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16. Abstract

This study reexamines the traditional use of the 30th highest hourly volume as the optimum design hour volume given the current era of limited funding, constrained right-of-way, environmental concerns, and increasing congestion. The report documents the Dallas/Ft. Worth region's use of an alternative design hour volume (4th highest hour of the day) based on funding constraints. The use of an alternative design hour volume requires that the planned system be optimized to manage peak person trip flows by identifying the best mix of general purpose, high occupancy and express lanes, and an aggressive program of transportation system management and demand management strategies.

A methodology is outlined to use a peak hour traffic model to forecast traffic volumes for different hours of the day. This approach has the potential to avoid some of the shortcomings of using average values for peak hour factors, directional split, and trucks to obtain hourly design volumes from daily forecasts.

If congestion is accepted during the peak hours of the day, then the design process for a facility must expressly consider congested conditions. Nine design elements that pose operational/safety concerns were examined under congested conditions to identify basic guidelines that should be considered when designing for congestion.

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# PLANNING FOR OPTIMAL ROADWAY OPERATIONS IN THE DESIGN YEAR

by

John C. Brunk, P.E.
Associate Research Engineer
Texas Transportation Institute

Mark D. Middleton
Assistant Research Scientist
Texas Transportation Institute

Keith K. Knapp
Graduate Research Assistant
Texas Transportation Institute

Carol H. Walters, P.E.
Research Engineer
Texas Transportation Institute

Timothy J. Lomax, P.E.
Research Engineer
Texas Transportation Institute

H.S. Oey, Ph.D.
Associate Professor of Civil Engineering
University of Texas at El Paso

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TEXAS TRANSPORTATION INSTITUTE
The Texas A&M University System
College Station, Texas 77843-3135

## IMPLEMENTATION RECOMMENDATIONS

1. This study was sponsored by TxDOT because the current era of limited construction funds, constrained right-of-way, environmental concerns, and rapidly increasing congestion have created a need to reexamine the traditional use of traffic estimates for the 30th highest hourly volume (HHV) in facility planning and design.

   a. To satisfy federal requirements, some TxDOT Districts and MPOs will need to develop a cost constrained Design Hour Volume as the basis for their regional transportation planning and facility design. However, there are benefits to developing both a "needs based" plan and a "cost constrained" plan to help frame a future transportation system. While the cost constrained plan must be the regionally approved transportation plan, the needs based plan can be used to help identify requirements for congestion management strategies, right-of-way preservation, project staging beyond the adopted plan, and to help make adjustments in the plan if additional transportation funding sources are identified.

   b. The potential for error inherent in the adjustment factors used to obtain hourly volumes from ADT forecasts could be more than 50 percent. This suggests that the approach used by the NCTCOG to select an alternative hour of the day as the Design Hour Volume in their regional transportation planning process might be enhanced through the direct forecasting of peak hour volumes, so that hourly adjustment factors do not need to be used. The report describes a rationale that can be used to factor the daily trip table to obtain a trip table for any hour of the day using data from a region's household travel survey.

2. In the past, TxDOT has designed highways for freeflow conditions using the 30th HHV for estimating future volumes. However, future designs in urban areas will not be able to satisfy peak hour demands, and a more constrained approach of accepting congestion

will be necessary. Designers must consider congestion as a factor in the design of future freeways.

a. A primary cause of congestion are elements that appear unexpected to drivers. To satisfy driver expectancy and to promote safety, TxDOT needs to maintain uniformity in design. Desirable freeway geometries, consistent use of signing, ramps located on the right, standard interchange designs, and route continuity should be considered to maintain uniformity.

b. To optimize traffic flow under congested conditions, TxDOT should make use of operational aids (e.g. ramp meters, incident detection and response, changeable message signs, lane control signals) and provide access to alternative routes in future designs.

c. So that facilities can be adjusted over time to meet changes in demand, TxDOT should use flexible freeway designs. Some examples of design features that could help ensure flexibility include shoulders built to the same pavement standards as travel lanes, clear span bridges at cross streets, and two-lane direct connector ramps between freeways.

d. Locations where vehicles interact on freeways such as at merging, diverging, or weaving areas also cause congestion. The standard designs for freeway elements as recommended by AASHTO operate adequately for most congested conditions. However, some elements are preferred and appear to provide a more orderly flow of traffic. For example, the parallel design single-lane entrance ramp is slightly preferred over the taper design because of the narrow lane width, and it is compatible with the introduction of an auxiliary lane. Table 10 in the report summarizes TTI's findings from the observed design elements.

These guidelines should be disseminated to TxDOT districts and metropolitan planning organizations (MPO) serving urbanized areas as well as to consulting engineers so they may be used in the major investment study (MIS), environmental (NEPA), and final design phases of project development.

# 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 of the Texas Department of Transportation (TxDOT) or the Federal Highway Administration (FHWA). This report does not constitute a standard, specification, or regulation, nor is it intended for construction, bidding, or permit purposes. The engineer in charge was Carol H. Walters, P.E. #51154.

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# SUMMARY

The current era of limited construction funds, constrained right-of-way, environmental concerns, and rapidly increasing congestion have created a need to reexamine the traditional use of the 30th highest hourly volume in facility planning and design. There will not be enough resources to build freeways that can provide free flow conditions during peak hours of the day. The purpose of this study was to address two key issues: (1) the basis of the 30th hour as the design hour volume and the implications of using alternative hours of the day as the design volume for future roadways, and (2) the identification of freeway elements that pose operational/safety concerns when designing for congestion and the development of suggested methodologies to guide design for safe operation.

The requirement to financially constrain regional transportation plans under the Intermodal Surface Transportation Efficiency Act (ISTEA) will force many urban areas to make difficult transportation project decisions. The use of an alternative design hour volume is a viable approach for urban areas that do not have sufficient funds to plan and design facilities for free flow conditions. The development of the "system plan approach" in the Dallas/Ft. Worth area suggests that the most cost effective system is one that maximizes person movement by utilizing a mix of general purpose, high occupancy vehicle (HOV), and express lanes that encourage mode shifts during congested peak hour travel. This report provides the rationale for regional transportation plans (RTP) to be developed using a lower DHV and identifies a methodology for directly forecasting hourly volumes for different hours of the day.

The use of a lower design hour volume means that during the peak hours of the day, freeways will be congested. Designers in the future will need to consider congested conditions in addition to design standards established for freeflow conditions when designing freeways. Several freeway design elements were identified and observed under congested conditions, including entrance and exit ramps, auxiliary lanes, lane reductions, weaving areas, collector-distributor roads, branch connections, and major forks. Some basic guidelines were developed that should be considered when designing for congestion.

# I. INTRODUCTION

The Clean Air Act Amendments of 1990 (CAAA) and the Intermodal Surface Transportation Efficiency Act of 1991 (ISTEA) have significantly altered the decision making environment within which urban roadways are planned and designed. These legislative acts mandate the consideration of the physical, multimodal, environmental impacts, cost-effectiveness, and funding sources of virtually all urban roadway projects. Whereas, roadways have traditionally been planned and designed to provide free flow conditions in the design year using the 30th highest hour as the design hour volume (DHV), limited construction budgets, constrained right-of-way, air quality concerns, and rapidly increasing congestion are forcing a reexamination of this process.

TTI and the University of Texas at El Paso (UTEP) undertook this joint study to begin to address two key issues:

1. What is an appropriate design hour volume for planning and designing transportation facilities in urban areas when there are not sufficient resources available to provide freeflow levels of service? This study examines the basis for the traditional use of the 30th highest hour as the DHV and explores the implications of using lower alternative DHVs focusing primarily on the experiences of the Dallas/Fort Worth region.

2. What are the implications of designing roadways for peak period congestion? Design standards have primarily been developed to ensure safe and efficient operation under freeflow conditions. This study identifies freeway elements that are particularly stressed when a roadway is congested and suggests design principles that can be used to guide the process of designing for congestion.

# II. DHV BACKGROUND

The DHV assumed, estimated or used in practically all the literature reviewed was the 30th highest hourly volume (HHV) expected in the design year of the facility. The use of this particular DHV is based upon its typical location near the "knee" of an ordered plot of HHVs existing or predicted on a facility. In urban areas, however, the "knee" of this type of curve is usually difficult to identify, and the typical weekday peak hour, 10th, 20th, and 200th HHV have been suggested for freeway planning and design purposes (*1*, *2*). Fortunately, due to the assumed regularity and predictability of urban freeway traffic flow, all of these DHVs typically produce the same roadway design. The "knee of the curve" method and the 30th HHV are emphasized in the two most utilized references in the transportation planning and design fields (*3*, *4*). Bridge design, signal timing, capacity analysis, and roadway geometric design (e.g., roadway sizing) are some of the many procedures that require the use of DHVs.

In general, a review of the literature discovered research related to the 30th HHV, the "knee of the curve" method, and several alternative DHV estimation procedures. No documentation was found that indicated the uniform or official acceptance of a DHV other than the typical 30th HHV, or the use of any alternative DHV selection methods. The only reference to the use of an alternative design hour has been a suggestion by the Dallas/Fort Worth region that the 4th highest daily hour (approximately the 1000th HHV) be considered. This chapter, therefore, focuses on the origins and use of the 30th HHV in freeway design, and some suggested alternative DHV estimation methods.

## HISTORICAL BACKGROUND

The ultimate objective of a roadway plan or design is to accommodate, at a reasonable level of service, the amount of traffic a facility can expect to carry during its design year. The use of hourly volumes to represent these levels of design year traffic was first advocated over 70 years ago. In 1921, Johnson stated that the average daily traffic (ADT) ". . .throughout the year does not give the number of vehicles that should be provided for due to the seasonal and hourly variations in the volume of traffic" (*5*). The American Association of State Transportation Officials (AASHTO) continues to support this position in its 1994 "Green Book" by recommending that the traffic volumes during time periods of less than a day reflect ". . .the operating conditions that should be used for design. . ." and "[i]n nearly all cases a practical and adequate time period is one hour" (*4*). These hourly volumes are then adjusted for the peak 15-minute rates of flow and used for roadway sizing and level of service analysis. "The selection of an appropriate hour for planning, design, and operational purposes is a compromise between providing an adequate level of service for every (or almost every) hour of the year and economic efficiency" (*3*).

The selection of a DHV from the "knee" of an ordered HHV curve was first proposed by Peabody and Norman in 1941 (*6*). Based on traffic volume data from 89 rural arterial highway locations, they plotted the number of hours that exceeded various hourly volumes for different levels of annual average daily traffic volumes. From this graph, Peabody and Norman concluded that for a highway with average traffic fluctuations it would not be practical to design for a volume greater than that exceeded 30 times each year, and ". . .that little will probably be saved in the construction cost and a great deal lost in expediting the movement of traffic if a design is used that will not handle the traffic volume exceeded during the 50 peak hours" (*6*). The economic and level of service reasons for initially choosing this range of highest hourly volumes (HHV) (the "knee" of the curve) for design purposes appears to have been intuitive and subjective. Unfortunately, this reasoning continues to be the basis for the current use of the 30th HHV in both two-lane roadway and freeway design. Peabody and Norman did, however, warn the reader that this range of HHVs would not necessarily apply to all locations (*6*).

The data originally analyzed by Peabody and Norman was later combined with information from additional automatic traffic recorder locations (for a total of 167 in 48 states) and evaluated within the 1950 *Highway Capacity Manual* (*7*). This data was used to produce the now familiar plot shown in Figure 1 which continues to be used by AASHTO in 1994 (*4*).

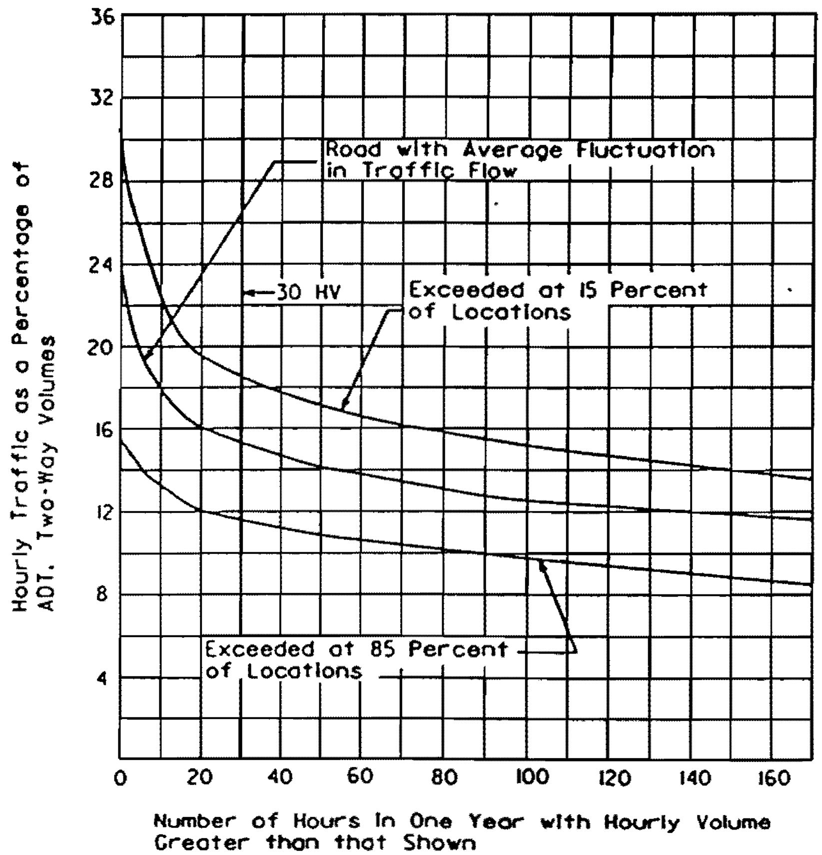



Figure 1. Relation Between Peak-Hour and Average Daily Traffic Volumes on Rural Arterials
Source: AASHTO, A Policy on Geometric Design of Highways and Streets

Based on Figure 1, and a certain amount of intuition, AASHTO recommends that the 30th HHV be used to design rural arterial roadways with average traffic flow fluctuations (*4*). AASHTO believes that the use of this particular DHV is reasonable because of the differences in design that would result from the choice of a slightly higher or lower HHV. Figure 1 shows that the volumes appear to change rapidly from the 1st to the 30th HHV, but much more gradually after the 30th HHV. In other words, the 30th HHV appears to be the point of maximum curvature (or "knee") on the graph. The 30th HHV is also recommended for use as a DHV because of its relatively consistent relationship over time with the ADT of a roadway (*4*). It was speculated that this consistency allowed the design year 30th HHV to be more easily estimated from existing volume relationships.

The validity of this consistency has been questioned (see the next section of this report). AASHTO also suggests that a DHV equivalent to 50 percent of the volumes expected during a few maximum design year hours be used for roadways with unusual or highly seasonal traffic flow fluctuations (*4*).

