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16. Abstract

The method used to prepare a traffic forecast needs to relate to the type of project for which the forecast is prepared. This report describes the various types of projects for which the Texas Department of Transportation uses traffic forecast data. For each project category, the type of traffic forecast data needed to perform the required analyses is identified and the appropriate forecasting procedure and level of forecast accuracy that can be expected is described. The project categories and forecasting requirements discussed include urban transportation planning; feasibility studies; advanced planning; environmental documentation; and geometric, signalized intersection, pavement, and bridge design.

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TRAFFIC FORECASTING REQUIREMENTS BY PROJECT TYPE

by

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Research Report 1235-8
Research Study Number 0-1235
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IMPLEMENTATION STATEMENT

This research report will assist the Texas Department of Transportation by establishing a relationship between the project type and kind of traffic forecast required. The report defines traffic forecasting requirements and identifies the type of forecast data needed and the appropriate forecast technique based on the end use of the forecast. The report also discusses the level of forecast accuracy needed for the project.

DISCLAIMER

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SUMMARY

Implementing a highway project requires a significant work effort that may involve several levels of planning; social, economic, and environmental documentation; geometric and structural design; operations analysis; and pavement design. Traffic forecast information is required in each of these stages of project development, although the type of forecast information and the level of detail and accuracy needed varies.

Various techniques are used for traffic forecasting and traffic forecast refinement. These techniques differ in complexity, cost, level of effort, sophistication, and accuracy. The method used to prepare a traffic forecast should relate to the type of project for which the forecast is being prepared as well as the project's scale and cost. In order to establish a relationship between the type of project and the appropriate forecasting procedure, an understanding of the nature of the decisions to be made with these forecasts is required.

Four broad areas of forecasting needs were identified for the Texas Department of Transportation (TxDOT): Transportation System Planning; Highway Project Planning and Design; Bridge Project Planning and Design; and Administrative Requirements and Policy Decisions. These areas were further divided to identify specific project types and stages of project development followed by TxDOT. This report discusses the type of forecast (regional or subarea) needed; the output data and level of detail required from the forecast; and how the forecast data are used for system planning, feasibility studies, advanced project planning (including environmental documentation), project design, and administrative and policy planning.

TRAFFIC FORECASTING REQUIREMENTS BY PROJECT TYPE

Various techniques are used for traffic forecasting and traffic forecast refinement which differ in complexity, cost, level of effort, sophistication, and accuracy. The method chosen to prepare a traffic forecast should relate to the type of project for which the forecast is being prepared as well as the project's scale and cost. To establish a relationship between the type of project and the appropriate forecasting procedure, an understanding of the nature of the decisions to be made with these forecasts is required.

The purpose of this research area is to identify the types of projects for which the Texas Department of Transportation (TxDOT) uses forecast data to identify the type of data needed and the appropriate technique based on the end use of the forecast, and to discuss the level of accuracy that is needed for the project. Accuracy in this research area refers to the accuracy required for the traffic forecast's end use.

Four broad areas of forecasting needs were identified for TxDOT:

- Transportation System Planning
- Highway Project Planning and Design
- Bridge Project Planning and Design
- Administrative Requirements and Policy Decisions

These categories were further divided to identify the level of detail needed by project type, the appropriate forecasting method, and the accuracy that is required from the forecasting procedures.

TRANSPORTATION SYSTEM PLANNING

Urbanized Area System Planning

The formalized urban transportation planning process was initiated in Texas in the early 1960s in response to 23 U.S.C., 134 which required the continuing, comprehensive, and cooperative transportation planning process as a basis for federal funding of transportation projects in urbanized areas with a population of 50,000 or more. This planning process, illustrated in Figure 1, involves TxDOT, Metropolitan Planning Organizations (MPOs), local governments, and area transit systems in the development of transportation services and

