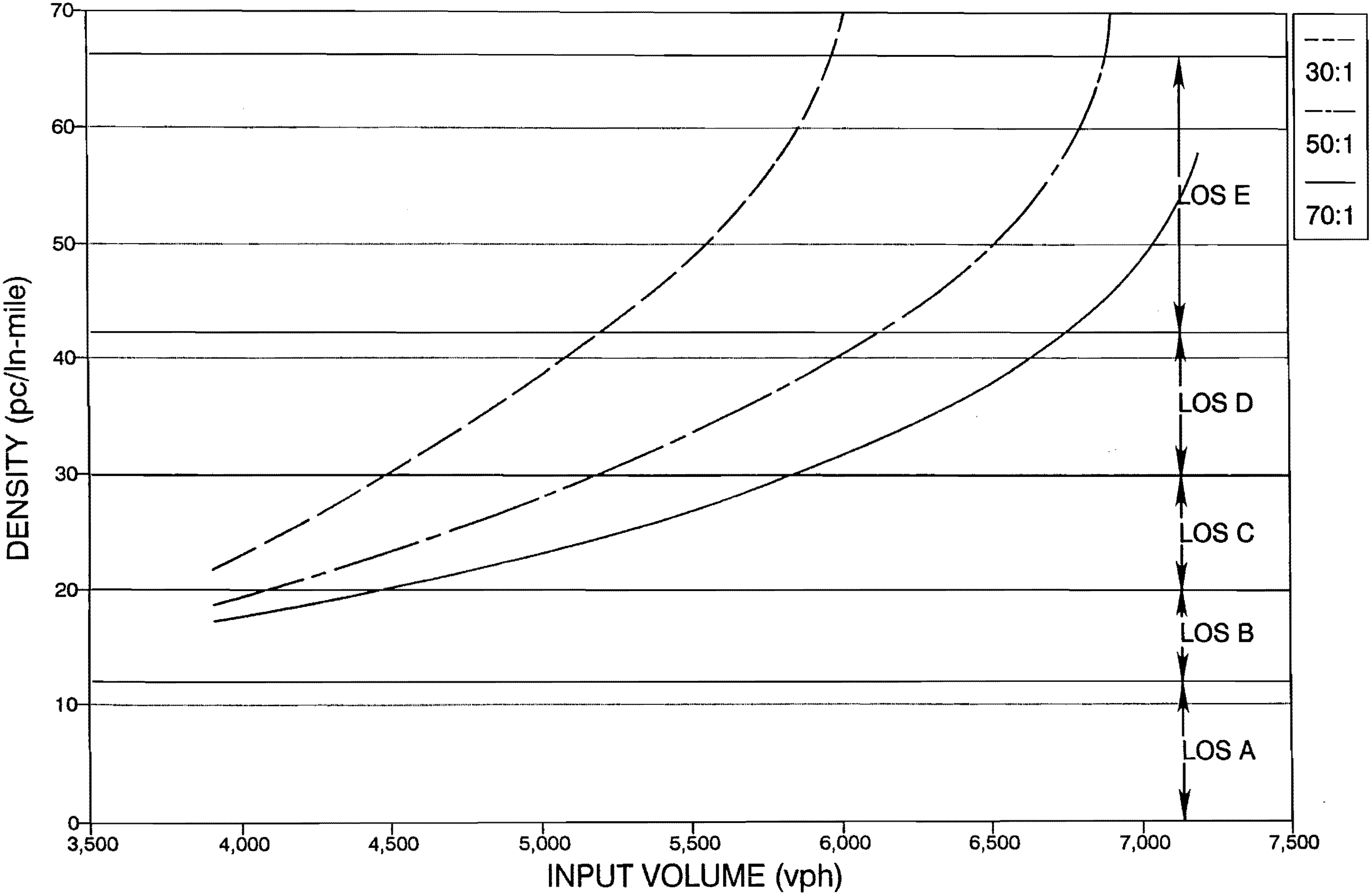


Figure 14. Exterior Taper Merge Configuration, 60/40 Traffic Split

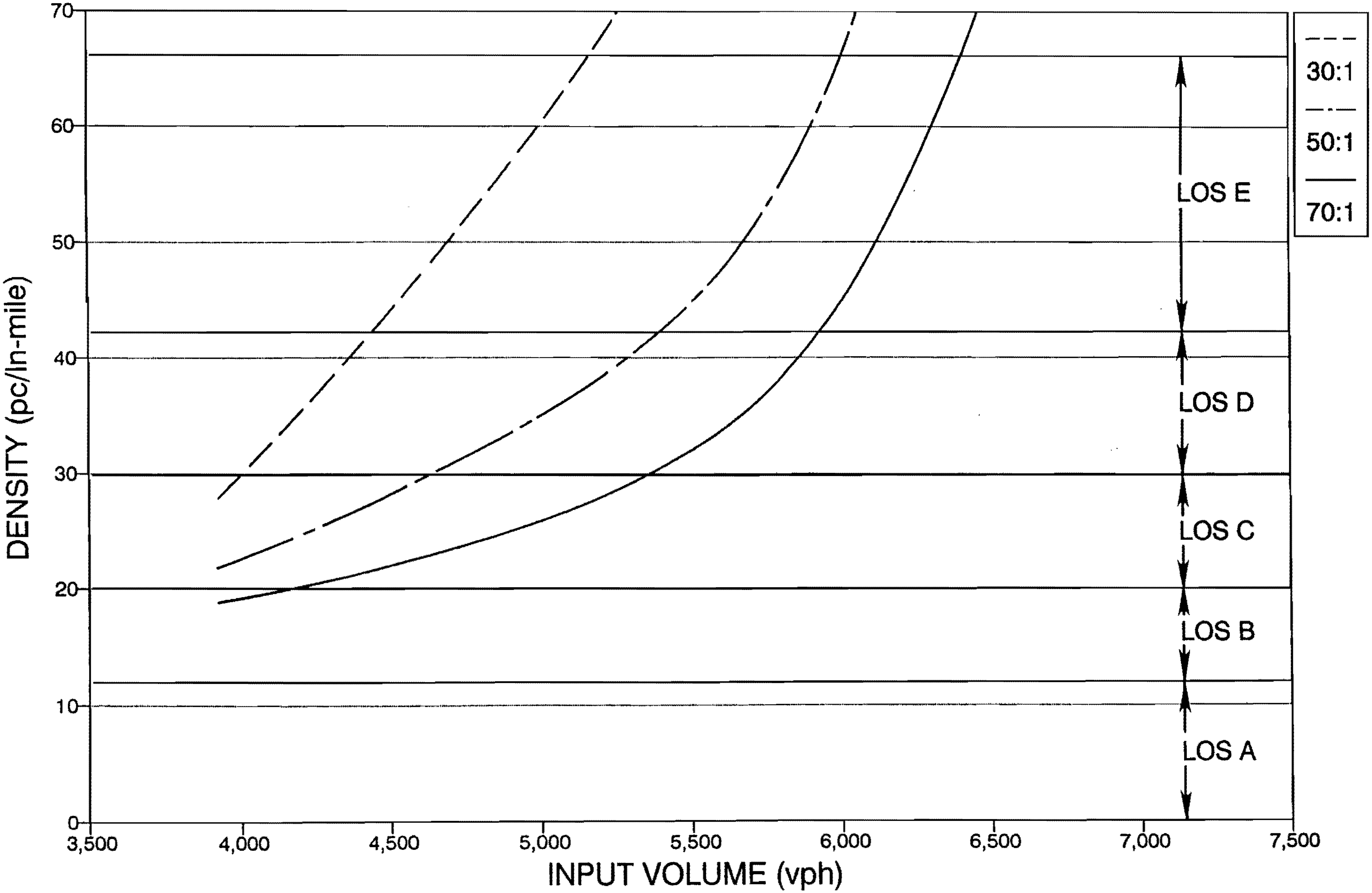


Figure 15. Exterior Taper Merge Configuration, 50/50 Traffic Split

Comparison of the results obtained from computer simulations indicate that exterior taper merge configurations provide an advantage over interior taper merge configurations. The operations within the merge area of interior taper merge configurations deteriorated to LOS D at relatively low traffic volumes compared to exterior taper merge configurations. This conclusion was supported by the operational output obtained for all traffic splits, taper rates, and input volumes used. Little difference exists in the cost of construction for the two configurations. The apparent operational benefits, therefore, suggest a preference of exterior taper merge configurations to interior taper merge configurations.

Some recommendations concerning the design of exterior taper merge facilities can also be made. The operational data used in this analysis was generated via a computer simulation model. As such, this data can be used only to evaluate the relative benefits of different geometric designs. Evaluation of Figures 13, 14, and 15 indicate that large taper rates improve merge terminal operations under high speed, high volume conditions. Minimum AASHTO recommendations indicate that 50:1 to 70:1 taper rates should be used under these types of conditions, with 70:1 being preferred (3 and 11). The results of this study suggest a substantial advantage of a 70:1 taper in the design of exterior taper merge configurations.