The 30th HHV is also recommended by AASHTO for use as a DHV in the planning and design of urban arterial roadways. This recommendation is related to, but not based on, the selection of a HHV near the "knee" of the curve shown in Figure 1. In fact, the difference between the 30th and 200th HHVs on urban arterial roadways is very small, and the "knee" is difficult to pinpoint (*4, 8*). An example of this is shown in Figure 2. In general, AASHTO's urban freeway DHV recommendation is predicated on the belief that the uniformity and predictability of the traffic flow on this type of facility allows the DHV to be calculated by averaging the highest weekly afternoon peak hour volumes over one year. This average is equal to the 26th HHV if the morning peak flows are generally less than the afternoon peaks, or approximately the 50th HHV if they are reasonably equal. The roadway plans and designs produced by the 26th and 50th HHV, however, are not usually significantly different than those produced by using the 30th HHV. Neither alternative HHV forces a change in design. Therefore, the 30th HHV can also be used as the DHV on urban arterial roadways.

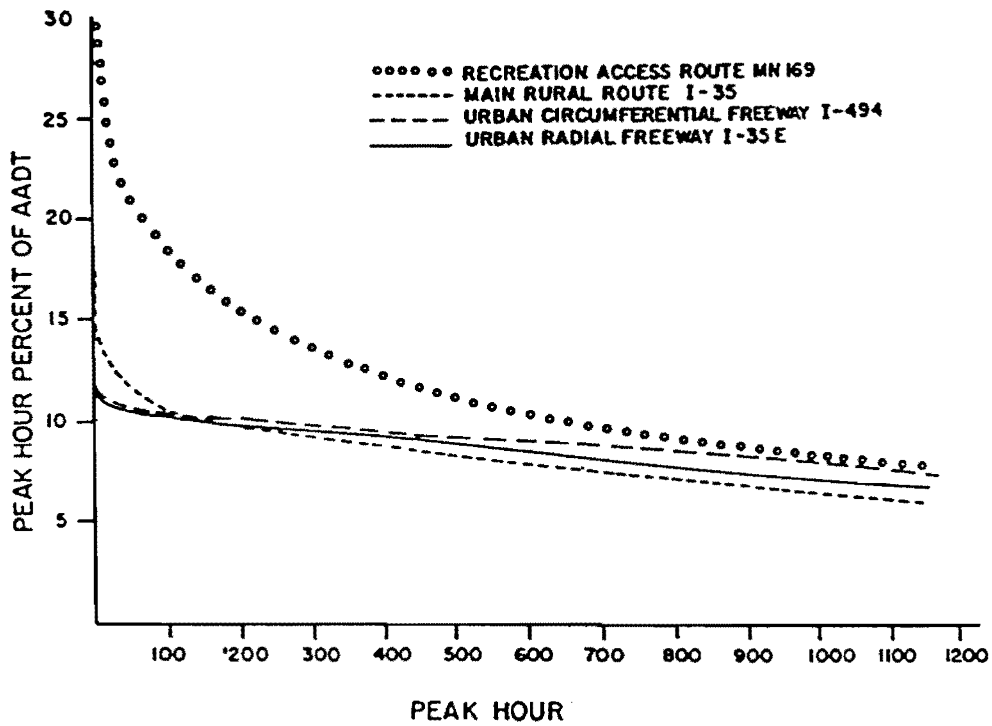



Figure 2. Ranked Hourly Volumes by Type of Roadway
Source: TRB, Highway Capacity Manual

## LIMITATIONS OF CURRENT APPROACH

The previous section summarized the basis for the current use of the 30th HHV as a DHV in urban freeway design. The reasoning behind its selection appears to be somewhat lacking, and several limitations to its use have been addressed in the literature. Some of these limitations apply to all types of roadways, but others are more relevant to the urban environment. These limitations are discussed in this section.

Several concerns have been raised about the use of the 30th HHV and the application of the "knee of curve" method in the selection of DHVs. The first and most important concern is that there does not appear to be any objective evidence that the 30th HHV, or the other DHVs found through the "knee of curve" method, are the most economical or efficient for design purposes (*9*, *10*). Secondly, the assumption that the 30th HHV/ADT ratio does not change over time is not entirely valid. In 1957, Walker found that the 30th-hour factor (a ratio of the 30th HHV and ADT) declined slightly over time (*11*). This conclusion was supported later by Bellis and Jones (*12*). The 1994 *Highway Capacity Manual* also includes the following conclusions about the 30th-hour factor (i.e., the K-factor) (*3*):

- K-factors generally decrease as annual average daily traffic (AADT) increases;
- The reduction rate of high K-factors is faster than low values;
- K-factors decrease with increases in development density; and,
- The highest K-factors typically occur on recreational roadways, followed by rural, suburban, and urban facilities, in descending order.

The ability to identify the "knee" of a HHV distribution is also a concern. In many cases, the identification of this point is very difficult and requires a significant amount of judgement (*8*, *13*).

Figure 2 shows that the identification of this "knee" can be a special problem for urban roadways. The nonexistence of a discernible "knee" in certain situations, and/or the possibility of the "knee" occurring at volumes other than the 30th HHV is supported by a number of references (*1*, *10*, *13*).

Additional concerns pertaining to the current approach of selecting DHVs include 1) the difficulty that exists with distinguishing between normal or average traffic flow fluctuations, and unusual or highly seasonal patterns, and 2) the fact that the current approach compares one design volume with one design capacity (i.e., service volume) at a given point of time (i.e., design year) (*8*). Also, a DHV for divided arterial roadways should be on a directional basis. The current approach for divided roadways requires the adjustment of two-way ADT values by factors created from bidirectional data. Walters, et al. have studied the effects of several variables on the estimation of a directional K-factor in large urban areas (*14*). They concluded that HHVs on a directional basis are highly variable in time. Sharma, et al. came to the same conclusion with their research on rural highways, and in another study showed that directional 30th HHVs can be very different than the 30th HHV for the entire roadway (*15*, *16*, *17*). Sharma is currently working on a study to estimate directional DHVs on multilane highways from automatic traffic recorder data and sample traffic counts (*18*).

## ALTERNATIVE APPROACHES

The "knee of curve" method is the most commonly used and accepted approach of determining DHVs. Typically, it involves the use or estimation of a HHV (sometimes and typically assumed to be the 30th HHV) for the design year of a roadway. The "knee of curve" approach is based on the facility (i.e., it provides a certain level of service for the majority of the design year hours) rather than the user. Several alternatives to the "knee of curve" approach have been suggested, and are described in the following paragraphs.

### Cost-Effectiveness Approach (*8*, *9*, *13*, *14*)

This approach is more detailed than the "knee of curve" method. It attempts to take the benefits and costs of alternative roadway improvements into account by selecting a DHV based on the results of a cost-effectiveness analysis. The impacts of each alternative improvement are analyzed and evaluated. One of the most important impacts evaluated is the economic efficiency or benefit-cost ratio of each alternative. This type of analysis requires a complete distribution of hourly flows for each vehicle type (in some cases by direction) over the life of the roadway, and improved techniques to estimate the user-costs sensitive to traffic volumes and design features (*9*). In their study, Crabtree, et al. concluded that current procedures do not always yield the most economical highway size; they recommend the inclusion of an economic analysis in the sizing of highways (*13*). Another cost effectiveness approach was developed by Walters, et al., which seeks to identify the lowest total public cost alternative in each corridor for a given volume of peak hour person trips. This methodology recognizes that some motorists will shift between mixed flow, HOV, and express lanes depending on the level of congestion; by balancing construction/operation costs and travel delay costs, the optimum lane combination can be identified for a facility.

### Use of a Range of Hours (*8*, *9*, *14*)

This approach is based on the belief that the design of a facility should not be based solely on the use of the 30th HHV. This is especially true when capacity and level of service are important to the decision making process (e.g., urban freeways). It is suggested that a range of the highest hours of traffic flow be used to evaluate alternative facility designs that provide the desired level of service.

**DHV from the User's Perspective (*8*, *9*, *14*)**

This approach is oriented toward the number of users that experience congestion on a roadway rather than the number of hours a facility is congested. The selection of the 30th HHV as a DHV accepts the possibility that the roadway may experience 29 hours (more or less) of operation with levels of congestion higher than desired. This method does not consider the percentage of roadway users that may experience this level of congestion. An alternative approach is to determine the DHV that provides a certain level of service to a specified percentage of user hours. It has been suggested that this is a more equitable approach, from the roadway user's perspective, than the traditional "knee of curve" method.

Another approach to predict the DHV of a roadway from the road user's perspective has been suggested by Sharma, et al. (*19*, *20*). These researchers believe that the type of roadway use or nature of travel along a roadway has a significant impact on its K-factor. Roadway use, therefore, should be considered a significant variable in the prediction of DHVs (*19*, *20*). Sharma, et al. have developed linear regression models based on monthly and daily traffic data to predict the DHV of a roadway from the user's perspective (*20*). Their conclusion was that the models they developed could predict an observed user DHV as accurately as the traditional methods could predict either the 30th or some other HHV (*20*).

**Traffic Assignment Models**

In general, traffic assignment models are currently used to forecast the ADT along roadways in large urban areas. However, three approaches can be used to estimate peak period or peak hour travel demands. They include the factoring of 24-hour trip tables, the factoring of 24-hour trip ends, or direct generation (i.e., peak hour or period traffic assignment) (*21*). In 1988, Benson, et al. developed a software package that introduced peak period (three hour) traffic assignment capabilities to the Texas Travel Demand Model Package (*21*). This software was based on data from Houston, Texas. The applicability of this peak period traffic assignment model to other areas was not addressed.

There are at least two important points that must be made with respect to the estimation of peak hour volumes with regional planning models. Most regional planning models are based on the concept of daily flow equilibrium (i.e., people go to work and then come home), and used to forecast roadway ADTs. A traffic assignment forecast of peak hour or period traffic volumes, on the other hand, requires that directional traffic data be collected, calibrated, and forecast. In addition, the accuracy and reliability of the ADT forecasts produced by current planning models has never been validated. Therefore, their ability to forecast more detailed directional design year peak hour or period traffic flows should also be questioned. The current assumption is that a properly calibrated model will produce accurate design year traffic forecasts. Another phenomenon worth mentioning is that of peak spreading. At least two articles were discovered that discussed the estimation and effects of peak spreading (*22*, *23*). This phenomenon occurs along highly congested roadways, and must be considered when choosing an appropriate DHV for this type of highway.

The next section of this report addresses the current federal and state polices toward the planning and design of roadways, and the use of DHV or its alternatives. In general, it was found that the recommended or accepted approach to the planning and design of urban freeways has not changed significantly in the past half century despite the DHV research done and the dramatic changes that have occurred within this decision making environment. Some policy adjustments have been made due to the mandates of ISTEA and the CAAAs, but the references used for actual planning and design have remained the same.

# III. DHV REGULATION AND POLICY

## FEDERAL

The mandates of the CAAA of 1990 and the ISTEA of 1991 have dramatically changed the factors that must be considered in the planning and design of urban roadways. The CAAA requirements are intended to ensure that the implementation of transportation facilities do not significantly delay the attainment of air quality health standards, and that funds are designated for projects that support this goal. All transportation projects in nonattainment areas must conform to CAAA requirements. ISTEA shifts much of the responsibility for transportation project planning decisions to the local metropolitan planning organization (MPO) level. In addition, it requires the identification of the environmental effects, cost-effectiveness, and funding sources for each transportation project planned in a metropolitan area. All transportation improvement plans (TIPs) at the local and state-level must also be financially-constrained (i.e., only projects with reasonably available funds are allowed in the plan). The impact of the ISTEA and CAAA mandates described above require a reexamination of the factors that most influence the planned and designed characteristics (e.g., number of lanes) of a proposed roadway. The most significant of these factors is the DHV.

The Federal Highway Administration (FHWA) evaluates and approves or disapproves all roadway projects that request federal funding. In general, the DHV selection and calculation procedures and processes described in the 1994 AASHTO Green Book must be followed for official FHWA approval. However, if more stringent planning and design procedures are adopted by a state, then these regulations have precedence. In the case of Texas, the procedures documented in the *Highway Design Division Operations and Procedures Manual* must be followed (*24*). These procedures are discussed in the next subsection of the report.

In recent years, the official position of the FHWA on the use, selection, and calculation of alternative DHVs has been somewhat uncertain. As indicated above, AASHTO policies and procedures must at least be followed for federal approval. Unfortunately, the recent mandates of ISTEA and the CAAAs, the lack of right-of-way, increased congestion, and the relatively permanent nature of transportation systems have made the actual application of these AASHTO procedures somewhat unrealistic in dense or congested urban areas. The FHWA has commended the innovation of a recent freeway/high-occupancy-vehicle system plan in Dallas, which accepted higher than typical levels of congestion (i.e., an alternative DHV), but suggested that this approach be used with caution. Additionally, a recent telephone conversation with an FHWA representative indicated that they would not automatically disapprove a TIP based on an alternative DHV. Therefore, it would appear that the officially approved use, selection, and calculation of the DHV in the planning and design of roadways has not changed, but the requirements for an approval of a TIP (and the projects it contains) have become more flexible (with respect to the role of DHV). The effect of this flexibility on the role and choice of DHVs in the planning and design of urban freeways (especially those in nonattainment areas) has not been investigated, and is one of the objectives of this research.

## STATE

The *Highway Design Division Operations and Procedures Manual* clearly defines what and how DHVs should be used in the planning and design of roadways in Texas (*24*). Specifically, the Texas Department of Transportation (TxDOT) designates the 30th HHV of a design year, typically 20 years from the time of construction, as the DHV that should be used. TxDOT does warn that some adjustment of this DHV might be necessary for situations with high traffic volume fluctuations. The manual also clearly describes the procedure that should be used to determine the DHV from a design year ADT estimate. Generally, the ". . .instructions in this Manual shall take precedence over AASHTO standards" (*24*).

Three traffic flow parameters are taken into account in the conversion of a ADT forecast to a DHV. They include the percentage of ADT that represents the 30th HHV or DHV (K-factor or DHV factor), the percentage of DHV traffic expected in the predominant direction of flow (D-factor), and the percentage of trucks expected to occur in the DHV (T-factor). K-factors for typical rural highways are normally between 12 and 18 percent, but between 8 and 12 percent for urban roadways. Directional traffic may be relatively equal in urban areas on circumferential roadways. However, it is not uncommon to measure a D-factor in the 60 to 70 percent range. The percentage of trucks is also included in the conversion of an ADT to a DHV because they require more space on a roadway and have lower operating capabilities than passenger cars. To account for this, the expected percentage of trucks is converted to equivalent passenger cars. This is done through the application of equivalency factors ($E_t$). These factors are provided in the *Highway Design Division Operations and Procedures Manual* and are dependent upon the type of facility, the type of terrain, and the estimated percentage of trucks at specific locations (*24*). T-factors can vary widely; in highly urbanized areas during peak periods, in the peak direction, truck percentages have been found to average as low as 1 to 2 percent on heavy commuter routes in Texas, although off-peak percentages are much higher (*14*). Equivalency factors are given for both an approximate analysis of general highway sections and for the analysis of more specific roadway grades. The equivalency factors listed in the *Operations and Procedures Manual* are similar to those in the 1994 HCM (*3*, *24*).