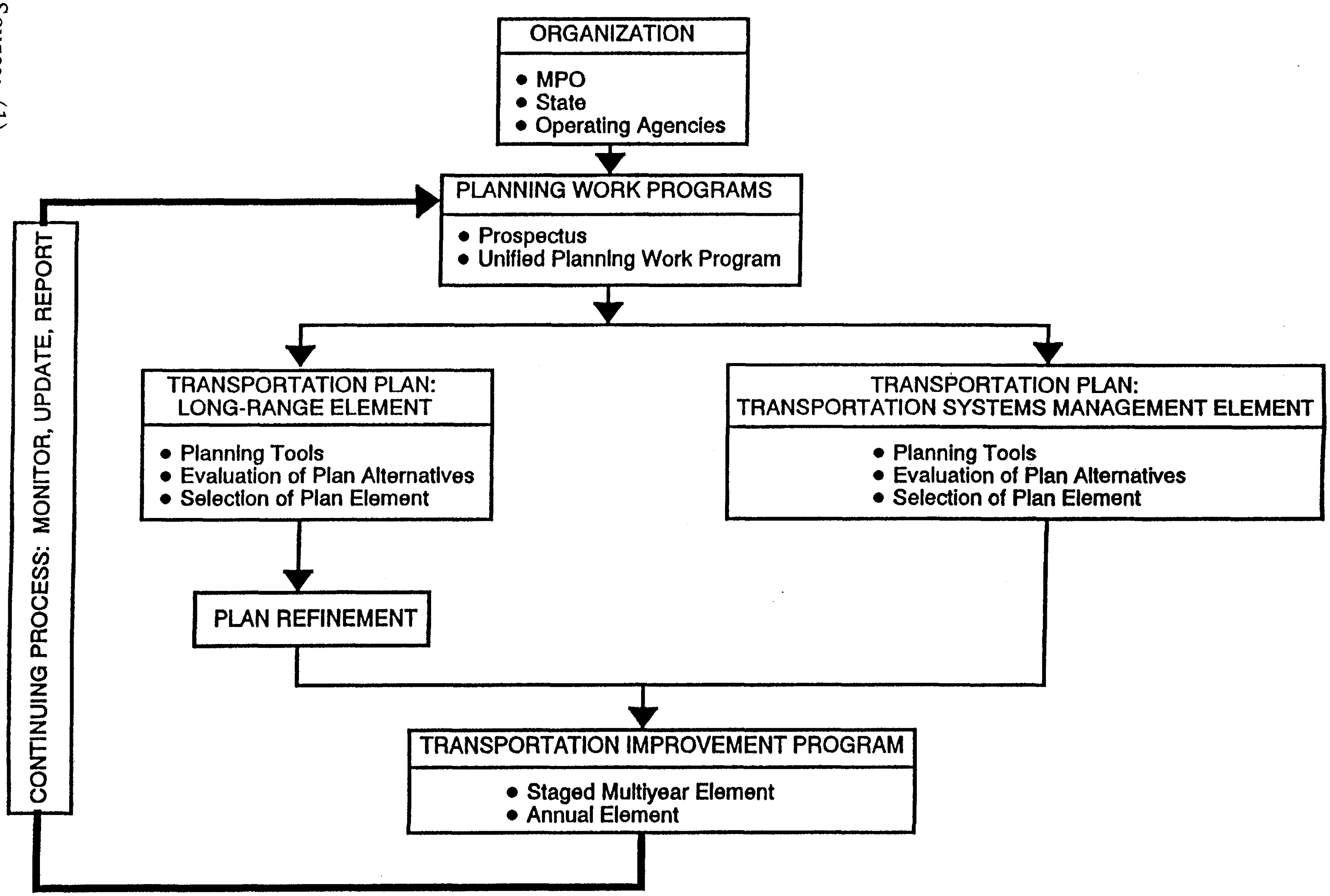

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Figure 1. The Urban Transportation Planning Process

facilities and leads to decisions on transportation policies and programs (1). The goal of the system planning process is to provide for the orderly and timely implementation of transportation improvements designed to meet the demand for travel within an urban area.