Figure 16 illustrates the exterior parallel merge configuration. This section was modelled using various lengths for the auxiliary lane ranging from 1000 to 2000 feet. Simulations indicate that each of these distances was sufficient for vehicles to merge into

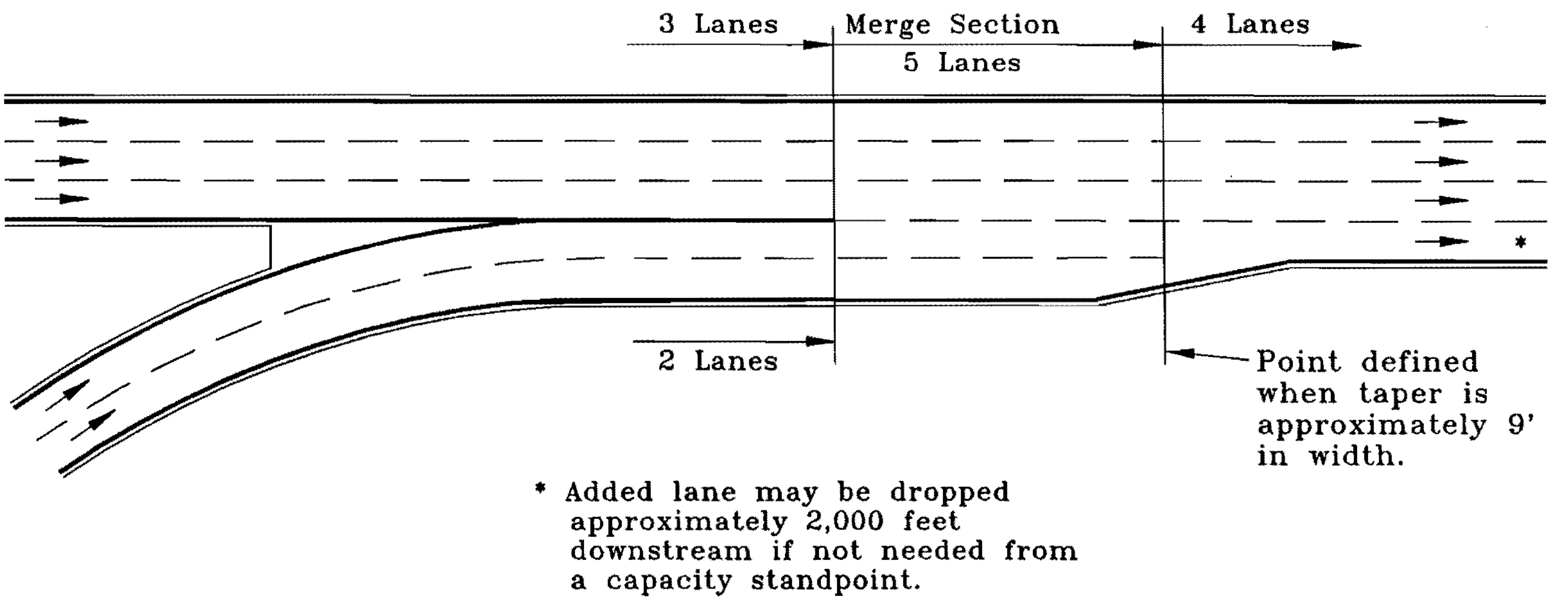


Figure 16. Exterior Parallel Merge Terminal Configuration

the mainlane traffic stream. The results of the computer analysis indicated that exterior parallel merge configurations provided the most operationally sound design of the three which were tested. Consideration should be given, however, to the uniform application of either taper or parallel merge terminal design within a geographical area due to their operational differences. Taper designs provide a long uniform tapering of a lane. Parallel designs, however, provide a lane of uniform width for a given distance, then utilize a much more abrupt taper rate relative to those associated with interior or exterior taper merge designs. It is important, therefore, that the type of merge terminal design be made based on policy rather than cost of construction.

A major advantage of exterior taper merge and exterior parallel merge sections (Figures 12 and 16, respectively) is that merging activity occurs in the slower outside lane where it is desired and where it is expected by drivers. The problem of a driver becoming trapped between two traffic streams is also be eliminated. Drivers can use the freeway shoulder as an outlet if they reach the end of the taper or auxiliary lane without merging. Elimination of these two problems facilitates more efficient, higher speed operations on the freeway ramp and at the ramp terminal. Safety benefits also exist because of the reduced speed differential between mainlane and ramp traffic.

Procedure for Case Study Evaluations

This section of the report presents case studies that focus on selected interchanges located in Houston, Texas. The interchanges selected for the case studies were intended to serve as typical examples of geometric configurations that are associated with both operational and safety problems and because of the availability of highly detailed geometric, operational, volume and accident information. Alternative design concepts that can be expected to improve the conditions at these locations are also presented. The case studies focus on:

- Left-hand ramps;
- Weaving sections;
- Heavy vehicles and,
- Route continuity and lane balance

The level of detail contained in the historical accident data in Houston was the primary reason that only Houston interchanges were used in the case study analysis. The accident information collected from Houston and Fort Worth assigns both a direction of travel and milepost location associated with each accident. This information allowed the accidents to be located relatively accurately on individual connectors and mainlanes within interchanges.

Schematic representations of each interchange with the accident locations noted were correlated with interchange line drawings depicting average LOS. Locations with corresponding operational and safety degradation were then highlighted for further evaluation. Many locations exhibited a high number of accidents that could not be related to geometric or operational problems. The cause of safety problems in these cases may be related to signing problems, problems with positive guidance, and/or other miscellaneous factors that were beyond the scope of this study.

The purpose of the alternative designs developed for these case studies was to suggest design improvements that would provide safety and operational benefits over existing configurations. The FREFLO model was used to help evaluate the relative benefits associated with these improvements. Capacity on all freeway and ramp facilities for each case study was maintained so that the operational affects of different ramp positions could be isolated. This model is not capable of evaluating operational advantages and disadvantages associated with driver expectancy, weaving activity, and other activities that are typically modelled by microscopic models. Consequently, the line drawings do not represent operational benefits that could be expected with these improvements.

All facilities that connected local street systems to freeway-to-freeway connectors in the study interchanges were moved outside the interchange in the alternative interchange designs. This was done to preserve the functional purpose of the freeway-to-freeway connectors and ramps, that purpose being primarily to provide high speed transfers from one freeway to another. The plan data that was collected was not sufficient to evaluate the feasibility of moving these facilities outside of the interchange area, nor were the affects on the local street system evaluated. Repositioning these facilities would, in actuality, require consideration of how adjacent service interchanges and the local street system would be affected. The importance of preserving the functional hierarchy within system interchanges, however, is essential to maximize the efficiency of these facilities.

Case Study - Left-hand Ramps

The H-5 interchange (Figure 17(a)) was selected to evaluate alternative designs for left-handed ramps. This interchange ranked fourth out of the total 36 interchanges evaluated in this study with respect to the number of accidents per million vehicle miles travelled. Figure 17(b) is a line drawing representation of the highlighted section that illustrates the geometric and peak-hour operational conditions along this section of roadway. Approximately 20% of the total accidents that occur at this interchange each year occur in this section of the interchange. The left-hand exit is associated with a substantial number of accidents occurring along this section of the freeway. The close proximity of the ramps in this section also appears to be contributing to the occurrence of accidents in this section. Figure 17(b) indicates that the distance between the two outer ramps exceeds the minimum recommended distance shown in Figure 5. The high level of congestion combined with the large amount of diverging activity in this section, however, appears to be compounding the problems associated with the left-hand exit. No deficiencies with respect to horizontal and