The traffic flow characteristics described above (i.e., ADT, K, D, and T) are provided to the Design Division by the Transportation Planning Division of TxDOT. These parameters and the truck equivalency factors given in the *Operations and Procedures Manual* are used to calculate the DHV. The DHV calculated can then be used to design intersections; determine the type of facility necessary and the number of lanes it will require; and analyze the operation (i.e., LOS) of mainline, ramp, and weaving sections. The equation used to convert a two-way ADT to a directional DHV is as follows:

$$DHV = (ADT)(K)(D)[1 + T(E_t - 1)]$$

Where: DHV = Design hour volume,

ADT = Average daily traffic (two-way),

K = Percentage, expressed as a decimal, of the ADT representing the 30th HHV or DHV,

D = Percentage, expressed as a decimal, of the ADT in the predominant direction of travel,

T = Percentage, expressed as a decimal, of trucks in the DHV,

$E_t$ = Passenger car equivalents for trucks.

This equation requires an adjustment when the ADT volumes are provided for a divided roadway on a directional basis. The entire calculation must be multiplied by a factor of two. This doubling is necessary because the D-factor applies only to bidirectional flow, giving the percent in the peak direction. Thus, doubling a directional ADT approximates the equivalent bidirectional ADT.

The previous paragraphs describe the existing federal and state polices toward the use and calculation of DHV in the design and planning of roadways. The approaches recommended at the federal and state level are basically equivalent and have not changed significantly in many years. In Texas, the procedures described in the *Highway Design Division Operations and Procedures Manual* should be followed; however, there are some changes needed in the values used (*24*).

# IV. DHV CASE STUDY: TRANSPORTATION PLANNING IN THE DALLAS/FORT WORTH REGION

The North Central Texas Council of Governments (NCTCOG) is the Metropolitan Planning Organization (MPO) for transportation for the Dallas/Ft. Worth area. The *Mobility 2010 Plan Update* (*25*) was developed by NCTCOG in 1993 in response to requirements in ISTEA and was an update of the regional transportation plan previously approved in 1990. In particular, the updated plan satisfied the ISTEA mandate that all long-range transportation plans must be financially constrained. Most MPOs across the country, including the NCTCOG, have struggled to reconcile transportation needs with available funding, especially with regard to their long-range transportation plans. Some MPOs have dealt with this problem through one or more of the following strategies (*26*):

- more optimistic revenue assumptions;
- revised cost estimates for projects;
- more aggressive deferral of maintenance;
- more optimistic expectations of operational efficiency; and,
- more optimistic forecasts of transit ridership.

While the NCTCOG also sought to be more rigorous in the estimation of project costs and future funding, they are perhaps the only MPO in the country that has adopted an alternative DHV in the development of their Metropolitan Transportation Plan. A review of this region's philosophy, planning methodology utilized, and an example of how this approach has affected a local project is instructive in seeing how alternative DHVs can be used in the current planning and design environment.

## SYSTEM PLANNING STUDY

The Dallas System Planning Study methodology was jointly developed by TxDOT, Dallas Area Rapid Transit (DART), NCTCOG, and TTI to provide an intermediate step between the system level planning carried out by TxDOT's regional planning office and NCTCOG, and the detailed corridor design done by TxDOT's district office (*27*).

The original study developed a methodology that selects the multimodal corridor alternative that best serves peak hour person demand at the lowest total cost to the public. These public costs were identified as person-trip congestion delay costs, capital costs of construction and right-of-way, and operating costs of HOV lanes. There have been two subsequent studies that have refined and extended the methodology to include environmental effects (e.g., fuel consumption, noise abatement) and nonrecurrent congestion (*28, 29*).

The system plan approach showed that the most cost effective transportation system is one that accepts some congestion on freeways during peak hours and offers carpools and transit for persons wanting an alternative (*27*). In effect, it suggested that the DHV used to size and design a facility for freeflow conditions should be from an hour of the day that has less traffic than the peak hour. It was this finding that led the NCTCOG to consider the use of an alternative DHV in order to satisfy the financial constraint requirement of ISTEA.

## REGIONAL PHILOSOPHY

Historically, it has been the goal of the NCTCOG to develop a RTP that would return the region to 1980 travel conditions, when most of the region's freeway system operated at level of service C conditions even during peak travel periods. However, based on the Mobility 2010 Plan adopted in 1990, there was a need for $16.5 billion in transportation improvements to maintain 1980 levels of mobility over the next twenty years, but only $8.9 billion in estimated revenue (*25*).

Given the $7.6 billion difference between needed funds and available resources, the NCTCOG identified two primary strategies to guide the development of the plan (*25*):

1. The development of a financially constrained plan that adopts a change in the traditional planning approach of identifying a system based solely on mobility needs.

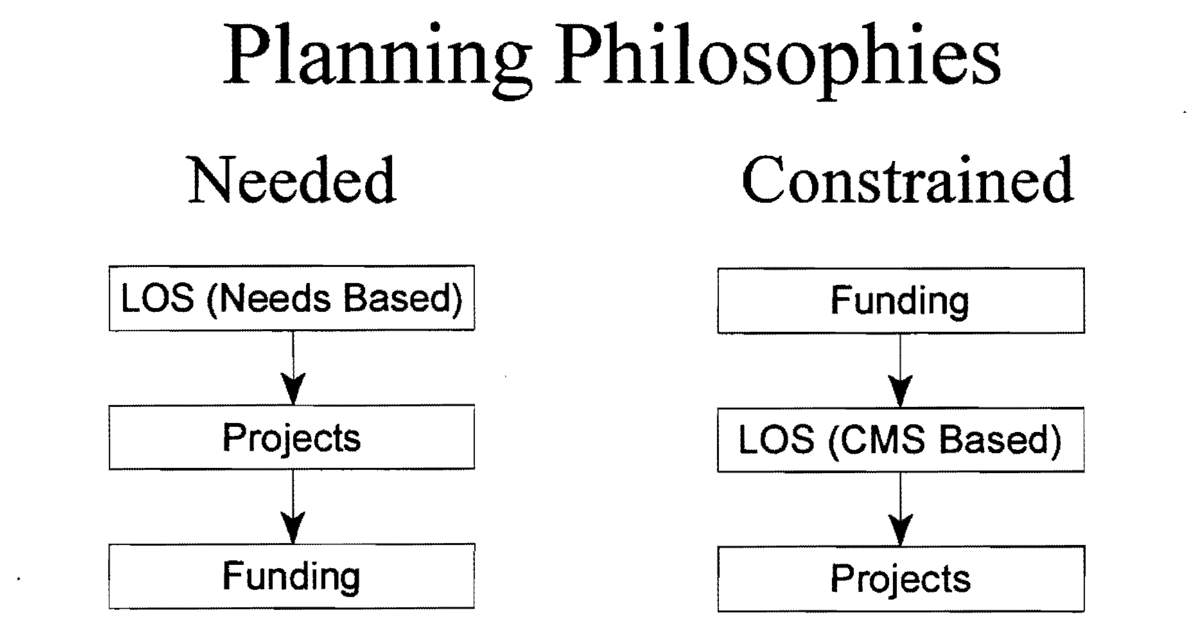
2. A process that identifies additional funding and management strategies to achieve a balance between available funds and mobility needs.

Fundamentally, this resulted in a philosophical shift from a "needs based" approach to a "congestion management based" approach (Figure 3). Instead of developing a wish list of transportation projects by first identifying needs, then projects to satisfy those needs, and finally, calculating the cost, the approach now would be one in which the available funds are estimated first, then the level of service that can be achieved based on that funding level, and finally a set of projects developed to achieve this level of service.

The key difference between the two philosophies was a willingness by the region to accept a reduced level of service in the future transportation system. If the traditional design hour volume and level of service criteria were used to identify projects for the plan, then the region would be forced to make some very difficult choices to include a limited number of projects in the RTP and leave many others out. Instead, the region adopted a process in which the DHV chosen for planning and design purposes would be based on what it could afford; for the *Mobility 2010 Update* this was the 4th highest hour of the day. This approach had two major effects on the planning process:

1. The RTP identified more small freeway improvement projects rather than a few large projects, and

2. Since the freeway system was not being planned to accommodate peak hour demand, there was an emphasis on CMS strategies to reduce peak period single occupant vehicles.




Source: North Central Texas Council of Governments, 2010 Mobility Plan Update

Figure 3. NCTCOG Alternative Planning Philosophies

**REGIONAL TRANSPORTATION PLAN METHODOLOGY (*25*)**

This section describes the part of the NCTCOG transportation planning process for the *Mobility 2010 Update* that relates to their methodology for determining which hour of the day to use as the DHV. NCTCOG made an initial determination to solve for capacity needs associated with the peak, third, fourth, and sixth highest hours of the day. It was believed that the third and fourth hours would be within the peak period, while the sixth highest hour would be at the beginning of the off-peak period.

The NCTCOG methodology begins with output from their 2010 daily (24-hour) traffic assignments and then converts those forecasts to hourly estimates by freeway segment using the same basic formula that TxDOT uses to estimate the 30th HHV (discussed in Section II). In the model, facilities are classified by "area type" to distinguish between some of their operating characteristics. Adjustment factors for peak hour (K), directional split (D), and trucks (T) had already been developed by area type for application in the *2010 Regional Transportation Plan* using Automatic Traffic Recorder (ATR) data collected by TxDOT in 1980, 1986, and 1990. The ATR data was examined further to identify K factors for other hours of the day. The D and T factors were held constant for each hour of the day; it was assumed that the D factor would decline and the T factor would increase in going from the peak hour to other hours of the day, but that these changes would essentially cancel each other out. The adjustment factors used by NCTCOG for the peak hour and the 4th hour are summarized in Tables 1 and 2. Once these factors were applied to the daily volume to obtain estimates of traffic for each target hour (peak, 3rd, 4th, and 6th), the number of freeway lanes needed to maintain level of service C was determined by dividing the hourly volumes by a service volume of 1,600 passenger cars per hour.

NCTCOG used an iterative approach in comparing the results of the hourly volume analysis/lane warrants with their financial analysis to determine that the fourth highest hour of the day was the appropriate DHV given expected financial constraints. In conjunction with this analysis, NCTCOG was evaluating the effects of implementing TSM, TDM, and non-SOV strategies such as HOV and rail transit in order to manage the transportation system during the peak travel hours of the day. Finally, NCTCOG worked with TxDOT and the Texas Turnpike Authority (TTA) to assess issues including cross-section feasibility, available right-of-way, lane balancing, and toll road feasibility to determine specific recommendations for the freeway system component of the Plan.

Table 1. Directional Design Hourly Volume Factors - Peak Hour of the Day

| Area Type | | Directional Factor (D) | Peak Hour Factor (K) | Truck Factor (T) |
|---|---|---|---|---|
| Central Business District | (1) | 0.53 | 0.089 | 1.067 |
| Outer Business District | (2) | 0.56 | 0.089 | 1.067 |
| Urban Residential | (3) | 0.57 | 0.090 | 1.080 |
| Suburban Residential | (4) | 0.58 | 0.095 | 1.108 |
| Rural | (5) | 0.60 | 0.092 | 1.151 |

Source: North Central Texas Council of Governments

Table 2. Directional Design Hourly Volume Factors - Fourth Hour of the Day

| Area Type | | Directional Factor (D) | Peak Hour Factor (K) | Truck Factor (T) |
|---|---|---|---|---|
| Central Business District | (1) | 0.53 | 0.068 | 1.067 |
| Outer Business District | (2) | 0.56 | 0.068 | 1.067 |
| Urban Residential | (3) | 0.57 | 0.068 | 1.080 |
| Suburban Residential | (4) | 0.58 | 0.068 | 1.108 |
| Rural | (5) | 0.60 | 0.068 | 1.151 |

Source: North Central Texas Council of Governments

**S.H. 161 EXAMPLE**

State Highway 161, from I-20 to S.H. 183, is planned as a limited access freeway with frontage roads on a new north-south alignment through the cities of Grand Prairie and Irving. Prior to the development and adoption of the *Mobility 2010 Plan Update*, S.H. 161 was being planned and designed as an eight to ten lane freeway (eight lanes between I.H. 20 and I.H. 30, and ten lanes north of I.H. 30). In the regional transportation plan prepared under ISTEA, S.H. 161 was identified as four to six lane facility using the 4th highest hour of the day as the DHV and a 2010 planning horizon.

A June, 1985 lawsuit resulted in an injunction and a requirement for TxDOT to prepare new environmental documents for the proposed freeway. Since this project was already in the NEPA process when ISTEA and MIS regulations were approved, it was categorized as a "pipeline project." This meant that the project development process was reviewed to determine what additional work would be needed to meet the requirements of ISTEA without starting over. One of these requirements was that a single occupant vehicle justification be conducted to determine if a targeted program of TSM and TDM measures could reduce the need for additional freeway lanes. Although the analysis showed that there was a potential to reduce DHV demand by about ten percent, this was not sufficient to reduce the number of lanes.

TxDOT adopted a strategy in the preparation of environmental documents to evaluate impacts associated with an ultimate buildout of S.H. 161 with 8 to 10 lanes even though its initial construction would be limited to 4 to 6 lanes as identified in the RTP. This approach was based on the following considerations:

- The initial construction of S.H. 161 with 4 to 6 lanes (based on DHV using 4th highest hour of the day) was viewed as the first stage of construction toward an ultimate facility with 8-10 lanes (based on DHV using 30th highest hour of the year);

- Right-of-way for the full 8 to 10 facility should be obtained with the first stage of construction to minimize any subsequent impact on adjacent properties;

- TxDOT wanted to be up-front in the environmental review and public participation processes about the full impacts associated with this transportation investment;

- If additional transportation funds become available and the RTP is updated to include more lanes on S.H. 161, TxDOT wants to be able to respond by providing a better facility without reopening the environmental review process; and,

- Federal CAAA requirements would still be protected in that no project larger than that currently identified in RTP can be built unless the regional plan is updated and a new conformity analysis completed.

The FHWA has not issued a record of decision on S.H. 161 as this report is being written. They are concerned that the environmental documentation on the project does not match the facility identified in the RTP and tested for air quality conformity. Representatives from NCTCOG and TxDOT are meeting with FHWA to present the rationale discussed above.