The federal transportation planning regulations (23 CFR Part 450.120) call for the following technical activities to be included in the planning process:

- Analysis of existing conditions, transportation facilities, and systems management;
- Evaluation of alternative transportation systems management (TSM) improvements (TSM element of the plan);
- Forecast of demographic, economic, and land use activities, and transportation demands based on these activities;
- Analysis of areawide new transportation investment alternatives (long-range element of the plan);
- Refinement of the transportation plan by corridor, transit technology, staging, subarea studies, and other appropriate methods;
- Assessment of urban development and transportation indicators and regular reappraisal of the plan; and
- Development of the Transportation Improvement Program (TIP).

Travel forecasts are used to fulfill several of these requirements such as projecting travel demands, analyzing areawide new transportation investments, and ranking projects for inclusion in the TIP in order of priority. Traffic forecasts are also used for specific planning studies performed to refine the transportation plan. The use and requirements for these types of forecasts are addressed under project planning.

In addition to these requirements, the Intermodal Surface Transportation Efficiency Act (ISTEA), passed in 1991, imposes additional factors that must be considered in the planning process. These include analyzing the interactive effects of transportation and development; developing transportation enhancements; developing congestion management and public transportation strategies and methods to evaluate their effectiveness; developing intermodal interchange management; and analyzing social, environmental, and economic effects of transportation plans. The organization, responsibilities, and technical approach for these added requirements have not yet been finalized. It is likely, however, that several of these planning requirements will also be based on travel forecasts performed for system planning.

The transportation system planning process is generally considered to be the first phase in the framework of TxDOT project development in urban areas (Figure 2). The planning process relies heavily on regional computer travel demand forecasting performed by TxDOT for each urbanized area within the state (with the exception of Houston and Dallas-Fort Worth). These urban area travel forecasts are made using the Texas Travel Demand Model, a traditional four-model mainframe package (including trip generation, trip distribution, mode choice, and traffic assignment) designed to predict the level of travel demand at the regional or state level. The output of these forecasts generally includes printouts of trip generation data such as the number of productions and attractions by zone and the demographic characteristics by zones; trip distribution data such as trip lengths by trip purpose and zone-to-zone movements; and a traffic assignment. These travel forecast data are used to develop the information needed to evaluate the effects that known and projected changes in population, employment, land use, and other socioeconomic conditions may have on the demand for travel on the urban area's major roadway and transit systems. The output of the travel forecasts prepared for system planning is used by TxDOT district offices, MPOs, and local governments to develop and/or update the transportation plan, to program projects for additional study in the planning work program, and to rank transportation improvements in order of priority in the TIP. Thus, the information developed in a system forecast ultimately guides the expenditure of state, local, and federal transportation funds.

The type of data needed to perform transportation system planning varies for each urbanized area within the state depending on the area size, growth rate (current and future), type of transportation facilities available, the range transportation problems and appropriate solutions, and the level of technical expertise available within the area. The most widely used information provided in system forecasts is the traffic assignment, the computer-generated representation of the major roadways with forecast average daily traffic (ADT) for each link.