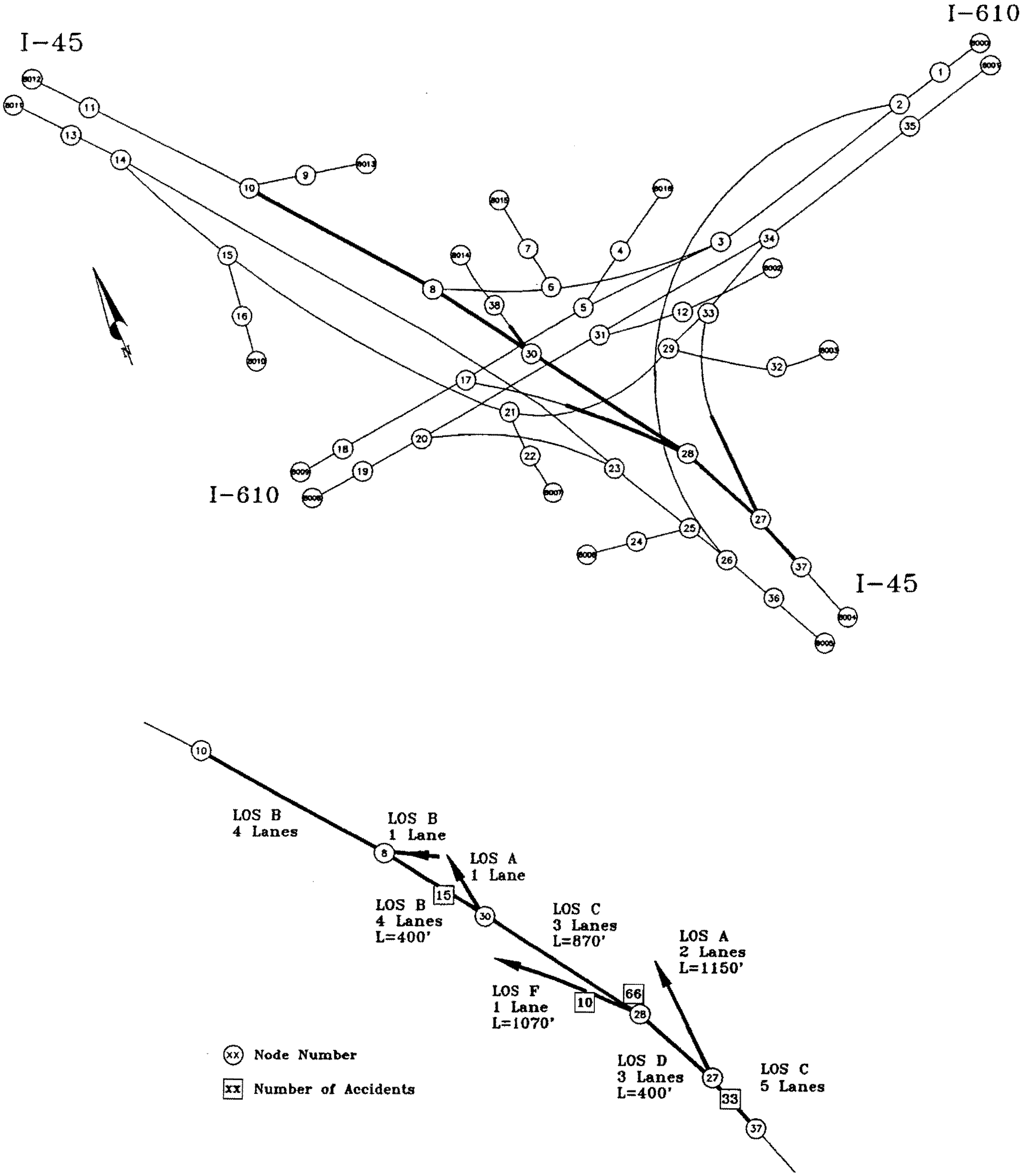


Figure 17. Line Drawing of the Existing H-5 Interchange

vertical alignment were identified other than the presence of the left-hand exit and the weaving area caused by three ramps being located in a 1400-foot section of the freeway.

Figure 18 illustrates an alternative design for this interchange. This alternative would reposition the left-hand connection so that drivers travelling from westbound I-45 to southbound I-610 would exit the freeway via a right-hand connection (node 28). This would eliminate weaving problems, improve driver expectancy, and simplify the signing layout prior to the mainlane section between nodes 27 and 28 of Figure 17(b). Computer simulations indicated that the LOS on the westbound exit ramp would change from LOS A to LOS F with this alternative. No relative improvement, however, was indicated by the computer analysis on the I-45 westbound to I-610 southbound freeway-freeway connector. The model used to evaluate these improvements was not capable of evaluating the improvement in operations associated with reducing diverging activity on the mainlanes or with improved levels of driver expectancy associated with eliminating the left-hand exit. Providing additional capacity would also be required to accommodate the demand on the facility.

Case Study - Weaving Sections

Weaving problems that develop as a result of inadequate ramp spacing are typified by analysis of the H-6 interchange (Figure 19(a)). The segments that are highlighted provide a good illustration of inadequate weaving sections. This interchange had low volumes relative to the 36 interchanges that were studied. The accident rate at this