## SUMMARY OF KEY ISSUES

The NCTCOG experience highlights several issues that should be considered by any region contemplating the use of an alternative DHV:

1. Use of a lower alternative DHV will allow a region to plan and implement improvements for more facilities/corridors than would be possible using the 30th HHV as the DHV.

2. Broad political support may be possible because a larger number of planned projects will likely touch more jurisdictions.

3. Although planning for a lower level of service, the transportation system should provide consistent performance (i.e., there should not be bottlenecks between a facility designed for a high DHV feeding into another facility that has not been improved at all because of a lack of funds).

4. Acknowledgment that the planned transportation system will have some congestion during peak hours must be accompanied by development of a mix of facility types (general purpose, HOV, express lanes) to take advantage of the mode shifts that can occur when there is congestion to maximize the person carrying capacity of the system.

5. There is greater emphasis on management of the transportation system in lieu of capital improvements; in addition to the mix of facility types, there must be an emphasis on aggressive TSM and TDM strategies to maximize the reliability of the transportation during peak periods.

6. Development of a "needs based" plan as well as the "congestion management based" (cost constrained) plan will help to identify ultimate right-of-way needs. This is particularly important for new facilities on new alignments where it would be appropriate to obtain all the right-of-way that will eventually be needed during the initial stages of construction.

7. Morning and afternoon peak hour traffic assignments are needed in addition to the DHV because elements of a facility will be "stressed" at different times.

# V. DHV SELECTION

The requirement to cost constrain transportation planning under ISTEA has had far reaching effects. Many urban areas will probably find themselves with significant funding shortfalls as they continue to develop their regional transportation plans within the ISTEA era. For this reason, a cost constrained DHV will probably be a necessity for most regions to satisfy federal requirements.

However, there are benefits to developing both a "needs based" plan and a "cost constrained" plan to help frame a future transportation system. While the cost constrained plan must be the regionally approved transportation plan, the needs based plan can be used to help identify requirements for congestion management strategies, right-of-way preservation, project staging beyond the adopted plan, and to help make adjustments in the plan if additional transportation funding sources are identified.

One of the outcomes of ISTEA has been that more of the detailed information that would normally be available for the design process is being required much sooner as part of the planning process. Transportation planning needs to develop more sophisticated processes and tools to keep up with the information demands that are being placed on it. The methodology utilized by the NCTCOG in the *Mobility 2010 Plan Update* to identify and evaluate the effect of using traffic volumes for different hours of the day was an effective approach given the time constraints under which they were operating. However, it has been noted in a previous study by Walters (*14*) that even if ADT forecasts were exact, the potential for error inherent in the adjustment factors to obtain hourly volumes could be more than 50 percent. This suggests that the approach used by NCTCOG in the RTP process might be enhanced through the direct forecasting of peak hour volumes, so that the hourly adjustment factors do not need to be used.

## PEAK HOUR ASSIGNMENT

The NCTCOG has had a peak hour roadway traffic assignment model for a number of years. As the requirements of ISTEA have increased the need for more detailed traffic information during the long-range planning and major investment study, there has been renewed interest in utilizing the peak hour model.

The peak hour assignment process requires the use of different volume-delay equations, a peak-period roadway network, and a peak hour trip table. In order to obtain a peak hour trip table, peak hour distribution factors by time-of-day (morning or afternoon), trip purpose (Home-Based Work [HBW], Home-Based Nonwork [HNW], Nonhome-Based [NHB], and Other trips) and trip orientation (production versus attraction) are applied to the daily production-attraction person-trip tables before the tables are converted to origin-destination vehicle-trip tables (*30*). The distribution factors are obtained from the 1984 household survey.

The peak hour assignment model is currently being used to generate hourly volumes for the Trinity Parkway Major Investment Study. The model is being validated for speeds, volumes, and directional distributions based on data that was already available from other studies. The results of the validation have been promising. Tables 3 and 4 show a comparison between the observed and modeled directional splits for the morning and evening peak hours, respectively.

## DESIGN HOUR VOLUME ASSIGNMENT

The ability to make traffic assignments for the morning and afternoon peak hours suggests that traffic assignments can be done for any hour of the day. Once the hourly assignment model has been validated for the peak hour, the daily trip table can then be factored for the 2nd, 3rd, or 4th highest hour of the day and those trips assigned to the roadway network. The first step is to determine the number of trips occurring in each hour of the day. Table 5 shows the trip distributions by time period and purpose that are obtained from a "time-sliced" origin-destination

Table 3. Trinity MIS Peak Hour Validation / AM Peak Hour Directional Distributions

| Location | Direction | 1990 Observed (%)[1] | 1995 Modeled (%) |
|---|---|---|---|
| I.H. 35E, S. of CBD | NB/SB | 69/31 | 68/32 |
| I.H. 30, W. of CBD | WB/EB | 41/59 | 40/60 |
| I.H. 35E, W. of CBD | NB/SB | 65/35 | 58/42 |
| I.H. 35E, N. of CBD | NB/SB | 59/41 | 55/45 |
| I.H. 35E, N. of DNT | NB/SB | 58/42 | 54/46 |
| Woodall Rodgers | WB/EB | 59/41 | 50/50 |
| I.H. 45, E. of CBD | NB/SB | 84/16 | 80/20 |
| I.H. 30, E. of CBD | WB/EB | 67/33 | 63/37 |
| I.H. 45, S. of CBD | NB/SB | 80/20 | 76/24 |
| I.H. 30, Canyon | WB/EB | 55/45 | 50/50 |

Source: North Central Texas Council of Governments [1]TTI/TxDOT

Table 4. Trinity MIS Peak Hour Validation / PM Peak Hour Directional Distributions

| Location | Direction | 1990 Observed (%)[1] | 1995 Modeled (%) |
|---|---|---|---|
| I.H. 35E, S. of CBD | NB/SB | 40/60 | 38/62 |
| I.H. 30, W. of CBD | WB/EB | 68/32 | 55/45 |
| I.H. 35E, W. of CBD | NB/SB | 45/55 | 44/56 |
| I.H. 35E, N. of CBD | NB/SB | 45/55 | 48/52 |
| I.H. 35E, N. of DNT | NB/SB | 43/57 | 46/54 |
| Woodall Rodgers | WB/EB | 38/62 | 43/57 |
| I.H. 45, E. of CBD | NB/SB | 12/88 | 29/71 |
| I.H. 30, E. of CBD | WB/EB | 39/61 | 46/54 |
| I.H. 45, S. of CBD | NB/SB | 25/75 | 39/61 |
| I.H. 30, Canyon | WB/EB | 50/50 | 49/51 |

Source: North Central Texas Council of Governments [1]TTI/TxDOT

Table 5. Distribution of Person Trip Start Times

| Military Time (Trip Start) | HBW | HNW | NHB | Other |
|---|---|---|---|---|
| 00-01 | 0.57% | 0.42% | 0.12% | 0.42% |
| 01-02 | 0.21% | 0.21% | 0.15% | 0.21% |
| 02-03 | 0.17% | 0.06% | 0.11% | 0.06% |
| 03-04 | 0.22% | 0.02% | 0.08% | 0.02% |
| 04-05 | 0.60% | 0.03% | 0.02% | 0.03% |
| 05-06 | 2.45% | 0.26% | 0.18% | 0.26% |
| 06-07 | 10.52% | 1.09% | 0.39% | 1.09% |
| 07-08 | 19.16% | 6.54% | 1.68% | 6.54% |
| 08-09 | 9.64% | 7.45% | 2.90% | 7.45% |
| 09-10 | 2.88% | 3.58% | 5.17% | 3.58% |
| 10-11 | 1.58% | 4.88% | 7.06% | 4.88% |
| 11-12 | 1.55% | 4.90% | 10.57% | 4.90% |
| 12-13 | 2.39% | 4.10% | 15.09% | 4.10% |
| 13-14 | 1.97% | 4.30% | 9.87% | 4.30% |
| 14-15 | 2.94% | 5.17% | 9.42% | 5.17% |
| 15-16 | 6.00% | 8.71% | 8.82% | 8.71% |
| 16-17 | 12.30% | 7.63% | 7.21% | 7.63% |
| 17-18 | 12.89% | 8.10% | 7.19% | 8.10% |
| 18-19 | 5.03% | 9.77% | 3.88% | 9.77% |
| 19-20 | 1.96% | 8.13% | 4.37% | 8.13% |
| 20-21 | 1.27% | 5.70% | 2.67% | 5.70% |
| 21-22 | 1.69% | 5.74% | 1.51% | 5.74% |
| 22-23 | 1.17% | 2.23% | 1.31% | 2.23% |
| 23-24 | 0.85% | 0.98% | 0.21% | 0.98% |
| Weekday | 100.00% | 100.00% | 100.00% | 100.00% |

Source: North Central Texas Council of Governments table for the NCTCOG region. The cells in this table are multiplied by the number of daily trips for each trip purpose and each row summed to obtain the total number of trips occurring during each hour of the day.

Using NCTCOG person trip data from their latest regional validation run for the year 1995 (Table 6), the results of this calculation can be graphed as shown in Figure 4. As expected, the 24-hour profile is essentially the same as would be found for most urban freeway facilities— morning and afternoon peaks with the highest peak in the afternoon, and an additional smaller peak at midday. Graphing this same data in descending order (Figure 5) clearly identifies the peak hour and each subsequent hour of the day from a system-wide standpoint.

Table 6. Daily Person Trips

| Trip Purpose | Person Trips |
|---|---|
| HBW | 3,288,090.00 |
| HNW | 6,158,272.00 |
| NHB | 3,506,304.00 |
| Other | 1,871,548.00 |
| Total | 14,824,214.00 |

Source: North Central Texas Council of Governments

As a check on the reasonableness of this data, main lane traffic counts were conducted by TTI on four freeway sections in the Dallas region (Table 7). The sections were chosen because they consistently experience high volume traffic but do not operate over capacity during peak hours on a daily basis. Each of the facilities matched the person trip data from the NCTCOG model for the four highest hours of the day, except for S.H. 360 which had its third highest hour from 6-7 p.m. instead of 7-8 a.m. as was observed at the other locations. A check of the Texas Ranger baseball schedule confirms that the data at this location was collected on days that there were home games. The heavier northbound traffic going to the games pushed the 6-7 p.m. time period


Figure 4. Number of Person Trips by Hour of the Day

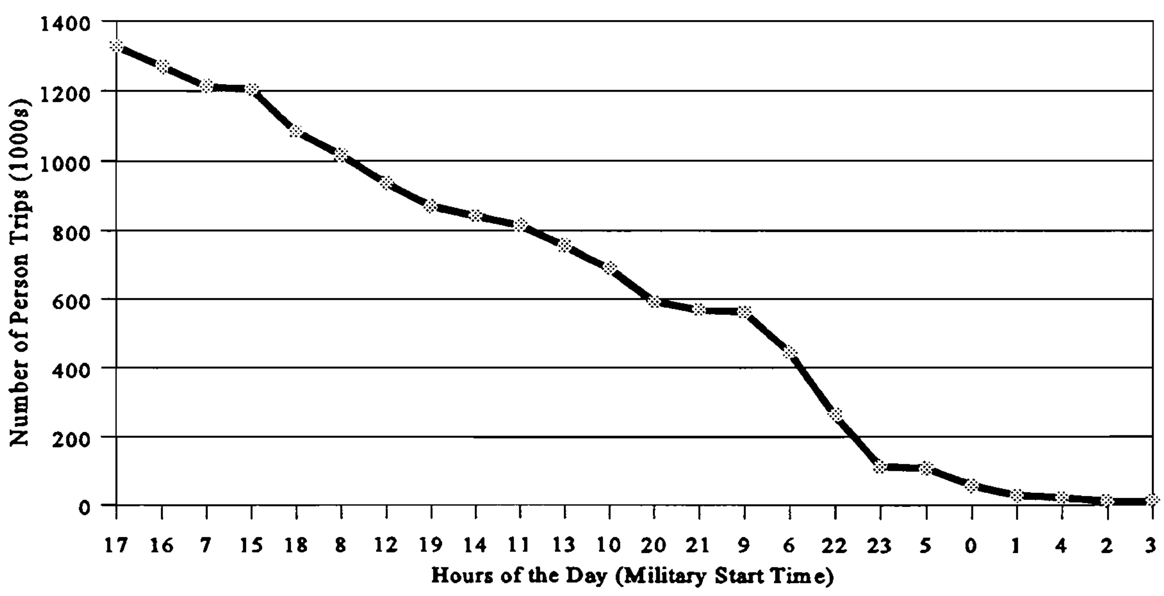



Figure 5. Number of Person Trips by Hour of the Day
Sorted in Descending Order

to the third highest hour at this location. The 7-8 a.m. time period was the fifth highest hour of the day on S.H. 360.

Table 7. Identification of Highest Volume Hours for Sample Freeway Locations

| Location | Peak Hour | 2nd Hour | 3rd Hour | 4th Hour |
|---|---|---|---|---|
| I.H. 30 @ Hampton | 5-6 p.m. | 4-5 p.m. | 7-8 a.m. | 3-4 p.m. |
| S.H. 183 @ MacArthur | 5-6 p.m. | 4-5 p.m. | 7-8 a.m. | 3-4 p.m. |
| S.H. 360 @ Arkansas | 5-6 p.m. | 4-5 p.m. | 6-7 p.m. | 3-4 p.m. |
| I.H. 635 @ Garland | 5-6 p.m. | 4-5 p.m. | 7-8 a.m. | 3-4 p.m. |
| Person Trip | 5-6 p.m. | 4-5 p.m. | 7-8 a.m. | 3-4 p.m. |

The production and attraction factors for the four highest hours of the day are shown in Table 8. These factors are used to adjust the daily production-attraction person-trip tables before they are converted to origin-destination vehicle trip tables and assigned to the peak hour roadway network.

Table 8. Production-Attraction Factors for the Four Highest Hours

| Military Time (Trip Start) | HBW | | HNW | | NHB | | Other | |
|---|---|---|---|---|---|---|---|---|
| | Prod. Factor | Attract. Factor | Prod. Factor | Attract. Factor | Prod. Factor | Attract. Factor | Prod. Factor | Attract. Factor |
| 17-18 | 0.47% | 13.24% | 3.69% | 4.39% | 3.60% | 3.60% | 3.69% | 4.39% |
| 16-17 | 0.88% | 12.15% | 2.74% | 4.84% | 3.60% | 3.60% | 2.74% | 4.84% |
| 7-8 | 17.72% | 0.32% | 6.00% | 0.67% | 0.84% | 0.84% | 6.00% | 0.67% |
| 15-16 | 0.91% | 5.37% | 2.58% | 6.05% | 4.41% | 4.41% | 2.58% | 6.05% |

Source: North Central Texas Council of Governments

This discussion represents a framework for extending the use of peak hour traffic assignments to generate traffic estimates for other hours of the day. It is anticipated that this approach will be utilized in the development and testing of alternatives for the Trinity Corridor MIS in Dallas and may be used by the NCTCOG in development of their Mobility 2020 plan. There are a number of potential concerns that need to be examined as these studies progress:

1. The assignment of vehicular trips associated with a particular hour of the day only ensures that you have the proper amount of traffic on a system wide basis, not for each facility; for instance, the sample data in Table 7 showed that the third highest hour of the day for S.H. 360 is sometimes 6-7 p.m. instead of 7-8 a.m. as it was for the other roadways.