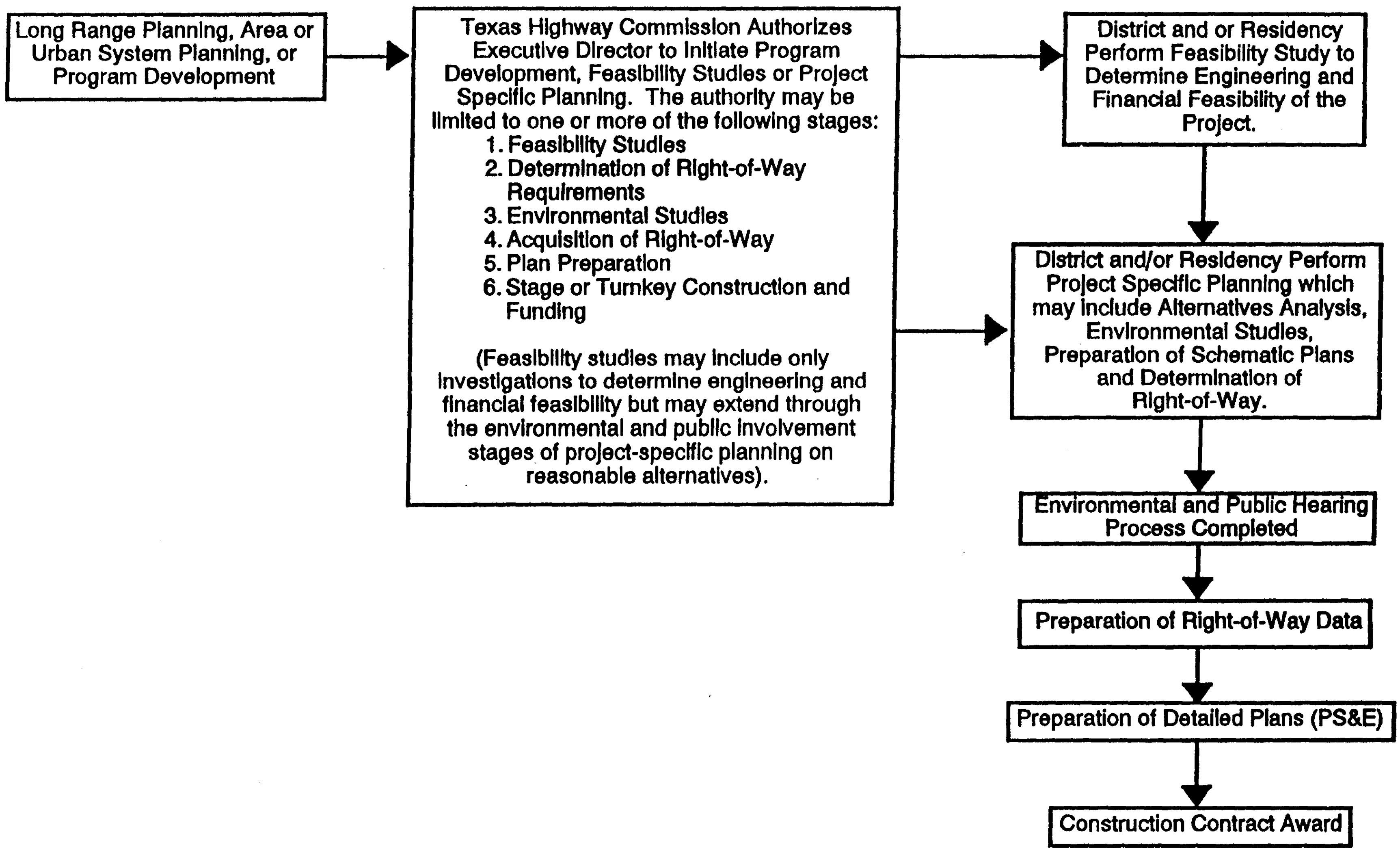

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Figure 2. Simplified TxDOT Project Planning and Development Flow Chart

The traffic assignment provides an indication of the type, location, and severity of possible transportation system deficiencies relative to a given land use arrangement and demographic forecast. Assignments are appropriate for evaluating alternative land use patterns and transportation systems, establishing priority programs for facility development, analyzing alternative locations for transportation facilities, providing information and feedback for project planning, and providing the basis for developing design volumes (2).

Other travel data that may be used in system planning include forecasts of the daily vehicle miles of travel (VMT), vehicle hours of travel, system travel time, and hours of delay. For urban areas where a large range of solutions may be appropriate, travel forecast data for various modes such as bus or rail transit, or carpools for high occupancy vehicle (HOV) lanes may be required. Areas experiencing severe congestion delay during the peak travel periods may also need peak-hour or peak-period forecast data to evaluate the effects of proposed improvements or policies.

Computerized travel demand modeling has been the basis for the transportation system planning process since the 1960s; extensive research has been devoted to improving the accuracy of the models in predicting travel and assigning that travel to the transportation system network. Although the level of travel forecasting detail required for system planning is less than that needed for project planning and design, these forecasts are ultimately used as the basis for developing traffic forecasts for project planning and design in urban areas. Thus, every effort should be made to produce accurate system forecasts.