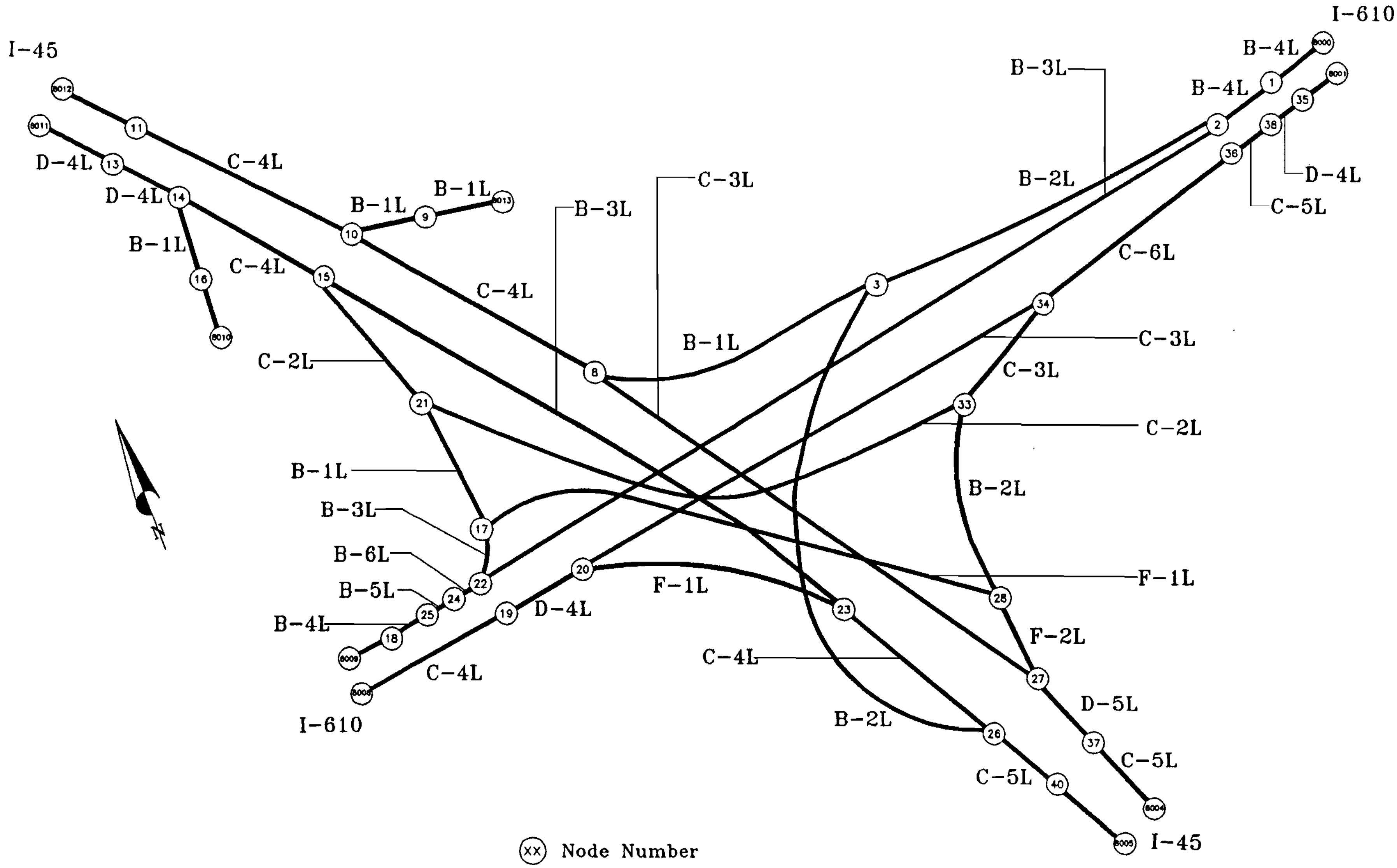


Figure 18. Alternative Design of Interchange H-5

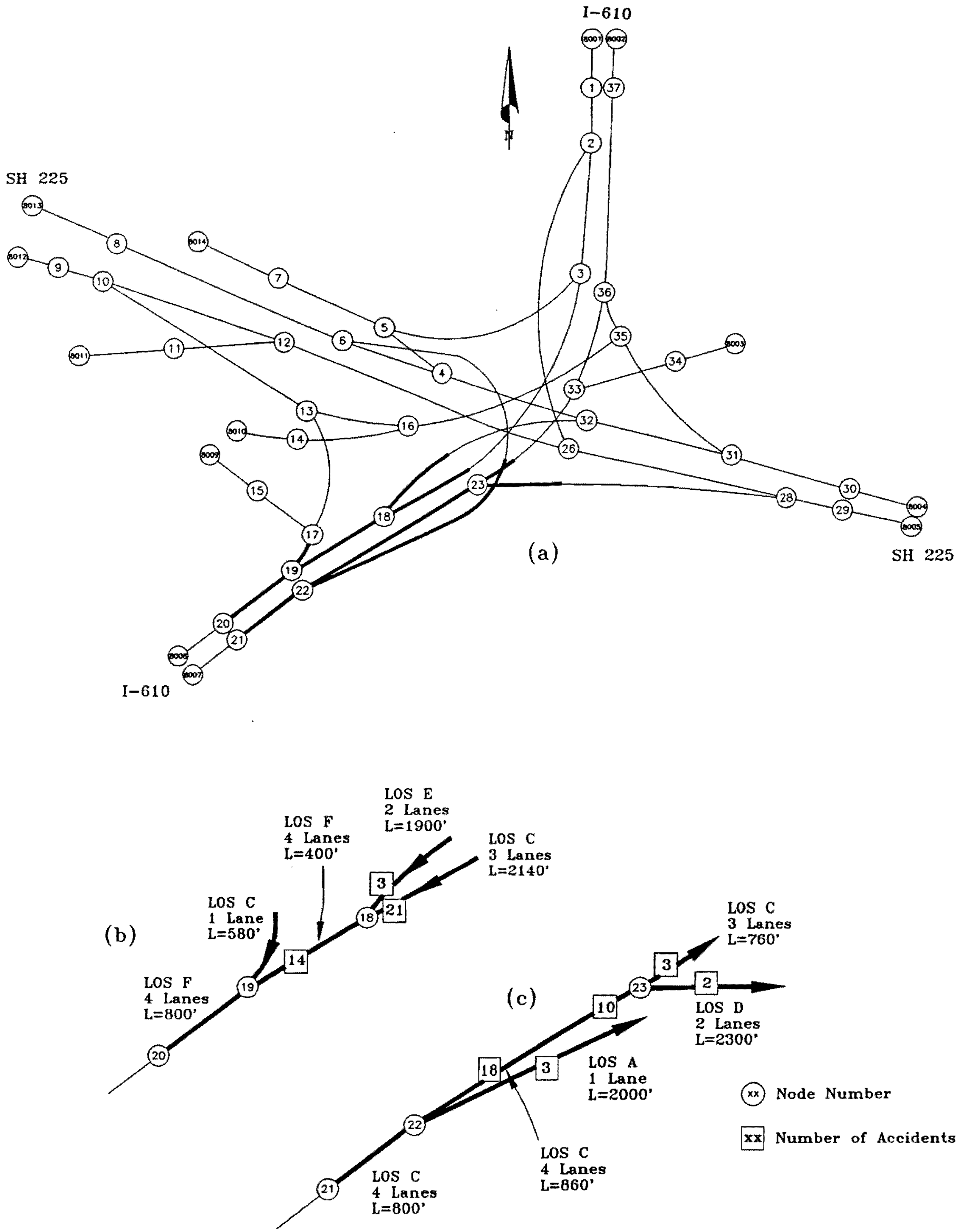


Figure 19. Line Drawing of the Existing H-6 Interchange

interchange, however, was the tenth highest of all interchanges analyzed in the study. Analysis of the interchange indicates that, while the total volume served by the interchange is relatively small, certain links are operating near the unstable flow region. A large percentage of the accidents that occur at this interchange occur in these areas.

Figures 19(b) (nodes 18 and 19) and 19(c) (nodes 22 and 23) indicate that the distance between successive ramps is much less than the recommended minimum value recommended by AASHTO (Figure 5). Figure 19(b) shows the geometry associated with the entrance-entrance configuration. The high accident frequency in this section appears to be the result of an inadequate weaving section, inadequate merge terminal design and low LOS. The amount of traffic on the mainlanes doubles after the first entrance ramp, yet only 580 feet is available for the entering vehicles to merge onto the freeway mainlanes. The operations and safety at this location are also adversely affected by the presence of an interior merge at the first merge terminal (node 18). Furthermore, three lanes are merged onto the mainlanes with two of those lanes being tapered out within 800 feet. Traffic entering the freeway from the second entrance (node 19) causes a constriction at the first entrance in this section relatively soon after congested conditions are observed. This bottleneck might explain many of the accidents that occur on the mainlanes prior to the first entrance ramp.

The majority of accidents occurring in the section shown in Figure 19(c) occur between the exit ramps (nodes 22 and 23). This is probably caused by the short weaving section and the fact that over 50% of the mainlane traffic uses the second (node 23) exit ramp. Based on output from the FREFLO model, the second connector is operating at a relatively low LOS. A large amount of weaving appears to be occurring between the first and second exit ramps. The short weaving section and relatively low LOS on the second exit ramp, therefore, appear to be contributing to the high accident frequency in this section.

Figure 20 illustrates the operational improvements at the interchange that results from realigning the left-hand exit so that it exits the freeway from the right (node 32) and connects to an entrance ramp (node 17). This eliminates the weaving section between the two direct connectors (Figure 19(b), nodes 18 and 19) and allows them to enter the freeway via the same entrance ramp and an auxiliary lane of adequate length (Figure 20, nodes 19 and 24, respectively). A second alternative that would allow the spacing between the entrance ramps shown in Figure 20 to be lengthened would also provide operational improvements over the existing situation and, possibly, over that of the alternative shown in Figure 20. The section shown in Figure 19(c) might also be improved by increasing the separation distance between the ramps or by combining these ramps.

Case Study - Heavy Vehicles

To illustrate the considerations associated with heavy vehicles in interchange design, the H-3 interchange was evaluated. This interchange is in the southwest part of Houston. The volume at this location was the highest of any of the 36 interchanges that were analyzed

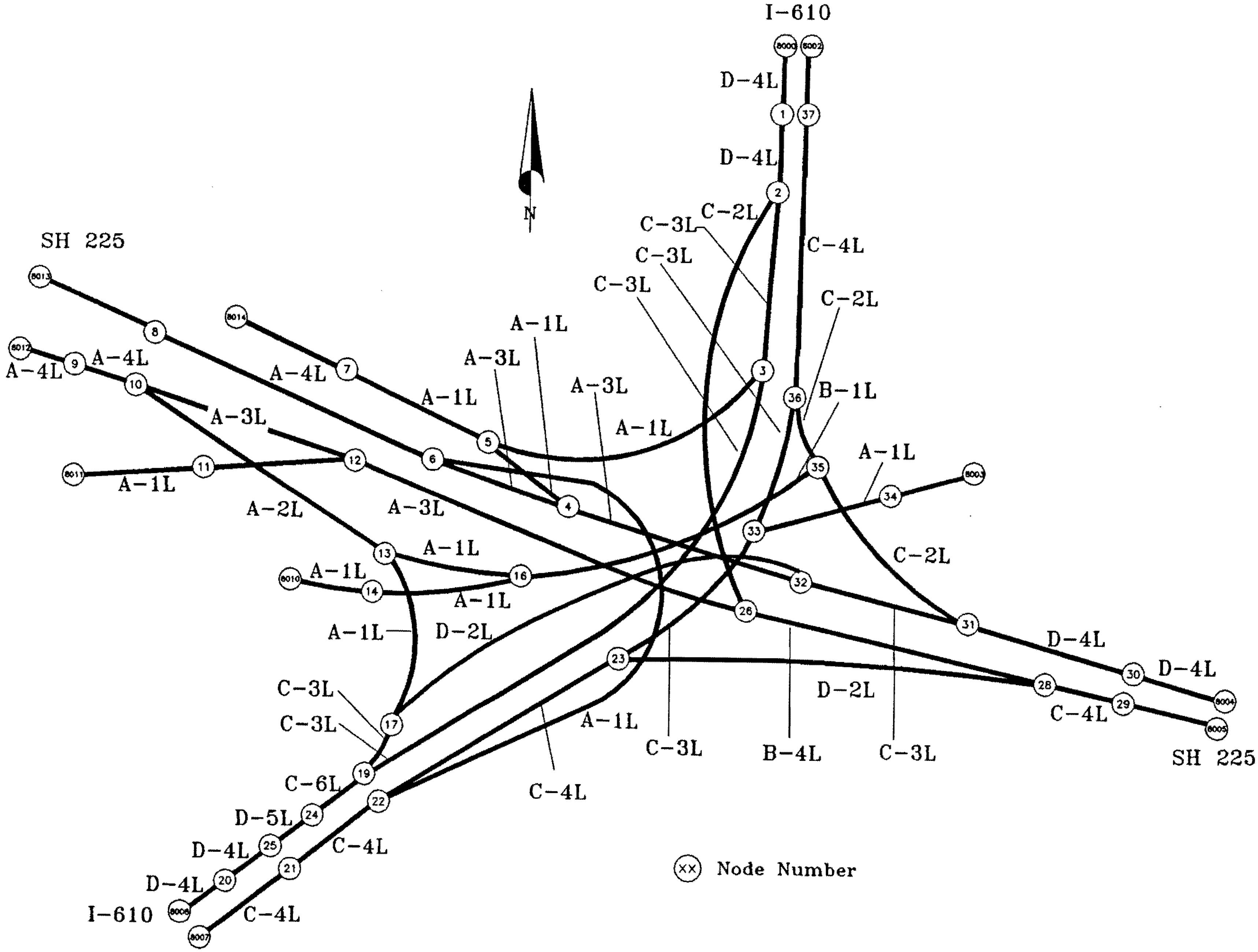


Figure 20. Line Drawing of Design Alternative for the H-6 Interchange

in this study. This interchange also had the second highest accident rate of the 36 interchanges in the study. Figure 21 shows that this facility is a fully directional interchange with single exit and entrance ramps. Heavy vehicles make up a large percentage of the traffic that uses this facility. Figure 22 shows the location of accidents in this interchange. The majority of the accidents at this interchange occur on the mainlanes near exit and entrance ramp terminals.