2. One of the strengths of a peak hour assignment is that it produces a directional traffic assignment. However, the assignment of traffic from hours further from the peak will probably show diminishing directional splits. This emphasizes the need to do a.m. and p.m. peak hour assignments, in addition to the DHV assignment, since a highly directional facility may affect the cross section that is planned.

3. An hourly assignment will load all the traffic to the roadway network; it cannot adjust for "peak hour spreading" that sometimes occurs when a facility is congested.

# VI. DESIGNING FOR CONGESTION

There are a number of freeway design elements that operate inefficiently under congested conditions. Congested conditions are created when demand in a section of freeway exceeds the capacity of that section of freeway. Capacity is a function of the particular freeway design, environmental factors such as the weather or the time of day, as well as the vehicles or vehicle activity on or adjacent to the freeway. Demand changes with the time of day and with the level of access provided by the freeway. Ideally, the best way to prevent congestion is to have good forecasts of the changes in demand so that the necessary capacity can be designed into a freeway. However, a freeway design that attempts to satisfy all demand will probably be excessively expensive and have unacceptable environmental/community impacts. Accepting congested conditions for at least part of the day may have potential value in planning and designing transportation facilities for the future.

In the past, the primary design factor has been design speed. During peak flow periods, however, vehicles should not be expected to be able to travel at the design speed of the freeway under the acceptance-of-congestion alternative. This raises new questions if freeway designers must consider congestion as a factor in the design of freeways as well as the design speed of the freeway since free flow conditions will be expected to continue during most of the day. Congestion generally begins at freeway elements that require vehicles to interact with other vehicles such as at merging, diverging, or weaving points on the freeway. Congestion can also be caused by elements that appear unexpected to a driver. These types of congestion are termed recurrent congestion and should not be confused with nonrecurrent congestion, which is a result of accidents or incidents on or adjacent to a freeway.

There are several general principles that should be considered when designing for congestion conditions (*4, 24, 31, 32*):

- Desirable freeway geometries, consistent use of signing, ramps located on the right, standard interchange designs, and route continuity should be considered in design regardless of the level of congestion in order to satisfy driver expectancy and to promote safety.

- Alternate routes and access to alternate routes should be provided to allow drivers to avoid congestion. Continuous frontage roads should be considered if there are no other alternative routes.

- Use operational aids (e.g. ramp meters, incident detection and response, changeable message signs, lane control signals) to maintain reliable freeway main lane flows during the peak period and off peak.

- Freeway designs should provide for flexibility so that the facility can be adjusted over time to meet changes in demand. Some examples of design features that could help ensure flexibility include shoulders built to the same pavement standards as travel lanes, clear span bridges at cross streets, and two-lane direct connector ramps between freeways. Elevated or depressed facilities, in general, will reduce flexibility because the long bridges and retaining walls that are required limit the ability to add pavement width.

- Lane balance is a basic principle of design, and not only should the number of lanes balance, but the capacity should match peak flow demands. Achieving lane balance without regard to peak flow is an empty exercise. When the capacity of a facility closely matches peak flow demands, then we move closer to the ideal situation in which all parts of a facility are used efficiently (i.e., become congested or uncongested at about the same time).

The most important factors in designing for congestion occur where vehicles interact on the freeway. Several freeway design elements that pose operational or safety concerns under congested conditions have been examined, and preliminary suggestions for design for safe operation have been identified. The design element study locations are listed in Table 9. Each location was observed in congested and uncongested conditions, and video recordings of the operations were made where possible. The study number found on the table corresponds with the locations shown on the map in Figure 6 for the Dallas Fort Worth area and on the map in Figure 7 for locations in El Paso. General findings are discussed under each of the design elements in the following sections. The majority of the design standards discussed below refer to Chapter X of *A Policy on Geometric Design of Highways and Streets* by the American Association of State Highway and Transportation Officials (also known as the AASHTO "Green Book") (*4*).

## SINGLE-LANE ENTRANCE RAMPS

### Definition

Probably the most common freeway design features are single-lane entrance and exit ramps. There are two general forms for entrance ramps, the taper design type and the parallel design type. The taper design entrance ramp is brought onto the through lane of a freeway with a uniform taper of 50:1 to 70:1. The taper design has the acceleration area entirely on the ramp. The parallel design entrance provides an added lane as part of the freeway of sufficient length for a vehicle to accelerate prior to merging. A taper of about 90 m (300 ft) at the end of the acceleration lane is suitable for design speeds up to 113 kph (70 mph) to guide a vehicle onto the through lane of a freeway. Figure 8 shows the taper design and the parallel design entrance ramps. According to AASHTO, either the taper design or the parallel design will operate satisfactorily, although there is a trend toward use of the taper design for both entrance and exit ramps. Some agencies use the taper design for exit ramps and the parallel design for entrance ramps (*4*).

Table 9. List of Study Locations

| Site Number | Location |
| --- | --- |
| Single-Lane Entrance Ramps | |
| 1 | Eastbound I.H. 30 to Northbound I.H. 35E |
| 2 | Singleton Boulevard to Northbound Loop 12 |
| 3 | Southbound U.S. 54 and Northbound I.H. 110 to Eastbound I.H. 10 |
| 4 | Eastchase Boulevard to I.H. 30 |
| Two-Lane Entrance Ramps | |
| 5 | Westbound Spur 366 to Northbound I.H. 35E |
| 6 | Southbound Spur 408 to Westbound I.H. 20 |
| Exit Ramps | |
| 7 | Northbound I.H. 35E to Northbound Dallas North Tollway |
| 8 | Southbound U.S. 54 to Eastbound I.H. 10 |
| 9 | Westbound I.H. 20 to Green Oaks Boulevard |
| Auxiliary Lanes | |
| 10 | Between Dallas Parkway and Midway Road on Westbound I.H. 635 |
| Lane Reductions | |
| 11 | Eastbound I.H. 635 at I.H. 35E |
| 12 | Southbound I.H. 35W at Alta Mesa |
| 13 | Westbound I.H. 635 to Southbound US75 |
| 14 | Eastbound I.H. 10 to Raynolds Street |
| Weaving Areas | |
| 15 | Westbound Spur 366 to Southbound I.H. 35E to Westbound I.H. 30 |
| Collector–Distributor Roads | |
| 16 | Northbound S.H. 360 at Spur 303 |
| Branch Connections | |
| 17 | Southbound I.H. 35E and Eastbound S.H. 183 |
| 18 | Northbound I.H. 35E and Northbound Loop 12 |
| Major Forks | |
| 19 | Northbound I.H. 35E to Northbound I.H. 35E and Eastbound I.H. 30 |
| 20 | Southbound I.H. 35E to Southbound I.H. 35E and Southbound Loop 12 |