System-level forecast accuracy depends on the accuracy of the various data used in the calibration of the models, the validation process, and the forecasts of urban activity (population, dwelling units, employment, land use, etc.) used as input to the models. Variances in the actual individual household trip-making characteristics from those developed from the travel survey; variances between the base-year traffic counts and actual average weekday link traffic; and miscalculation or unforeseen changes in the predicted population, employment, or land use of an area can all impact the accuracy of the travel demand forecast. Much of the congestion being experienced in major Texas cities is the result of the unforeseen, and thus not forecast, growth in population and employment. Underestimating the growth of these cities will result in greatly increased user costs in terms of delay; increased maintenance costs; increased construction and reconstruction costs due to right-of-way and traffic maintenance costs; increased social costs due to air quality degradation; and noise, energy, and overall quality of life impacts.

Those who use the planning data developed through the travel demand model need to be aware that the model predicts future travel based on the estimates of future land use, population, and economic conditions used as input. While there is little likelihood that these estimates will be made without some error (either misadjustment or unforeseen changes), this process is more likely to result in a better estimate of future travel than reliance on historic trends. This is because these estimates can reflect known or expected land use changes due to a change in zoning or policy, shifts in population and employment due to new or expected construction, or changes in travel patterns due to the addition of new facilities. It is not uncommon for an assignment of future traffic to show traffic estimates on some facilities to be close to existing traffic. This leads many to believe that the forecasting process is not credible because, based on past trends, the facilities in question should show more growth. However, this type of situation may be due to known or expected changes in land use or population of the area, the addition or improvement of a new parallel roadway, or some other change relative to the information used as input to the travel demand forecast.

Numerous efforts have been made to determine methods by which the assignment results produced by various travel demand models can be evaluated, but a "standard" level of accuracy has not been determined. It has been suggested, however, that accuracy in system planning should be at a level that would ensure that the design developed from the system forecast volumes would not be over- or underdesigned by more than one lane of traffic (3). Thus, the precision required in system-level planning is the difference in the capacity of 2 versus 4 versus 6 versus 8 lanes or about 15,000 vehicles per day for an arterial facility and about 40,000 vehicles per day for a freeway. This method translates into the average errors for volume ranges shown in Table 1 (2).

Table 1
Percentage Error by Volume Range

ADT Volume Range (000s)	Acceptable Percentage Error in Volume
5 - 10	35 - 45
10 - 20	27 - 35
20 - 30	24 - 27
30 - 40	22 -24
40 - 50	20 - 22
50 - 60	18 - 20
60 - 70	17 - 18
70 - 80	15 - 16
80 - 90	14 - 15

Source: (2)

Other Considerations

Originally, the accepted approach to transportation planning involved making long-range projections that resulted in a fixed statement of recommended transportation capital improvements for a target year (usually 20 years in the future) with provisions for re-examination of the plan on a fixed cycle. It has long been recognized that the transportation planning efforts of each urbanized area should be based on the needs of that area, not on a specified set of requirements or tools. Consideration should be given to the area's local goals, demographic and economic characteristics, transportation system facilities, the range of viable transportation solutions, existing planning process, and the available planning resources. Federal regulations call for the required planning activities to be conducted in accordance with the size of an area and the complexity of its transportation problems. There are no set procedures, forecasts, or models that must be adhered to by every urban area. This means that effective transportation system planning may vary from area to area, not just in the methods used, but also in the time frame for which planning is accomplished.

TxDOT performs computerized travel demand forecasting for most urbanized areas within the state. This process requires extensive effort to prepare the necessary base-year and planning-year data for input into the models and is, thus, costly and time-consuming. For many areas these forecasts have been prepared approximately every five years regardless of the need. In certain urban areas more direct and less sophisticated tools for developing transportation demand estimates may be more appropriate and cost-effective.

Travel demand forecasting should be used for areawide major transportation systems testing when conditions such as rapid growth and major changes in land use and/or policy require that a forecast be made to evaluate the impact of the changed conditions. The need for the traditional five-year reappraisal exists only in very large urbanized areas (over 750,000), areas experiencing or anticipating rapid growth (+ 2 percent or more annually), or those areas where air quality is a concern. Table 2 generalizes the relative level of effort that should be spent on long-range versus short-range planning given several broad criteria (4).