Selection of horizontal and vertical alignment at interchanges must take into account the operational constraints of heavy vehicles. The terrain at the location of the interchange, however, often limits the degree of flexibility afforded to the engineer in these areas. Existing literature and guidelines are available that provide the engineer with information in these areas. It is important that the engineer minimize the effects of horizontal and vertical alignment on heavy vehicles that use interchange facilities.

The presence of interior taper merge configurations at the entrance ramps presents a safety problem at this interchange. This problem is compounded because of the large number of trucks and the high overall demand at this interchange. Heavy vehicles have different operational characteristics associated with them as compared to passenger cars. These operational differences cause heavy vehicles inhibit to the merging and diverging activities of smaller vehicles. Larger gaps and longer distances are required, furthermore, for these vehicles to complete a merge maneuver.

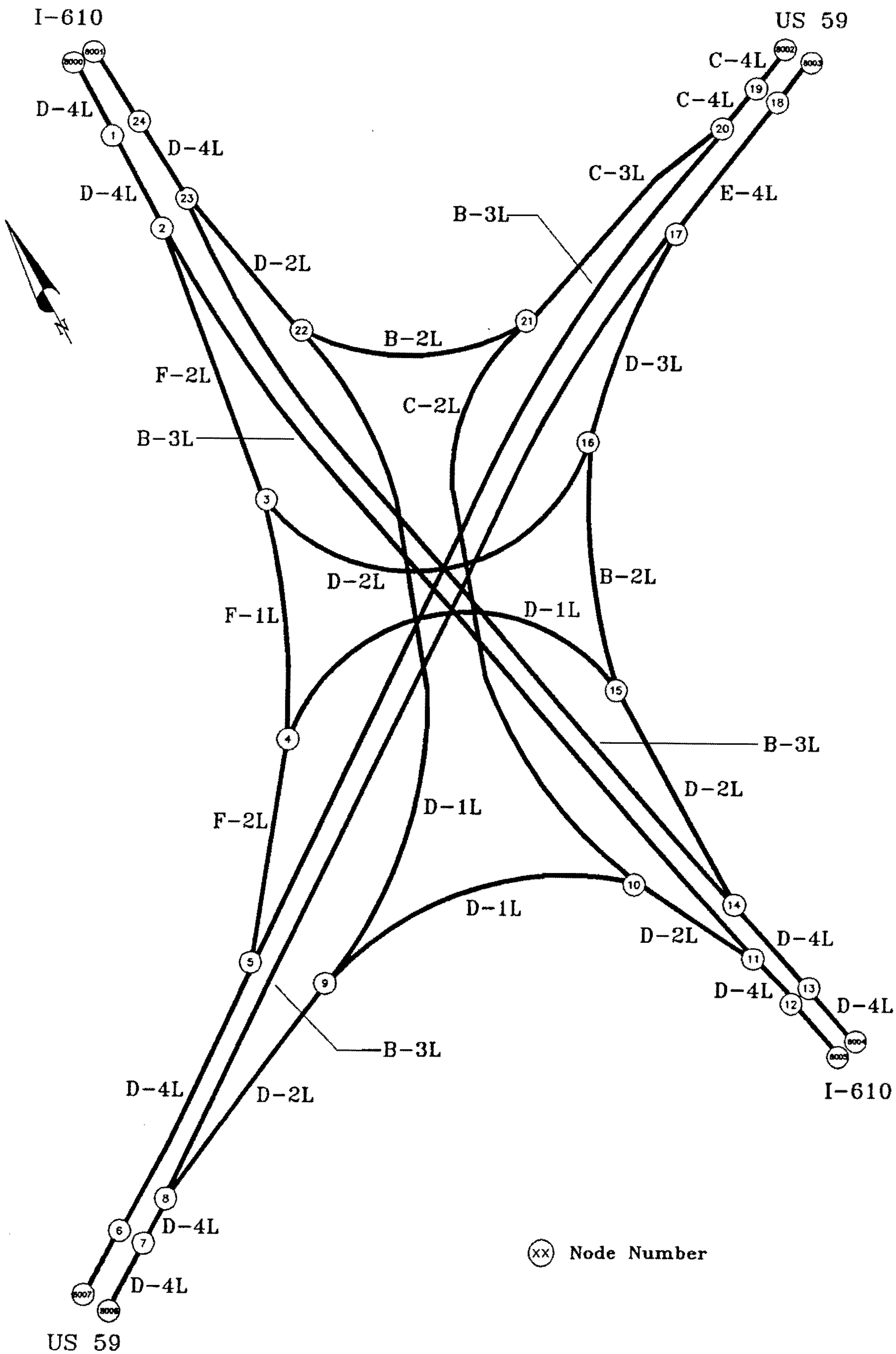


Figure 21. Line Drawing of the Existing H-3 Interchange

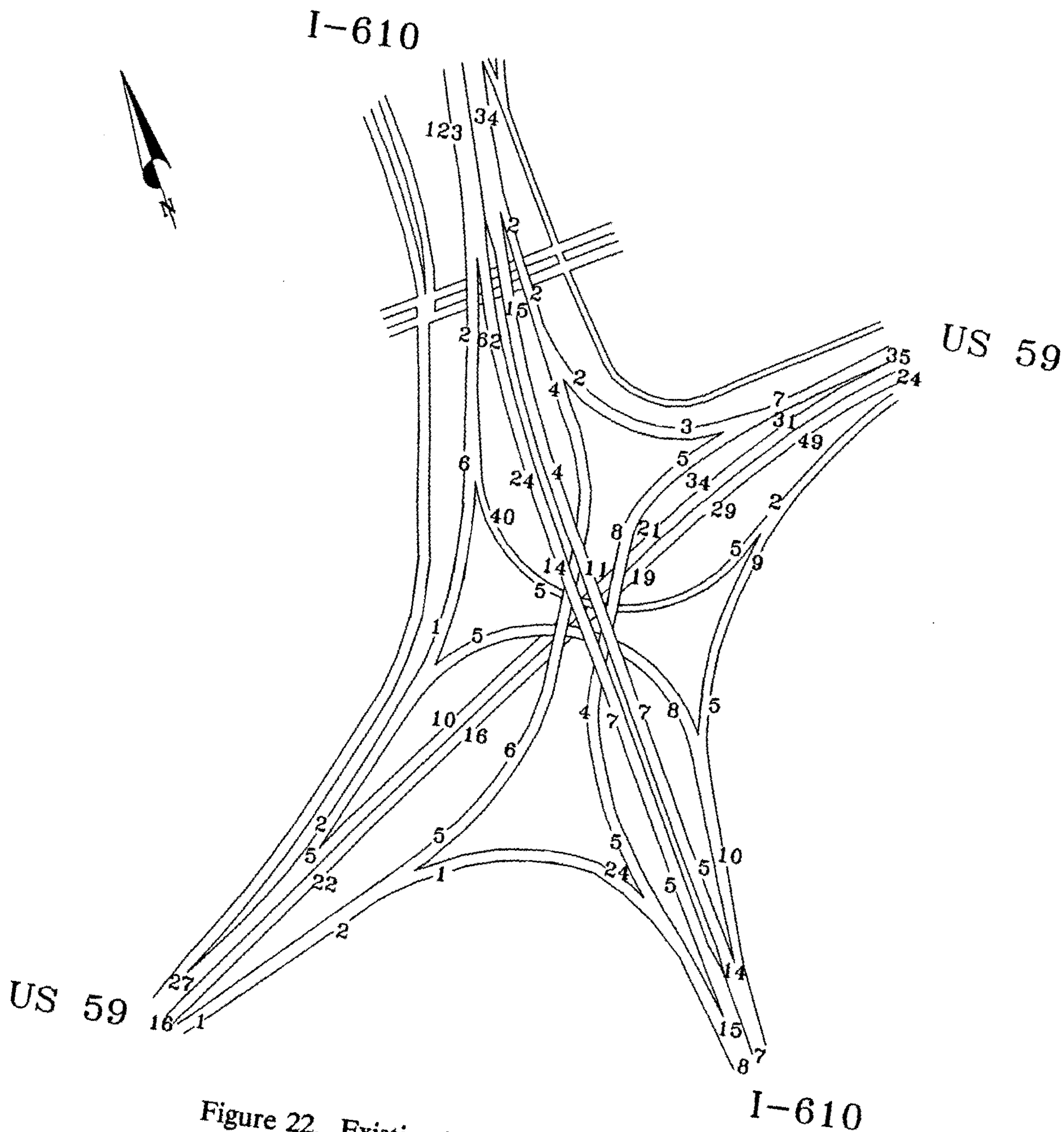


Figure 22. Existing H-3 Interchange With Accidents

Based on FREFLO output, six of the eight connectors and all of the exit and entrance ramps at this interchange operate at LOS D or worse in peak hour conditions. This correlates well with observed operations. A large portion of the traffic that uses this facility uses the freeway-freeway connections. The high number of accidents at exit ramps may result from ramp traffic backing onto the freeway, creating a speed differential on the mainlanes prior to these facilities.