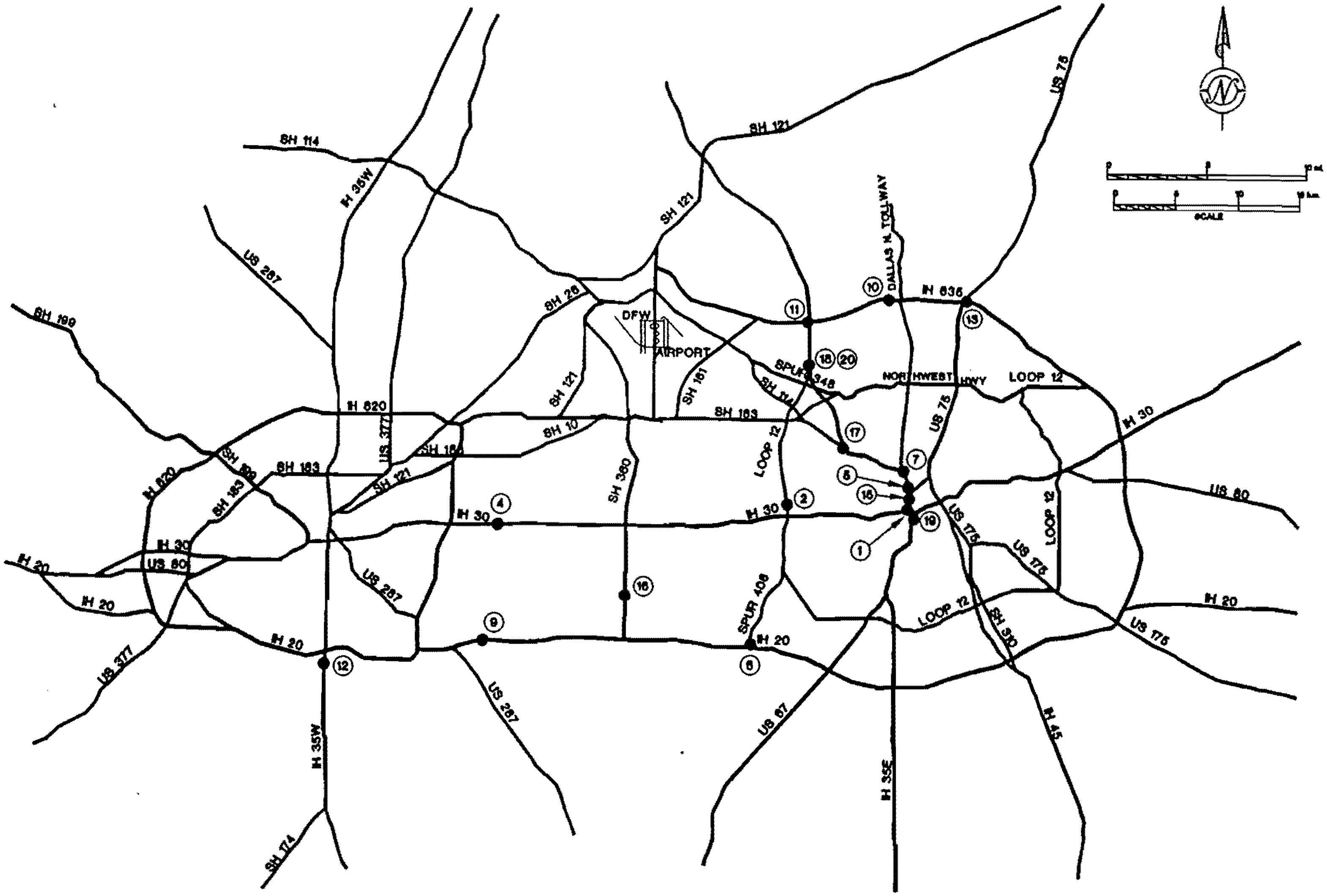



Figure 6. Study Locations in Dallas/Fort Worth

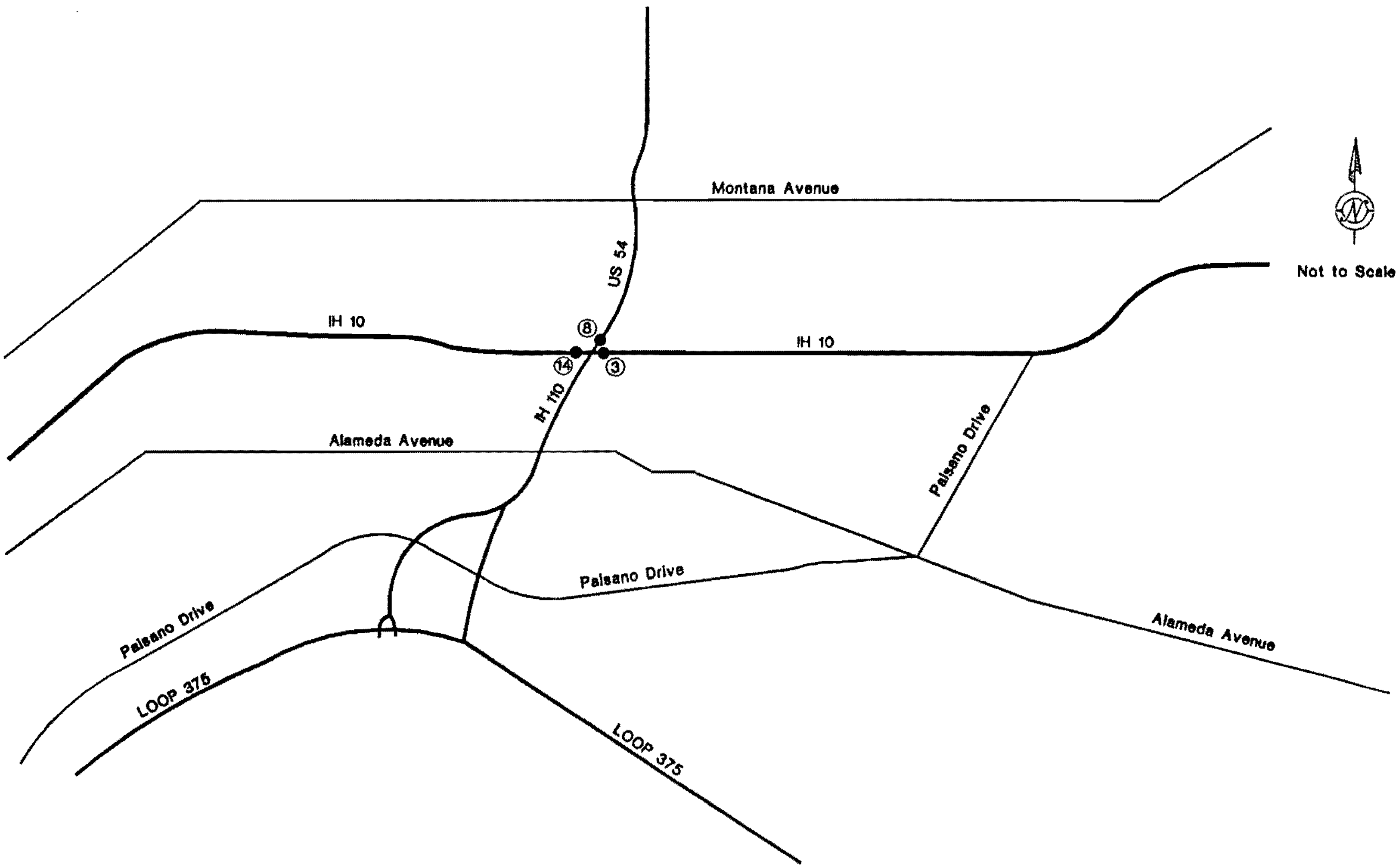



Figure 7. Study Locations in El Paso

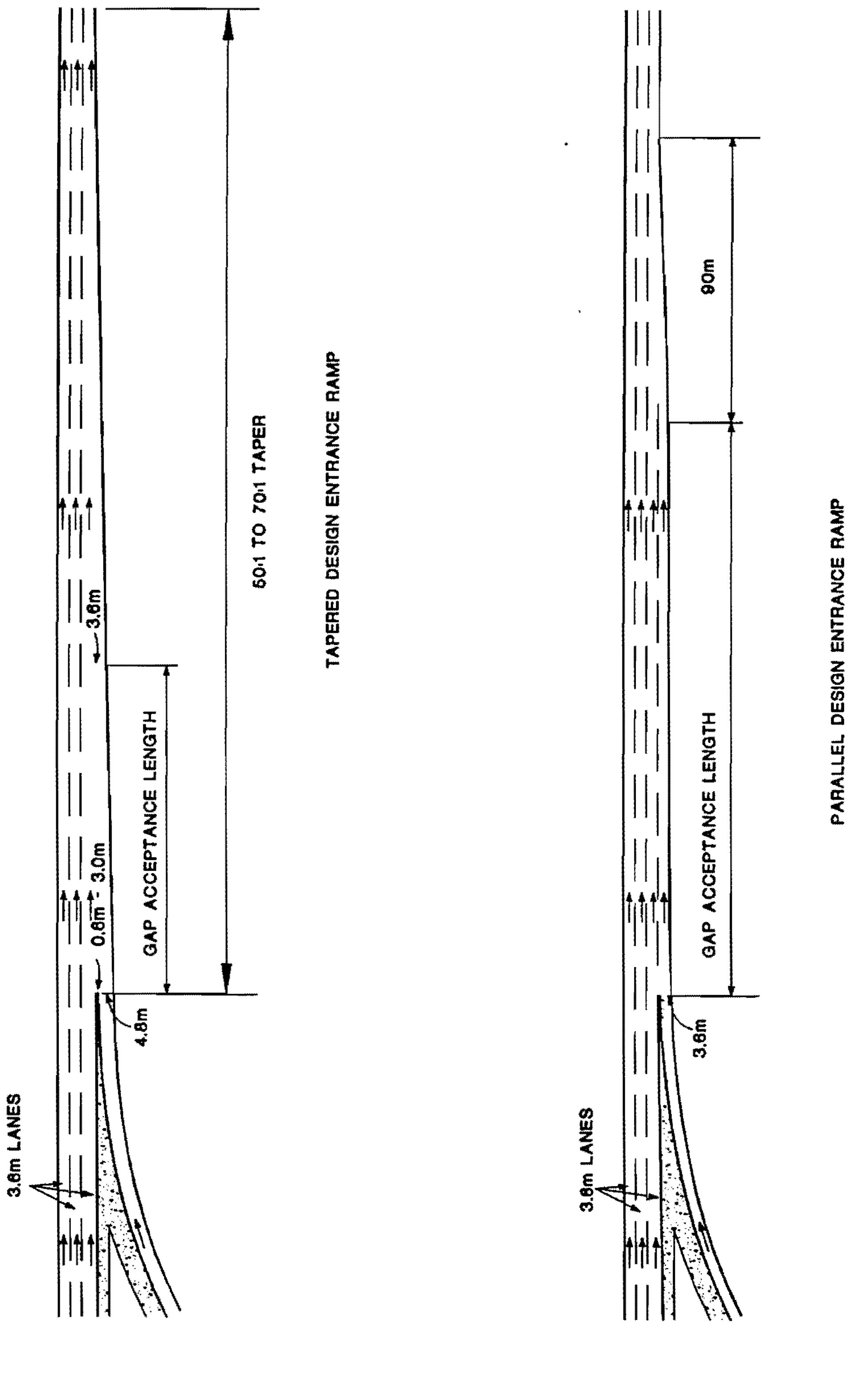



Figure 8. Typical Entrance Ramp Design Types
Source: AASHTO, A Policy on Geometric Design of Highways and Streets

## Study Locations

Four example entrance ramp locations where recurrent congestion is known to exist were identified for observation. The connection from eastbound I.H. 30 to northbound I.H. 35E (Stemmons) in Dallas is an example of a non-standard taper design entrance ramp. The Singleton Boulevard entrance ramp to northbound Loop 12 (Walton Walker) is an example of a standard taper design entrance ramp. The combined connection from southbound U.S. 54 and northbound I.H. 110 to eastbound I.H. 10 in El Paso is an entrance ramp that continues as an added lane. The Eastchase Boulevard entrance ramp to eastbound I.H. 30 in Fort Worth is an example of a standard parallel design entrance ramp.

## Observations and Discussion

Several problems related to congestion were identified through observation of operations on both ramp designs. On the taper design entrance ramp at congested speeds, vehicles appear to be unsure of the proper path to merge into the through lanes. This was observed at the Singleton on ramp and at the ramp connection from I.H. 30. The wide (4.9 m) available pavement encourages some drivers to pass vehicles following the proper path on the inside of the entrance ramp by crossing the entrance gore, or on the outside of the entrance ramp onto the shoulder. This appears as vehicles enter two or three abreast at some locations and may actually increase the capacity of a ramp, but at the risk of greater accident potential.

On a parallel design entrance ramp, in congested conditions, many vehicles do not know when to merge with the slower speed through lane traffic. This was observed at the Eastchase Boulevard on ramp. Some vehicles will attempt to merge into the first available gap or attempt to force a gap without using the acceleration lane, and other vehicles will travel to the end of the acceleration lane passing the slower through lane traffic and force a merge at the end of the ramp.

The phenomenon of vehicles entering two abreast was not observed at the parallel design example location as it was for the taper design. The 3.6 m lane width that is standard for a parallel type entrance ramp may discourage this type of behavior; however, many vehicles were observed at the Eastchase Boulevard on ramp passing merging traffic on the shoulder.

Under congestion, both the taper and parallel ramp designs have similar problems with more aggressive drivers overtaking less aggressive drivers and forcing gaps into the through traffic at locations upstream or downstream of the expected merge location. Although there is not strong empirical evidence to recommend one type over the other, the narrow lane width for a standard parallel type design appears to be slightly more desirable. The parallel design is also desirable because it is compatible with the introduction of an auxiliary lane. An auxiliary lane continuing to the next exit should be considered for conditions where the through lanes are near maximum capacity, though the entering and exiting volumes should be near equal. If an auxiliary lane cannot be provided for a heavy entrance volume, ramp metering should be considered. A design that provides a down grade for the entrance ramp, either to a depressed freeway section or from an elevated cross street, will allow a better view of the through lanes as well as easier acceleration during free flow conditions.

## TWO-LANE ENTRANCE RAMPS

### Definition

Two-lane entrance ramps are needed where demand exceeds the capacity of a single-lane entrance ramp. The basic design of two-lane entrance ramps is similar to the single-lane entrance ramps. There is a taper design type and a parallel design type. To help with lane balance, at least one of the entering lanes must continue as a through lane or an auxiliary lane. With the taper design, the inside lane of the two-lane ramp merges into the through lanes, also known as an inside merge, and the outer lane continues as an auxiliary lane. With the parallel design, the inner lane of the two-lane ramp continues as an auxiliary lane, and the outer lane is forced to merge into the inner ramp lane (*4*). Two-lane entrance ramps are almost exclusively found at freeway-to-freeway connections. Figure 9 shows both two-lane entrance ramp design types.

### Study Locations

Two example locations were selected for observation of congested conditions. The westbound Spur 366 (Woodall Rodgers) entrance to northbound I.H. 35E (Stemmons) was studied as part of a bottleneck project that is currently being implemented. The bottleneck improvement is planned to allow both lanes to continue with the outer lane ending as an exit-only lane to the next major exit at the Dallas North Tollway. The southbound Spur 408 connection to westbound I.H. 20 was also studied as part of a bottleneck project. This ramp was originally a single-lane entrance ramp, and it was converted into a two-lane ramp to relieve excessive demand for this movement in the evening peak period. Both these ramps are of the parallel design type. An example of a two-lane taper design entrance ramp is the southbound I.H. 35E connection to westbound I.H. 30, though recurrent congestion is not a problem at this location.

### Observations and Discussion

The primary problem observed with the two-lane entrance ramp from Spur 366 occurs at the merge with I.H. 35E. The demand on the ramp exceeds the capacity of the single added lane, which requires the excess demand to merge into the through lanes that are near congestion in the peak periods. The merging condition at this location results in speed differentials between the inside through lanes and the outer lanes. The two-lane entrance ramp from Spur 408 has similar demands; however, there is available capacity on the through lanes of I.H. 20 during the peak periods. As a result, no specific problems were observed at this location. Another problem discussed in the AASHTO "Green Book" is that of driver expectancy. Drivers behave differently on the two-lane entrance ramp designs. With the taper design, drivers tend to use the right ramp lane, and with the parallel design, drivers tend to use the left ramp lane. In either case, the drivers

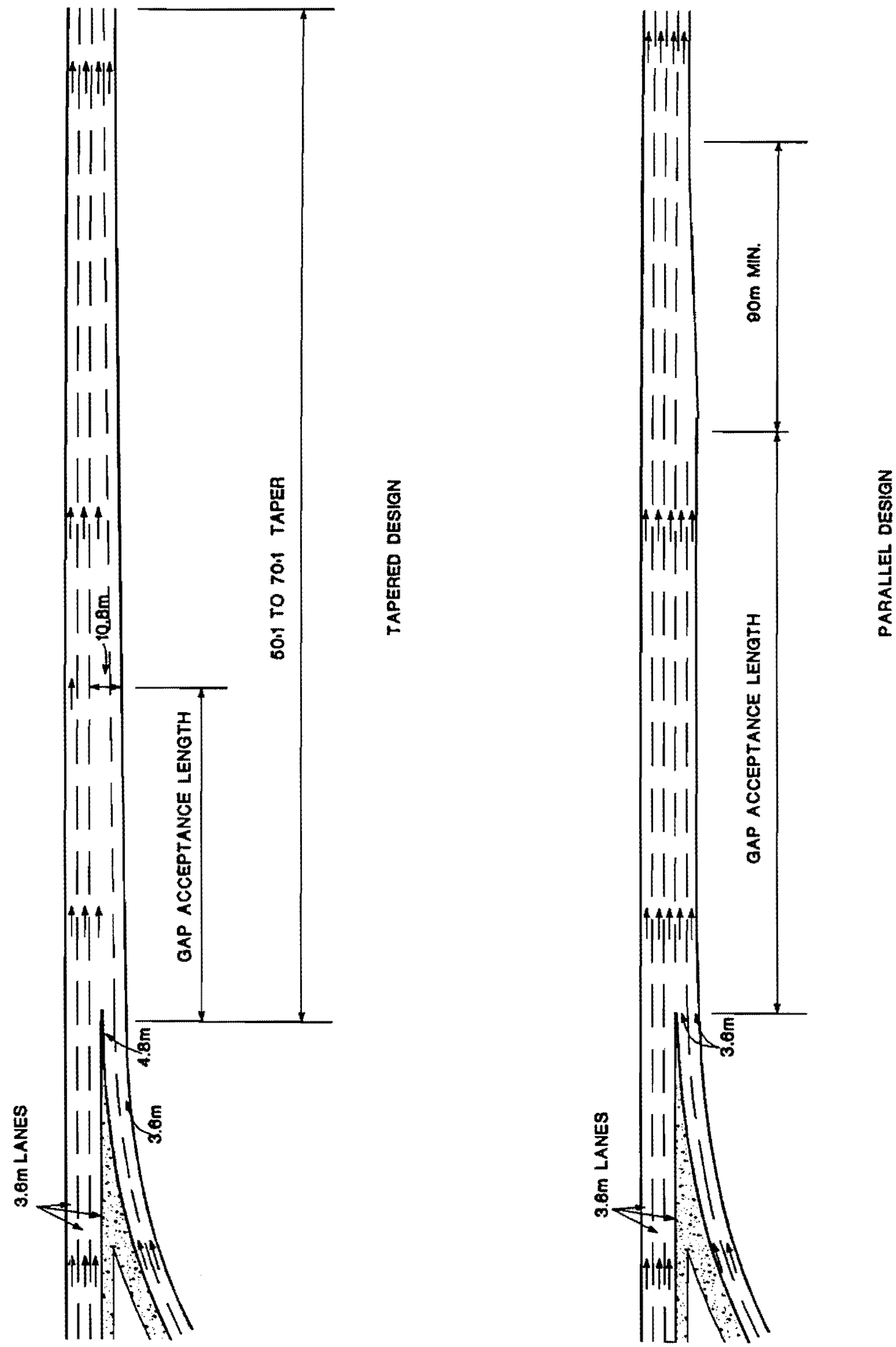



Figure 9. Typical Two-Lane Entrance Ramp Design Types
Source: AASHTO, A Policy on Geometric Design of Highways and Streets

tend not to use the merge lane until the continuing lane is near capacity. The problem occurs when a driver who is used to one design is confronted with the other. According to AASHTO, either two-lane ramp design has been shown to work well, but it is important that one design be selected exclusively throughout a region to satisfy driver expectancy (*4*).

The simplest solution to the problems noted above is to have both lanes continue for some distance; however, this can create a problem of lane balance. In order to maintain the same number of lanes for every lane addition, there has to be a lane reduction either upstream or downstream. Ideally, this lane reduction should happen at a high volume exit, which frequently occurs just upstream of a high volume entrance. A strong case can be made for carrying a reduced number of main lanes through major interchanges in urban areas.

## SINGLE AND TWO-LANE EXIT RAMPS

### Definition

There are two basic design types of single-lane exit ramps, which are similar to entrance ramps: a taper design type and a parallel design type. The taper design has the deceleration area entirely on the ramp with a clear diverge point from the freeway, while the parallel design uses a deceleration lane that is adjacent to the freeway. Two-lane exit ramps are sometimes necessary for capacity requirements and lane balance, and also have taper and parallel design types similar to single-lane exits. Figure 10 shows both one-lane and two-lane exits with both the taper and parallel exit designs.