In most of the state's urban areas, the monitoring phase of the system planning process should be given more emphasis. The purpose of monitoring is to assess the performance of the transportation system relative to the trends forecast in the planning process. Monitoring consists of two activities: gathering sufficient data to assess the trends of development and travel and keeping the basic data needed for system planning current.

Table 2
Long-Range versus Short-Range Level of Effort

Criteria	Range of Criteria	
	Short-Range Emphasis	Long-Range Emphasis
1. Local Issues and Policies		
Growth policies	No growth	Growth
Air quality	Not a problem	Transportation Control Plan
Land use development patterns	Limited (in-fill)	New concentrations
CBD	Allow decline	Revitalize/promote growth
Attitude of state/local officials	Good, open to new approaches	Closed - Status quo
2. Area Characteristics		
Growth potential and past trends	Low (- to +1 percent)	High (+5 percent)
Physical constraints	Limits growth	Allows spread growth
Size	Near 75,000 population	Near 300,000 pop.
Type of area	Self-contained/limited attracts new industries	Bedroom community to rapidly growing metro area
Employment location/distrib.	Low density/scattered	Significant major generators
Land use development patterns	Limited (filling in)	New concentrations
CBD	Declining	Growing and dynamic
3. System Characteristics		
Complexity & nature of problems		
a. Extent and limits of congestion	Limited and localized	Severe and areawide
b. Traffic flow and capacity	Level of service B - C, few links w/high v/c ratios	Level of service E extensive, high no. of links w/high v/c ratios
c. Status of existing system	Mature	Developing and expanding
d. Extent of public transportation	Good regional service	None or limited service
e. Source of traffic problems	External (thru travel predominate w/existing Interstate bypass)	External (no existing bypass) *or* internal (area is attractor)
f. Parking supply	Adequate	Limited
Effect of other services	Transp. facilities developed as part of comprehensive development – orderly	Transportation service plays "catch up" only
4. Range of Feasible Solutions		
Amount of capital resources	Limited	Not as limited
Possible transportation solutions	Traffic engineering, transit operation	New highway systems, major facilities
5. Constraints on Planning		
Local regulating constraints	Supportive of planned development; effective	Hinder effective TSM actions – more coordination
Staff capability	Little or no staff	Full skills mix
Amount of planning resources available	Limited	Not as limited
Attitude of local and state officials	Good and open to new approaches	Closed - Status quo
Program level support by planning	Supportive	Significant justification required
6. Existing Planning Process		
Status of existing plans and planning	Established process	None or long-range plan evaluation needed
Existing data base	Adequate data base	None

Source: (4)

Although the exact monitoring activities will vary depending on the urban area's size and growth rate, most areas should follow the changes occurring in:

- Land use, population, dwelling units, and auto registration;
- Employment;
- VMT;
- Transit patronage; and
- Transportation system service and operation.

These changes should be compared to the trends forecast. If the trends are in line with those forecast, then monitoring should continue. If changes in any of the items reflect that travel and/or development are not following that of the forecasts, additional analyses must be performed to identify the location, specific problems, and impacts of the new trends. Based upon these analyses, the forecasts can be evaluated and appropriate changes to the long-range plan and transportation program made (5).

Improved monitoring has several benefits. First, it is less costly than preparing a system forecast in terms of funding and staff time. Also, deviations from the trends anticipated are likely to be isolated for most urban areas experiencing stable or moderate growth. Localized changes can be more efficiently handled through monitoring and updating rather than a new system forecast. Second, regularly monitoring and updating the basic data used in travel forecasting may reduce the time needed to update the files when a major update of the long-range forecast is needed. It will also make more current data available for use in forecasts for project-level planning.

PROJECT PLANNING

Implementation of a highway project requires a significant work effort that may involve several levels of planning; social, economic, and environmental documentation; geometric and structural design; operations analysis; and pavement design. Traffic forecast information is required in each stage of project development although the type of forecast information and the level of detail and accuracy needed varies. This section discusses the general forecast requirements for project planning.