Alternatives that would provide improved operations would be to increase the capacity of the ramps and connectors to accommodate the demand on them. The large amount of interchanging traffic also dictates the need for improved design of the merge terminals at this interchange. Exterior parallel ramp design should be used because of the high volumes of merging traffic and because of the different merging characteristics of the vehicles that make up the traffic stream.

Case Study - Route Continuity and Lane Balance

The H-13 interchange (US 290 and IH-610) was used to illustrate the subjects of route continuity and lane balance. This interchange is a fully-directional 3-leg interchange that consists of a north-south freeway (US 290) that terminates at a major loop freeway (IH-610). The accident rate for this interchange was not excessively high as compared to other 3-leg interchanges that were analyzed in the preliminary stages of this study. Locations within the interchange that exhibited high accident rates were associated with interior taper merge terminals and/or inadequate weaving sections. Figure 23 illustrates the existing alignment as well as the accident frequency at locations throughout the interchange.

Figure 24 illustrates estimates of the LOS obtained from FREFLO for each link in the interchange. Exit and entrance ramps attach several direct connectors to the frontage roads and the local street system. These ramps reduce the LOS on the connectors, thereby inhibiting their primary function of providing high speed operations for interchanging vehicles.

The design of this interchange also violates the principles of providing route continuity and a basic number of lanes. The north-south portion of IH-610 is connected to the east-west portion of IH-610 via two-lane direct connectors. The US 290 freeway mainlanes essentially become the mainlanes of the north-south portion of IH-610 (Figure 23). This configuration violates route continuity along IH-610, which is the major route through the interchange. The basic number of lanes along IH-610, furthermore, are not maintained through this interchange in either direction.

Figure 25 illustrates an alternative design that would provide route continuity along IH-610 and allow traffic going to and from US 290 to access IH-610 in a manner more consistent with route continuity principles. The frontage road connectors within the interchange have been moved out of the interchange. Additional lanes were provided between nodes 7 and 8, 8 and 9, 12 and 13, as well as between 13 and 4 so that three basic