### Study Locations

Three example locations of exit ramps were identified for observation of congested conditions. The northbound I.H. 35E (Stemmons) exit ramp to northbound Dallas North Tollway (DNT) is a single-lane taper design. The southbound U.S. 54 exit ramp to eastbound I.H. 10 in El Paso

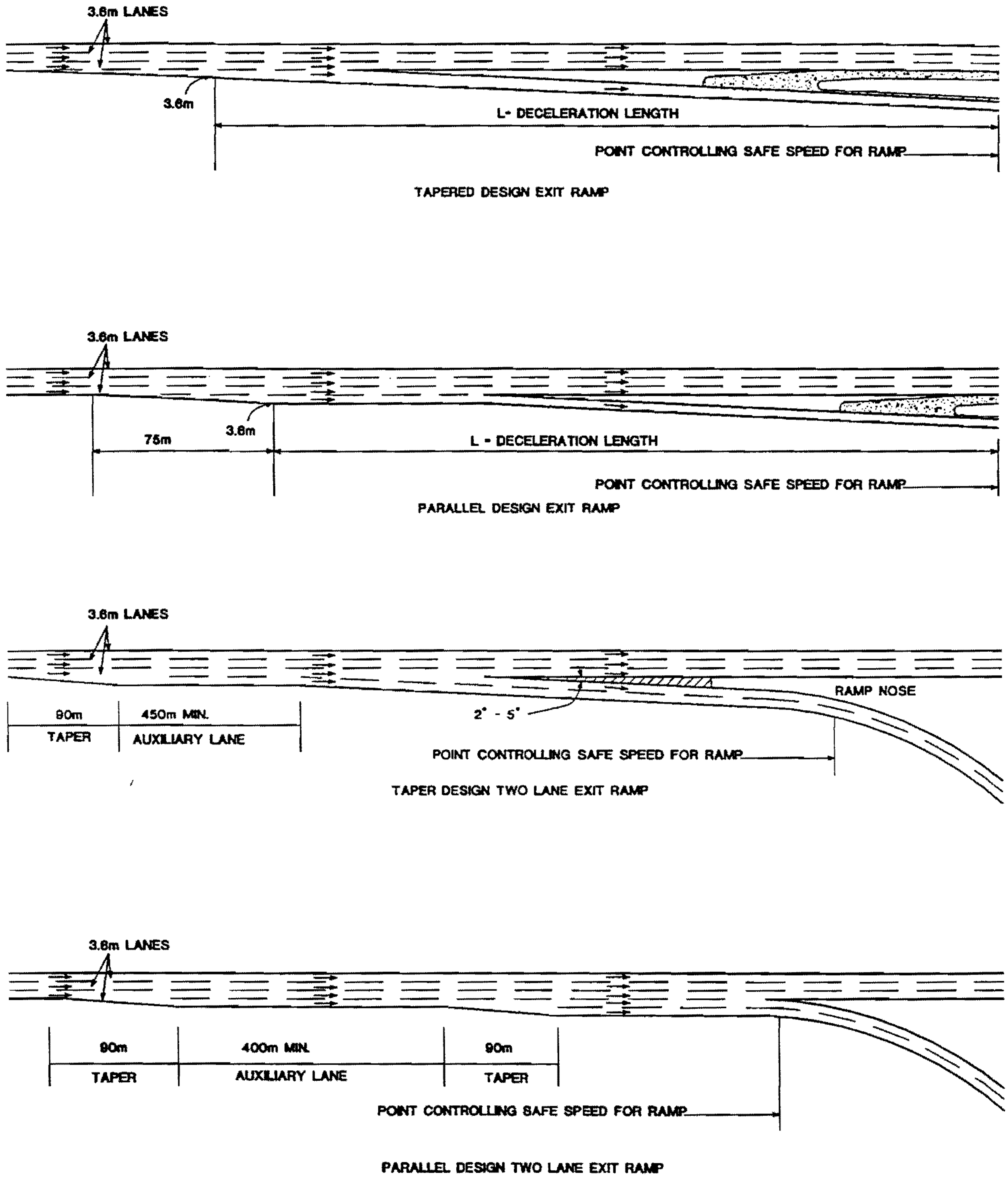



Figure 10. Typical Exit Ramp Design Types
Source: AASHTO, A Policy on Geometric Design of Highways and Streets

is a non-standard two-lane exit ramp that was studied as part of a bottleneck improvement project. The westbound I.H. 20 to Green Oaks Boulevard exit in Arlington was originally a single-lane exit-only, which was converted into a two-lane taper design exit with an option lane and an exit-only lane as part of a bottleneck improvement.

**Observations and Discussion**

Several problems were identified for exit ramps. Exiting vehicles may disrupt the through vehicles by slowing down or performing radical maneuvers to make an exit. This problem was observed at the exit to the DNT as through and exiting vehicles would slow to change lanes approaching the exit. Also observed at the DNT ramp, as well as at the U.S. 54 ramp connection in El Paso, was the problem of the ramp backing up or queuing onto the through lanes. The proximity of a cross-street intersection with the end of an exit ramp may also cause a backup or queue onto the through lanes of the freeway.

It is noted in *A Short Course on Freeway Design and Operations* that the taper exit design is preferred primarily due to the fact that at free-flow conditions many vehicles do not make use of the deceleration lane of the parallel exit design and exceed the design speed of the off ramp (*31*). Under congestion, this is not a factor; however, if the exiting demand is low, some through traffic may use the deceleration lane of a parallel design to pass slower traffic or queued traffic and then merge back into the through lanes. This is referred to as "queue jumping" in this report.

If the exit lane is the end of an auxiliary lane or through lane, a two-lane exit with an option lane and an exit-only lane should be considered even for exiting volumes that are less than a lane of capacity. The taper design for two-lane exits is also preferred. According to AASHTO, there is less lane changing associated with the taper design than with the parallel type (*4*).

A two-lane exit will be more effective for storage problems than a deceleration lane adjacent to the through lanes. Though both will provide additional storage, stopped vehicles adjacent to moving through vehicles creates an undesirable condition. A design that places the exit ramp on a rising grade, either from a depressed freeway or to an elevated cross street, may allow approaching traffic to better view conditions at the exit ramp. For any ramp design, successive ramps should, if possible, be uniform in design and signing.

## AUXILIARY LANES

### Definition

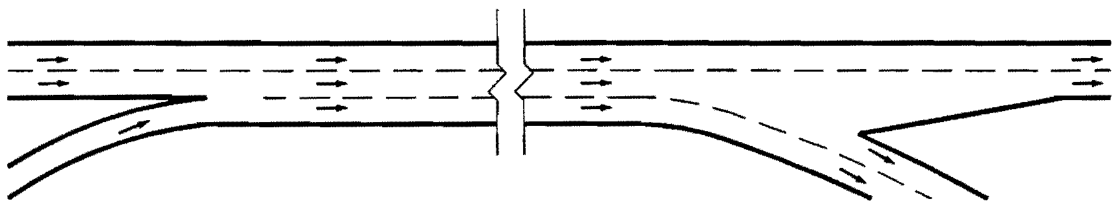
The portion of the freeway adjoining the traveled way for speed changes, weaving, and maneuvering of entering and exiting traffic is known as an auxiliary lane. Auxiliary lanes should be provided to maintain lane balance, for additional capacity in freeway sections with steep grades, for weaving areas, and for the maneuvering and speed changes of entering and exiting traffic. Auxiliary lanes are generally the same width as adjacent through lanes (*4*). Figure 11 shows some typical auxiliary lane design types.

### Study Locations

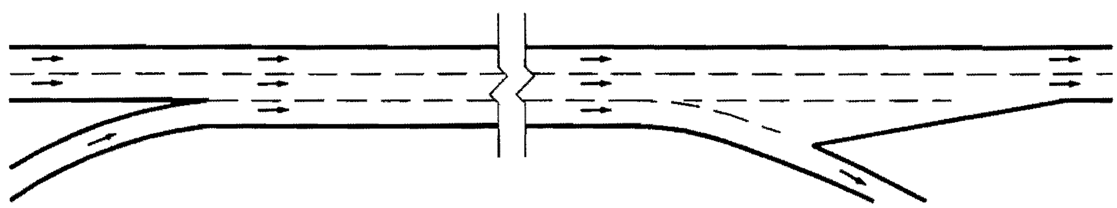
Any of the speed change lanes or lane additions associated with the entrance and exit ramps discussed above are examples of auxiliary lanes. A good example of an auxiliary lane that begins at a high volume entrance ramp and ends at a high volume exit ramp is on westbound I.H. 635 (LBJ) between the Dallas Parkway entrance and the Midway Road exit. The feasibility of incorporating this auxiliary lane into a separate collector distributor road was studied as an early implementation project for the LBJ Major Investment Study.

### Observations and Discussion

Auxiliary lanes where merging or weaving occur appear to be the primary location where congestion begins due to the turbulence caused by vehicles changing lanes attempting to enter,



AUXILIARY LANE DROPPED ON EXIT RAMP



AUXILIARY LANE BETWEEN CLOVERLEAF LOOPS OR CLOSELY SPACED INTERCHANGES DROPPED ON SINGLE EXIT LANE

Figure 11. Typical Auxiliary Lane Design Types
Source: AASHTO, A Policy on Geometric Design of Highways and Streets

exit, or bypass slower traffic. At the location observed on I.H. 635, a high through-vehicle demand and high entering and exiting demands result in routine congestion. From observation, it is uncertain which movement is the source of the congestion as each movement contributes to the congestion.

Under congested conditions, auxiliary lanes should only end at exit ramps with high exiting volumes. If not, the lane reduction will become a bottleneck. If the distance between an entrance ramp and an exit ramp is less than 450 m, an auxiliary lane should be provided, which will create a weaving area (*4*). Short auxiliary lanes that provide an inadequate length for weaving should be avoided. A long auxiliary lane should not be provided between a low-volume single-lane entrance and a low-volume single-lane exit, since it may be mistaken for a through lane by through traffic or used for queue jumping. However, the design should be flexible enough to provide an auxiliary lane in the future if traffic demand increases. The use of a white lane marking with short stripes can also be effective in distinguishing between the boundary between an auxiliary lane and an added through lane.

A preferred design for auxiliary lanes for congested conditions should be a single-lane entrance ramp to an auxiliary lane of adequate length for weaving maneuvers and a two-lane exit with an option lane and an exit-only lane. This type of design is being constructed on U.S. 75 (Central Expressway) between I.H. 635 (LBJ) and downtown Dallas. The value of this design is the higher weaving capacity created when the exiting vehicles are not required to change lanes to exit.

## LANE REDUCTIONS

### Definition

Lane reductions are necessary for ending auxiliary lanes or for changes in the basic number of lanes on a freeway. A lane may be reduced simply by merging it into the remaining through lanes as a lane drop, or by ending it with an exit-only lane or a two-lane exit with an option lane and an exit-only lane.

## Study Locations

Two-lane drops were observed for this study. The first is on eastbound I.H. 635 (LBJ) at I.H. 35E (Stemmons) in Dallas where the outer lane of three lanes is dropped just prior to the merge of the connection from southbound I.H. 35E. This lane drop was first studied as part of a bottleneck improvement project on I.H. 35E between I.H. 635 and Loop 12. The other lane drop is on southbound I.H. 35W just past the exit to Alta Mesa in Fort Worth. This lane drop is the end of the inside auxiliary lane from the connection from I.H. 20. The outer auxiliary lane ends as an exit-only lane to Alta Mesa.

Two exit-only lanes were observed. The exit-only lane from westbound I.H. 635 to southbound U.S. 75 (Central Expressway) in Dallas is the end of the inside through lane that exits to the left side. The exit-only lane from eastbound I.H. 10 to Raynolds St. in El Paso is also the end of a through lane on the right side of the freeway.

## Observations and Discussion

Any lane drop on a congested freeway is probably inappropriate and can be expected to become a bottleneck. The lane drop on I.H. 635 appears to be the main cause of congestion at this location throughout the morning and evening peak periods. Vehicles that remain in the lane and merge to the left at the end of the lane seem to experience the least delay. This tends to promote "queue jumping" and increased delay for vehicles in the through lanes. Similar problems were observed at the lane drop location on I.H. 35W. For both of the exit-only lanes observed, "queue jumping" appeared to be the primary problem as vehicles, whether intentionally or inadvertently, used the exit-only lane to bypass queued through traffic and merge into the through lanes near the exit.

Lane reductions on freeways near capacity should always end with an exit-only lane rather than a lane drop, and the exit-only lane should occur where a substantial exiting volume exists, as near to a full lane as possible. AASHTO recommends some general guidelines that should be followed for any lane reduction: (1) the lane reduction should be on the right, (2) it should be on the approach side of a vertical curve so as to be apparent to oncoming traffic, and (3) it should be on a tangent horizontal alignment. Where a lane ends at an exit-only lane, a recovery lane should be provided that extends the lane to the nose of the exit or a short distance past an exit. The recovery lane should not be so long as to be confused for a continuing lane, and it should have a taper of between 50:1 to 70:1, similar to the taper type design for an entrance ramp (*4*).

## WEAVING AREAS

### Definition

Weaving areas are freeway segments where the pattern of traffic entering and exiting a freeway at contiguous points of access result in vehicle paths crossing each other. Weaving areas may occur within an interchange, between entrance ramps followed by exit ramps, and on overlapping freeways (*4*). Usually, weaving maneuvers are one-sided and to the right of the through lanes, although double-sided weaves may occur where vehicles must weave across the through movement to or from a left side entrance or exit.

### Study Locations

Four example locations of weaving areas were identified for observation. Southbound I.H. 35E (Stemmons) from the westbound Spur 366 (Woodall Rodgers) entrance to the westbound I.H. 30 exit is a double-sided weave between a high volume entrance ramp and a high volume left handed exit, through a five-lane congested freeway. The westbound I.H. 635 (LBJ) Dallas Parkway entrance to the Midway Road exit is also a good example of a weaving area. The southbound Walnut Hill entrance to southbound I.H. 35E is a double-sided weave that crosses two through lanes connecting to southbound Loop 12 (Walton Walker). The weave from northbound Loop 12 (Walton Walker) to the Walnut Hill exit from northbound I.H. 35E is also a double-sided weave.

### Observations and Discussion

One of the primary problems associated with weaving areas is the length available for vehicles to complete weaving maneuvers. However, the weaving length only becomes a problem at design speed and was not observed to be a problem under congested conditions at any of the study locations. Another potential problem is the difference in speed of the weaving vehicles with the through or non-weaving vehicles. This is a particular problem with the double-sided weaves that were observed. From the Walnut Hill entrance, weaving vehicles must cross two lanes of traffic going to Loop 12 to get to I.H. 35E, which results in speed differentials and an increased potential for accidents.

Weaving areas, as discussed above, are a primary source for turbulence in the traffic stream, and any interchange design that removes weaving from the through lanes or eliminates the weaving areas is desirable. However, interchanges with weaving areas are usually less costly. Generally, a weaving maneuver can be made more easily in congested conditions rather than free flow; however, the weaving maneuvers may be creating the initial congestion. Cost is the primary reason for accepting congestion during peak flow periods, which means that some weaving areas and the congestion related to them should be accepted when designing for congestion.