Figure 2 in the system planning section provides a simplified flow chart of TxDOT's project planning and development process. Once the need for a project has been identified in the transportation system planning process (through program development or by concerned groups and/or individuals), project planning is performed to evaluate alternative improvements and associated impacts for a specific transportation corridor or facility. The goals of these planning studies are to establish the project feasibility; to identify the preferred alternative; and to define the improvements to the extent that the facility size, the probable social, economic, and environmental impacts, and the cost versus benefits can be assessed as accurately as possible. Accurate definition of the needed improvements is particularly important in urban areas where underestimation of the need for improvements can result in excessive congestion and costly delays, and the overestimation of the need for improvements can cause a project to be so costly that it may never be built.

It is important to note that each step shown in the chart may not be performed for every project and that there are no standard project planning study processes defined by TxDOT for the different types of studies identified. A feasibility study, for example, may not be prepared for a specific project because the work has been accomplished by another agency (local city or county government, MPO, etc.). For other projects, the extensive analysis of alternatives may not be required because there may be a limited range of solutions with which to improve travel.

Each stage of project-level planning is well suited to subarea planning and forecasting techniques which allow an accurate simplification of the areas outside of the corridor or project study area and permit a more detailed level of analysis within the subarea. With subarea analysis, smaller zones can be used; the network can be coded to accurately represent the highway improvements being studied. Subarea analysis minimizes the data set required to be manipulated and reduces the computer time. Thus, this type of analysis is more cost effective when a number of alternatives must be studied with some degree of detail. This is not to imply that system-level forecasts should not be used. For some projects, it is necessary to analyze the effect of proposed improvements on the entire transportation system. The decision to use system-level or subarea forecasting will depend on the project and the area location.

No accuracy standards have been developed for project-level planning forecasts. Each successive level of traffic forecasts prepared during project planning, however, should contain more detail than the previous level; and more extensive analysis and checks of the forecast volumes should be made.

Feasibility Studies

Feasibility studies are performed by TxDOT to determine the engineering and economic feasibility of a proposed project. These planning studies are more detailed than the studies performed during the areawide system planning analyses. Feasibility studies are focused on improvements for a particular transportation facility or corridor. Such studies, however, do not require the level of detail and accuracy needed in traffic forecasting for the advanced planning phase of project development. Feasibility studies are often limited to analyzing a certain type of improvement, such as constructing a new highway route instead of adding capacity to an existing facility or adding a high-occupancy vehicle facility or express lanes to an existing roadway rather than adding mainlanes.

Feasibility studies involve preliminary engineering to establish design feasibility, general right-of-way requirements, and associated project impacts. This information is then used to develop cost estimates to determine the financial feasibility of the project in terms of a cost-benefit analysis. This type of study is not intended to result in detailed design, environmental analysis, or cost estimates. Rather, feasibility studies determine if a project warrants further consideration and development.

Traffic forecasts are used in highway feasibility studies to analyze a preliminary facility alignment, cross-section, and access scheme to determine the effectiveness of the project to serve the projected demand and to estimate the financial feasibility of the project through a cost-benefit analysis of the project. Studies performed for projects in air quality nonattainment areas will need to broadly assess the probable impacts and/or benefits to air quality as part of the feasibility study as well.

Regional traffic forecasts are usually performed for feasibility studies. Subarea forecasts, however, are also appropriate for these studies, particularly when the system effects of the project have been analyzed previously during the transportation system planning process.

Forecast data required for feasibility studies usually include:

- Build and design year average daily directional traffic (ADT) assignment with estimates of the peak hour and truck percentages; or, build and design year nondirectional ADT assignments with estimates of the directional distribution and peak hour and truck percentages;

- Build and design year daily VMT for the system and the corridor;

- Build and design year daily vehicle hours of travel for the system and the corridor; and

- Build and design year daily vehicle hours of delay for the system and the corridor.

The forecast data needed for the cost-benefit analysis (listed as the daily vehicle hours of delay above) will vary depending on the procedure used. Generally, however, cost-benefit analyses for feasibility studies use an estimate of user benefits based on estimates of reduction in travel delay or savings in travel time for the design year. These user benefits may be estimated for a specific facility for added capacity projects or for the corridor or entire transportation system for new location projects.