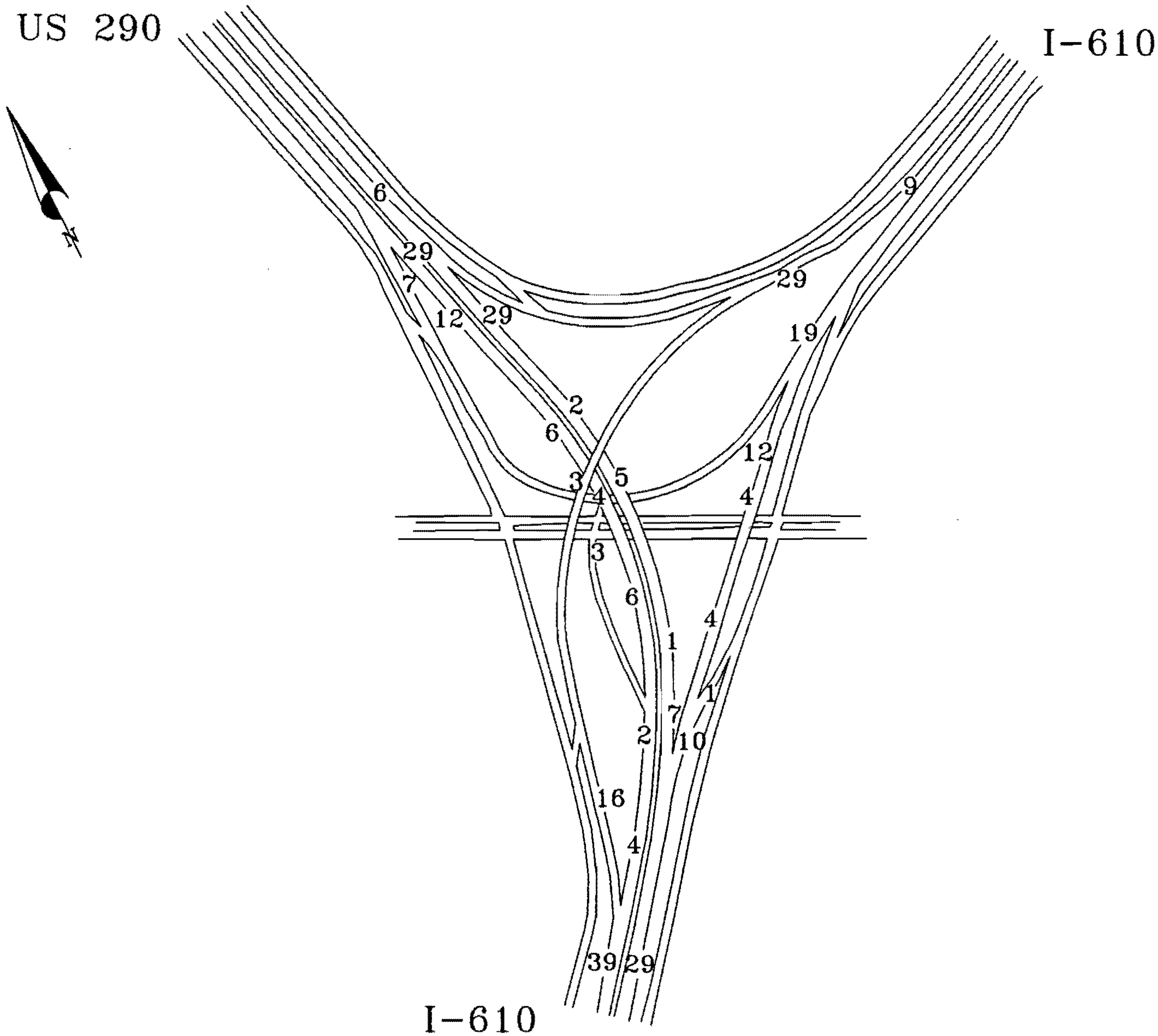


Figure 23. Existing H-13 Interchange With Accidents

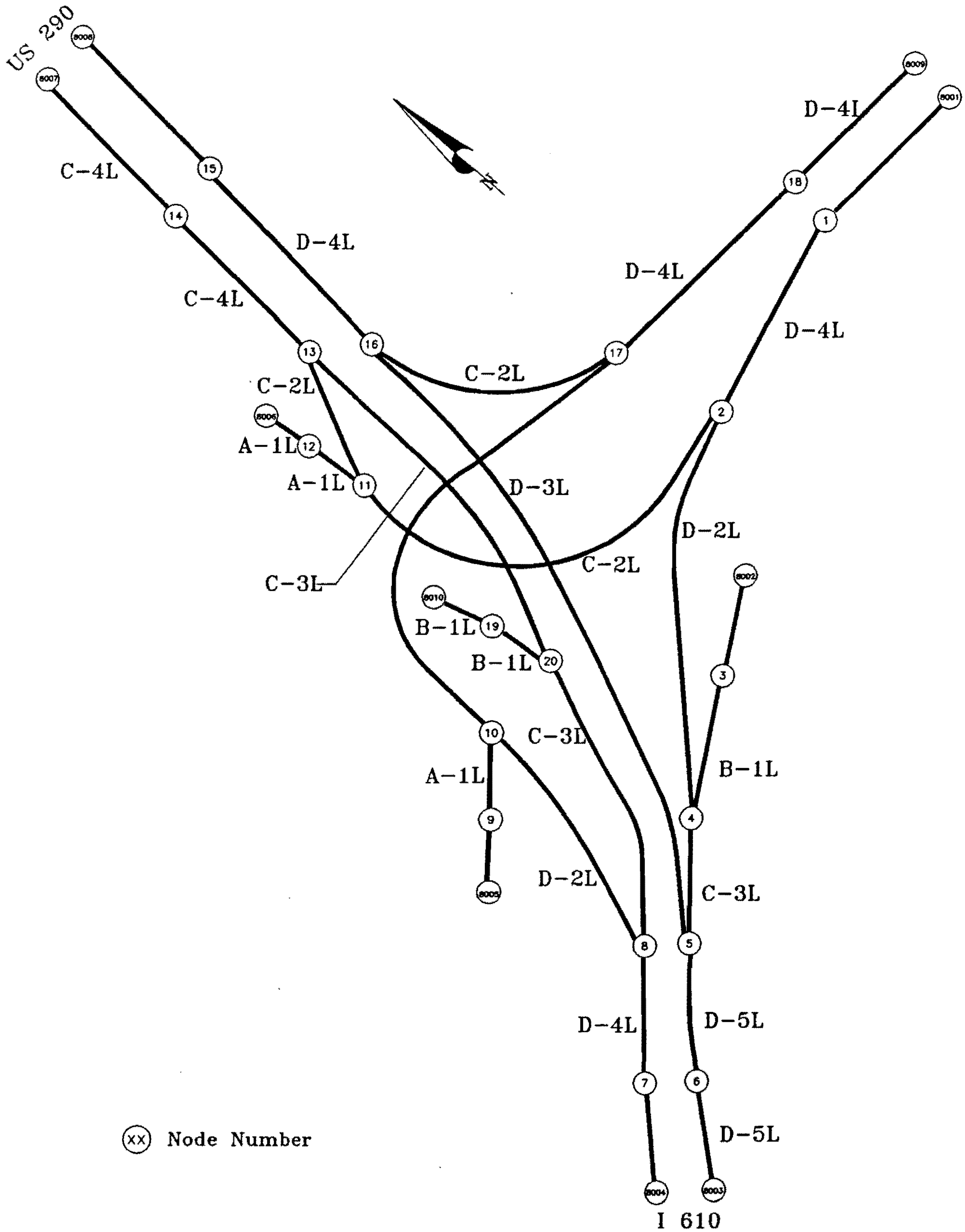


Figure 24. Line Drawing of the Existing H-13 Interchange

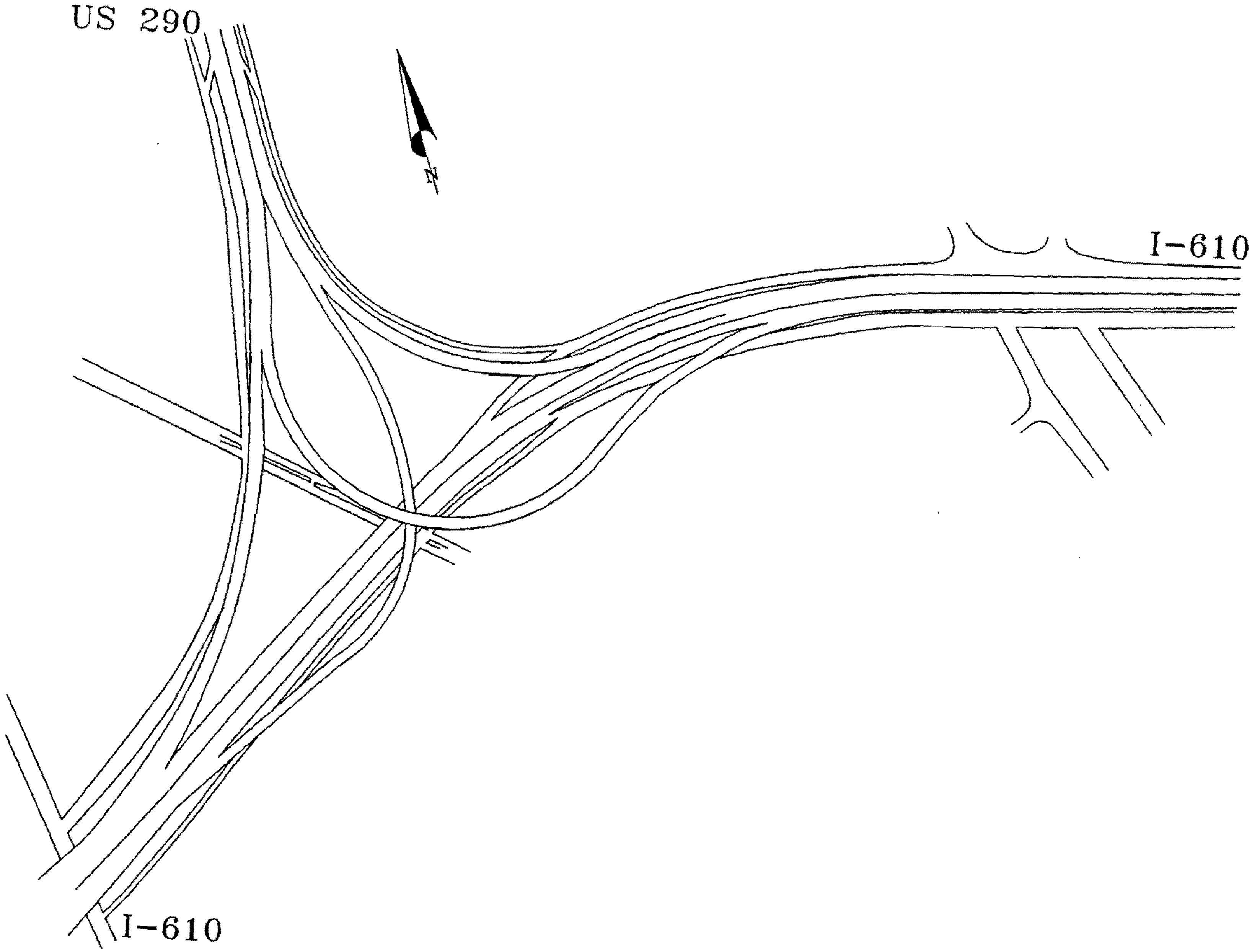


Figure 25. Alternative Design for the H-13 Interchange

lanes along IH-610 were maintained. The US 290 facility terminates at this interchange. A more functional interchange configuration can be obtained, therefore, by using freeway connections to attach the US 290 mainlanes to the IH-610 mainlanes.

Figure 26 illustrates the operational improvements associated with this alternative interchange design. The basic number of lane requirements previously discussed require that an additional lane be added to both directions along IH-610. The operational improvements indicated in Figure 26 reflect the addition of this capacity rather than the realignment of the facilities. The improved level of operations, however, re-emphasizes the importance of using functional design concepts in the design of interchanges rather than basing the design on projected freeway volumes. This alternative provides a more logical approach of maintaining route continuity on IH-610 and of terminating the US 290 facility.

Several ramps had been used to attach freeway-freeway connectors to the local street system in the existing design (Figure 23). This reduced the ability of the freeway-freeway connectors to accommodate interchanging traffic. Exit and entrance ramps from the local street system were attached to the freeway mainlanes within the interchange. Adequate distance was provided along the freeway between merge and diverge points. Providing adequate distance between ramps should eliminate mainlane weaving problems and minimize the degree to which traffic entering the freeway from these ramps affects interchanging vehicles.