## COLLECTOR–DISTRIBUTOR ROADS

### Definition

A separated roadway parallel to a freeway where several entrance and exit ramps are connected to the roadway to remove weaving and reduce the number of entrances and exits to and from the through traffic lanes. Collector–distributor (CD) roads may be provided within a single interchange, or through two or more adjacent interchanges. Depending on the required capacity, a CD road can be one or two lanes wide. Generally, the CD road and the ramps to and from the CD road are designed at a reduced speed, between 10 and 20 kph below the design speed of the freeway lanes (*4*).

**Study Location**

One example location of a CD road was observed for congested conditions. The CD road on northbound S.H. 360 at Spur 303 was studied to test the feasibility of ramp metering as a potential bottleneck improvement.

**Observations and Discussion**

The primary problem observed with CD roads when the through lanes are congested and if the interchanging demands are low or uncongested is that some through vehicles may use a CD road to bypass the congested through lanes. This was observed at the study location. Another problem observed at the S.H. 360 location was the lack of a continuing lane at the end of the CD road. If demands on a CD road are high, other problems related to the exits and entrances to and from the CD road will occur.

To avoid the problem of vehicles using a CD road as a queue bypass of the through lanes or to manage high entering demands, ramp metering should be considered at the end of a CD road on the entrance to the through lanes. For problems of individual entrances to and exits from the CD road, solutions discussed above for entrance ramps and exit ramps should be considered. According to AASHTO, the ramps to and from a CD road should have lane balance with the through lanes. A CD road should be considered a solution to the problems created by several closely spaced low-demand ramps to and from the through lanes of a freeway. Access to and from several cross streets or interchanges can be maintained with little disruption to the through lanes. The CD road creates a single exit and entrance where two or more exits and two or more entrances from the through lanes might have existed (*4*).

## BRANCH CONNECTIONS

### Definition

Branch connections are defined in the AASHTO manual *A Policy on Geometric Design of Highways and Streets* as "the beginning of a directional roadway of a freeway formed by the convergence of two directional multilane ramps from another freeway or by the convergence of two freeway routes to form a single freeway route" (*4*). Some general examples of branch connections are shown in Figure 12.

### Study Locations

Two example locations of branch connections were observed. The connection of southbound I.H. 35E (Stemmons) and eastbound S.H. 183 (Airport Freeway) into southbound I.H. 35E is a connection of two three-lane freeways into a five-lane freeway with an inside merge. The connection of northbound I.H. 35E and northbound Loop 12 (Walton Walker) was a former inside merge that was studied as part of a bottleneck improvement project, and now is a connection of a two-lane and a three-lane freeway into a five-lane freeway.

### Observations and Discussion

The first location observed at I.H. 35E and S.H. 183 was recently restriped to remove the inside merge. However, after a short time, the connection was returned to its original striping due to operational problems on the S.H. 183 leg of the connection. The inside merge had been removed by a lane drop on S.H. 183 upstream of the connection. At the other location of I.H. 35E and Loop 12, the inside merge was removed by adding a lane downstream of the connection. Few

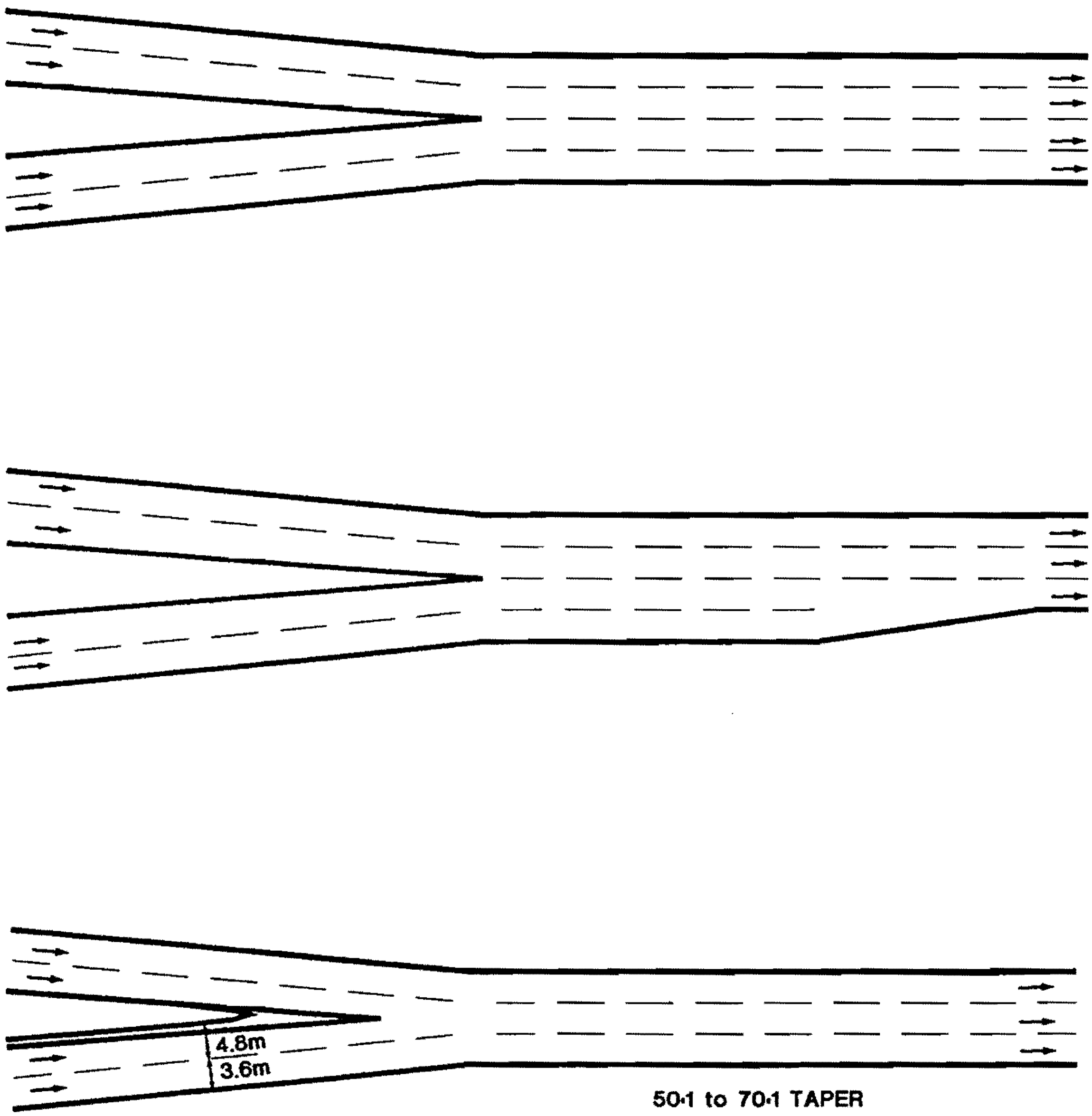



Figure 12. General Examples of Branch Connections
Source: AASHTO, A Policy on Geometric Design of Highways and Streets

operational problems were observed at either location in their current configurations. Both locations have adequate downstream capacity for the current demands. The merge at the connection of I.H. 35E and S.H. 183 appears to operate without problems due to the fact that the traffic demands on I.H. 35E are lower than the demands from S.H. 183 during the peak flow period. If the demands from each branch of a connection are nearly equal, there is a potential for safety problems at an inside merge, especially for the off-peak condition. Signing becomes extremely critical, and a long taper for the merge is most desirable. However, during congested conditions, there are fewer safety concerns; an existing inside merge should not be removed under these conditions unless capacity can be added downstream. In general, for congested conditions, it seems necessary at branch connections for each lane to continue. Any lane reductions should occur at high volume exits as exit-only lanes. A downstream exit ramp should be a suitable distance away to avoid creating a double-sided weaving problem.

## MAJOR FORKS

### Definition

Major forks are defined in *A Policy on Geometric Design of Highways and Streets* as "the splitting of a directional roadway, either by terminating a freeway route into two directional multilane ramps that connect to another freeway, or as the diverging area created by the separation of a freeway route into two separate freeway routes of about equal importance" (*4*). Figure 13 shows some general examples of major forks.

### Study Locations

Two example locations were observed. The northbound I.H. 35E (South R.L. Thornton) is a five-lane freeway section, which divides into a two-lane ramp connection to northbound I.H. 35E (Stemmons) and a three-lane connection to eastbound I.H. 30 (East R.L. Thornton). The other location at the southbound I.H. 35E (Stemmons) is a five-lane freeway section, which divides into

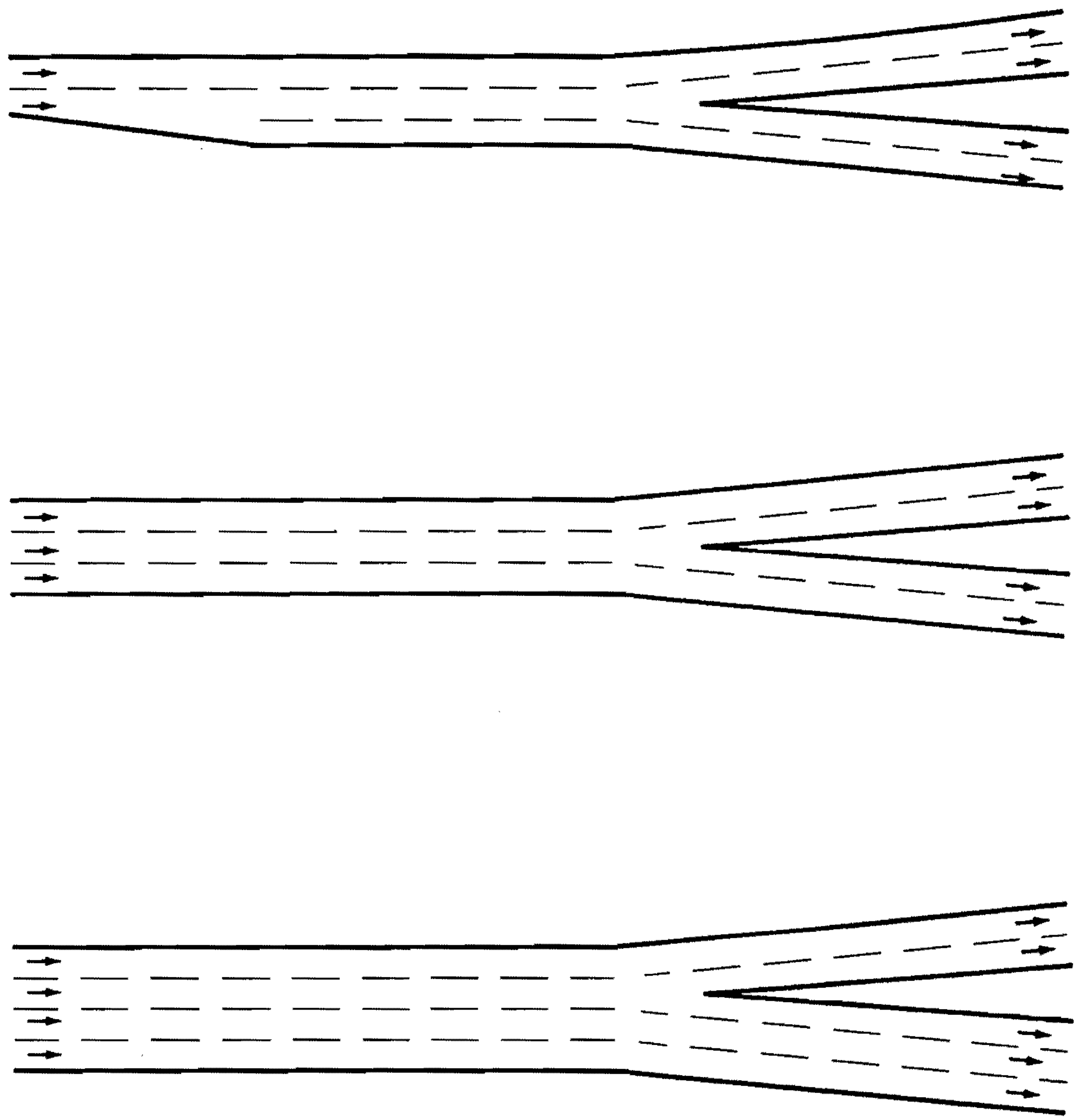


Figure 13. General Examples of Major Forks
Source: AASHTO, A Policy on Geometric Design of Highways and Streets

two, three-lane freeway sections, southbound I.H. 35E and southbound Loop 12 (Walton Walker).

**Observations and Discussion**

The only significant problem that was observed at a major fork occurs at the diverge. If there is no option lane, when one side of the fork is in congested conditions and the other in free-flow conditions, "queue jumping" may occur as well as safety problems at the nose of the fork. This was observed at the northbound I.H. 35E diverge to I.H. 35E and I.H. 30. The location at southbound I.H. 35E and Loop 12 has an interior option lane that results in an increase in capacity downstream of the fork. AASHTO recommends that one of the interior lanes approaching a major fork should be an option lane. Furthermore, any upstream entrance ramp should be a suitable distance away to avoid creating a double-sided weaving problem.

Table 10. Summary of Design Element Findings

| Design Element | Congestion Design |
|---|---|
| Single-Lane Entrance Ramps | Parallel type entrance ramp is slightly preferred over taper type design<br><br>Auxiliary lanes between heavy entrance ramps and exit ramps are desirable<br><br>Ramp metering should be considered if an auxiliary lane cannot be provided for a high volume ramp |
| Two-Lane Entrance Ramps | Single-lane entrance is more desirable than a two-lane entrance unless capacity is available downstream.<br><br>Two-lane entrance ramps should be given two additional lanes on the freeway rather than require a merge. |
| Single-Lane and Two-Lane Exit Ramps | Taper type exit ramp (single-lane and two-lane) is preferred over parallel type design<br><br>Two-lane exit is a better way to provide storage capacity off the freeway than a single-lane parallel type design<br><br>Two-lane exits with an option lane and an exit-only lane are preferred over a single-lane exit-only lane |
| Auxiliary Lanes | Auxiliary lanes should only end at high volume exit ramps<br><br>Auxiliary lanes between heavy entrance and exit ramps are desirable<br><br>A dotted white lane marking should be used to delineate the boundary between an auxiliary lane and through lane |
| Lane Reductions | Exit-only lanes are the preferred method to implement lane reductions; avoid lane drops that merge through lanes |
| Weaving Areas | Eliminate over capacity weaving areas if not cost prohibitive |
| Collector–Distributor Roads | Use to combine low-volume, closely spaced ramps<br><br>Should have lane balance with the through lanes<br><br>Consider use of ramp metering at the end of a CD road to discourage use of the CD road as a through lane bypass |
| Branch Connections | Inside merge should only be considered where adequate signing and extended taper length can be provided.<br><br>Any downstream exit ramp should be a suitable distance away to avoid creating a double-sided weaving problem |
| Major Forks | One of the interior lanes should be an option lane<br><br>Any upstream entrance ramp should be located to avoid creating a double-sided weaving problem |

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