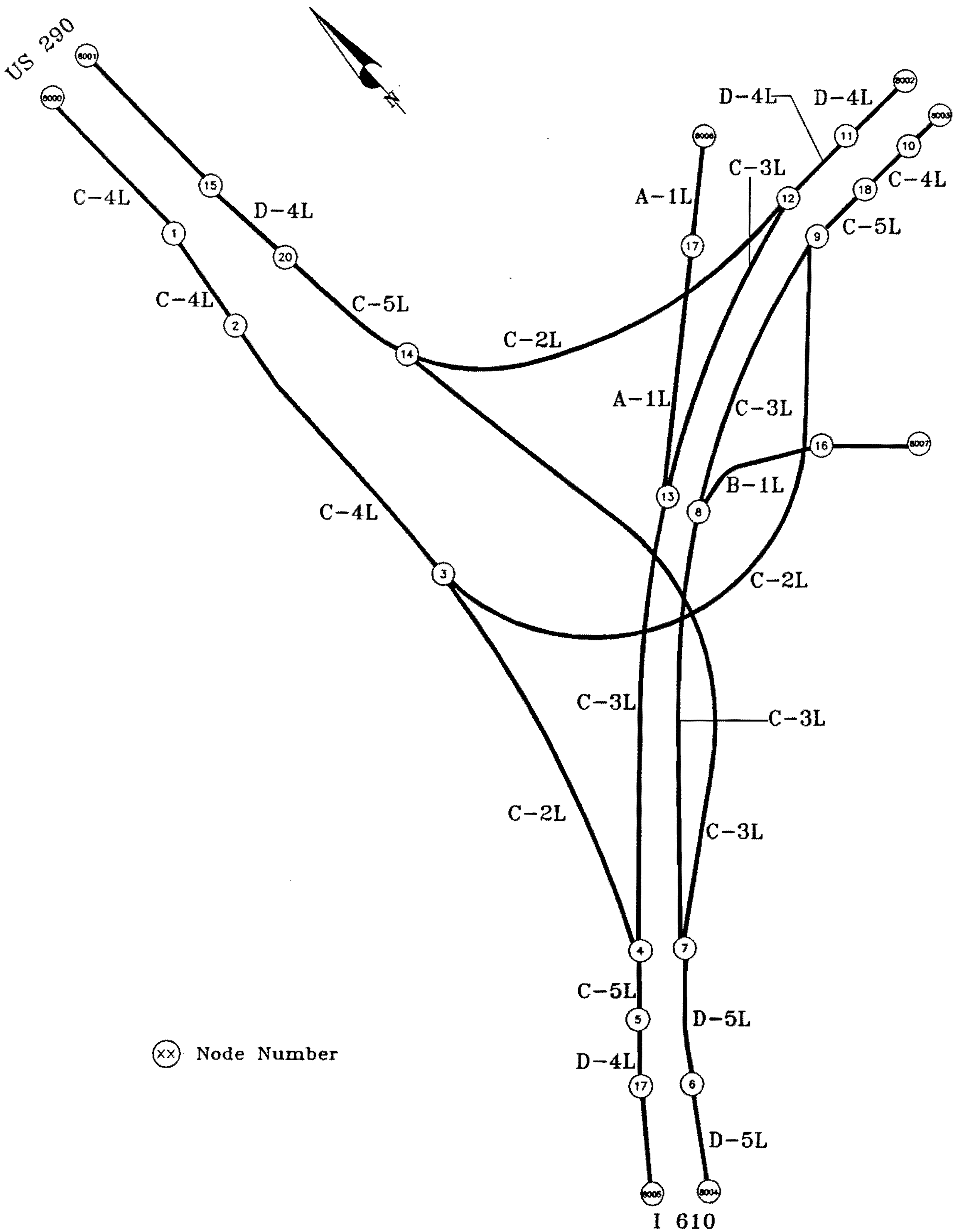


Figure 26. Line Drawing of design Alternative for the H-13 Interchange

Summary

These case studies demonstrate the inherent effects of interchange geometrics on interchange operations. Items that these case studies focused on included: 1) left-hand ramps; 2) weaving sections; 3) high traffic demand and truck volumes; and 4) route continuity, lane balance, and maintaining a basic number of lanes. Alternative design concepts were presented to illustrate methods of eliminating or minimizing undesirable conditions currently existing within the interchange configurations. As previously mentioned, the interchanges used in these case studies are only intended to serve as examples and were selected because of the high level of detailed geometric, operational, volume, and accident data available.

STUDY SUMMARY

The purpose of a freeway-freeway interchange is to facilitate interchanging movements between the two major high-volume, high speed highways and to minimize conflicting movements between these facilities. During the past 20 to 40 years, several types of interchange designs have been implemented. Table 7 discusses the specific advantages and disadvantages of two major types of four-leg interchanges. Similar advantages and disadvantages may be applied to three-leg interchanges with loop ramps and direct connectors. The type of interchange found in the field depends greatly on the location and topography of the interchange, as well as on the policies of the organization in charge of the design and implementation of the interchange, and available knowledge at the time of the original design. Geographical location has a strong influence on what type of design is used. Local drivers may come to expect certain ramp configurations.

Table 7. Comparison of the Major Four-Leg Interchanges

CLOVERLEAF INTERCHANGES	
Advantages	1. No at-grade intersections between the cross-streets and the ramp facilities. 2. Can accommodate higher volumes on the cross street that a diamond interchange can.
Disadvantages	1. Increases the potential for accidents typically found where large numbers of merging and diverging movements take place. 2. Excessive speed differentials exist under high volume conditions when collector-distributor roads are not used or when low speed ramps are designed. 3. Requires extensive amounts of right-of-way. The amount of right-of-way as well as the distance the driver must travel increases rapidly with increases in design speed. 4. A left-turn maneuver is made by going beyond the cross-street, exiting to the right and negotiating a 270° turn. This is typically an unexpected maneuver by the driver.
FULLY DIRECTIONAL INTERCHANGE (SINGLE AND DUAL RAMP DESIGN)	
Advantages	1. Requires less right-of-way than do cloverleaf interchanges. 2. Accommodates all movements. 3. Both designs are relatively easy to sign and present a more driver expectant alternative than do cloverleaf designs. Some mainlane weaving problems are eliminated via the use of single ramp design. 4. Provides higher capacity and higher speed exit and entrance facilities since higher design speeds can be obtained without the need for large amounts of right-of-way. 5. Reduce travel distances to make desired movements.
Disadvantages	1. Increased cost in amount of structure prohibits the use of these facilities to high volume facilities.

Figure 4 illustrates freeway-freeway interchange accidents as related to interchange ADT. Below 150,000 ADT there is little variance in the number of accidents occurring in the various interchange. This indicates that, within this ADT range, interchange type and configuration probably have a minimal influence on accidents. Figure 4 also indicates that once interchange ADT volumes exceed 250,000, there is a higher degree of variability in the number of accidents. Once ADT levels reach this range, interchange type and configuration may have a significant influence on accident rates.

Alternative Designs

This report evaluated design considerations that are important in the design and redesign of interchanges. Computer models were used to evaluate and compare specific interchange elements as well as several interchanges the were used in case studies to evaluate various other design factors. Alternative designs were used to demonstrate methods for minimizing or eliminating existing design and operational problems in typical "on-the-ground" interchanges. These case studies focused on:

- Left-hand ramps;
- Weaving sections;
- High traffic demands and high truck volumes; and
- Route continuity and lane balance.

Historically, the highest number of accidents within freeway-freeway interchanges occurs at exit and entrance ramp terminals. Consequently the design of these terminals must provide high-speed freeway egress and ingress, respectively, for large volumes of interchanging traffic. Adequate capacity must also be provided on ramps and connectors such that traffic exiting the freeway does not back onto the mainlanes, thereby deteriorating the level-of-service and safety on the freeway.

The three ramp configurations analyzed in the study were the interior taper merge configuration (Figure 8), the exterior taper merge configuration (Figure 12), and parallel ramp design (Figure 16). Analysis of these configurations using the freeway simulation model INTRAS indicated that the parallel ramp design provided the greatest benefit with respect to operations, followed by the exterior taper merge design and interior taper merge design configurations.

The parallel configuration provides a longer distance for drivers to select a gap in the traffic stream with which they are merging than does the outside taper configuration. The same is true for the outside taper configuration relative to an interior merge. Both the exterior parallel merge and outside taper merge configurations eliminate operational and safety problems associated with the inside lane merge characteristics of an interior merge.

Left-hand ramps have been associated in previous research (7, 8, and 9) with poor operational characteristics and higher accident rates (Table 2). Lundy's (9) research showed that left-hand ramps (both entrance and exit) had the highest accident rates of all ramp types considered. The accident analysis presented in this report was inconclusive in the regard of associating left-hand ramp with higher interchange accident rates.

Experience has shown that left-hand entrance ramps may be implemented and operated successfully with the proper design, one that includes an additional lane exclusively for these entering vehicles. Guidelines for appropriate lengths of the additional lane are given in both SDHPT Design (11) and AASHTO (3) manuals. Designers should consider avoiding left-hand ramps in new designs and remove them from existing interchanges where possible, even if special attention is given to signing and sight distance, since these facilities violate driver expectancy.

Weaving sections are defined as roadway segments where the path of travel of vehicles entering and exiting at contiguous points of access result in a crossing of vehicle paths. These segments may occur in interchanges as the result of ramp configuration or overlapping routes. The problems associated with these sections can be minimized by providing adequate separation between ramps or by using single ramp configurations to attach freeway-freeway connectors to freeway mainlanes.

This case study illustrated the considerations associated with accommodating high numbers of heavy vehicles at interchanges. The primary consideration associated with accommodating these vehicles is to provide good geometry on interchange facilities that take into account the different operational characteristics associated with heavy vehicles. The case study used to illustrate this point focused on the use of different types of merge terminals to provide more efficient merge activities. The use of exterior parallel ramp design was felt to reduce the safety problems associated with the entrance ramps at this facility.

The principles of route continuity, lane balance, and the provision of a basic number of lanes are interrelated. The AASHTO manual (3) provides a basic description of these principles. Applying these principles in interchange design assists in: 1) simplifying the driving task by reducing the number of lane changes, 2) simplifying the layout of signing, 3) providing proper definition of the route, and 4) reducing a portion of the drivers' work load of searching for directional signing. Figure 18 illustrates the relationship between the basic number of lanes and lane balance concepts. These concepts were applied in the case study of the H-13 interchange to illustrate the operational effects of their implementation.

Conclusion

Safety and operational breakdowns within freeway-freeway interchanges have been found to occur under high volume demands observed in urban areas of Texas. Specific geometric features exhibiting undesirable effects under designated conditions have been highlighted through analysis and evaluation. The potential level-of-service and safety associated with improvements have been presented and discussed. The efforts devoted in this study will hopefully direct and guide designers to focus on those geometric aspects of freeway-freeway interchanges which will optimize traffic operations.

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