The level of accuracy needed in a traffic forecast for a feasibility study is similar to that of system planning. Although it is desirable to have the forecast data necessary to accurately determine the facility cross-section, this is not a necessity. Except where right-of-way is limited and the expense of added right-of-way would change the financial feasibility of the project, traffic forecast data for this stage of project planning should be at a level of detail and accuracy such that the resulting cross-section is not over- or underdesigned by more than one lane per direction. Thus, the percentage of error relative to the different volume ranges would be the same as those in Table 1.

It is not necessary to perform extensive adjustments of the assigned traffic in a feasibility study. A check for reasonableness of the forecast data produced and the assigned traffic, however, should be made. Areas with unusual data or unreasonably high or low assigned data should be analyzed to identify any coding errors or miscalculations in input data. If errors are found, they should be corrected and a new forecast and assignment completed.

Several forecasts may need to be developed during the study if major changes to the facility size and/or alignment are required due to unforeseen conditions or impacts. The forecast data required and the level of detail and accuracy needed should remain the same for any forecast made.

Advanced Planning

The advanced planning phase of project development involves the environmental and public involvement process of project planning on reasonable alternatives, the preparation of the schematic design for the preferred alternative, and the determined right-of-way requirements. Forecast data are required in this phase to evaluate alternative improvements, to determine the environmental impacts of reasonable alternatives, and to prepare and analyze the preliminary schematic design of the project. Although each successive analysis within this stage of project development depends on and builds on previous analyses, the forecast requirements for each major process in this phase (planning, design, and environmental documentation) are discussed separately to facilitate understanding of the different forecasting procedures that may be required by various projects. The advanced planning stage of project development is understood by TxDOT personnel to include advanced planning, schematic design, and environmental documentation. However, it is clearer and easier to discuss these separately rather than as one stage.

The level of planning performed for a project during this stage of project development will depend on the project size, the area in which the project is located, the type of improvement(s) to be studied, and the number and range of alternatives to be evaluated. For some large scale urban projects where a wide range of solutions may be possible, an alternatives analysis will be needed in order to identify the reasonable alternatives to be analyzed in the environmental and public involvement process. For other projects the options available for improving travel along a facility or corridor may be limited to one type of improvement (e.g., adding mainlanes to an existing freeway). For these projects the evaluation of numerous alternatives is not required and the environmental and public involvement processes can be used to determine the preferred action, build or no-build.

The goal of alternatives analysis is to identify the improvement that best addresses the design year travel demand, is cost-effective, and is publicly and environmentally acceptable. Due to the differences that exist among specific TxDOT projects, the number, type, and level of detail of traffic forecasts required during advanced planning will vary. Ultimately, however, the daily design hour volumes (DDHV) used to develop and analyze the preliminary schematic plan will be produced from a forecast made during this process.

Generally, three series of forecasts may be needed during this stage of project development:

1. Forecasts for the initial evaluation of alternatives;
2. Forecasts for the evaluation of selected alternatives; and
3. A forecast for the preferred alternative.

Each successive stage requires an increasing level of detail and accuracy; although, as mentioned, not all levels of forecasting will be needed for every project.

Initial Evaluation of Alternatives

For projects where the evaluation of a number of alternatives is necessary, travel forecasts should be prepared for each distinct alternative to develop information needed in the evaluation. Preliminary engineering during the alternatives analysis generally involves sufficient information to prepare the horizontal alignment of various alternatives on aerial photography of the project area. Refined traffic assignments are not required at this stage. Unadjusted directional ADT assignments (checked for reasonableness) may be used during this initial evaluation and are actually desirable since using informed judgment to make adjustments to one assignment and not another may unintentionally bias the evaluation process. Preparing alternative forecasts using the same design year socioeconomic forecast and design year base network is most important. The only difference between alternative forecasts should be the specific network improvements being analyzed.

The forecast data needed will vary for each project depending on the range of improvements considered and the specific evaluation criteria set for the project. Regional traffic forecasts and assignments of directional ADT are usually made for the initial evaluation of a number of alternatives. These forecasts may be made for the design year (usually a 20-year horizon from a selected base year) or for the designated build year and design year (20-year horizon from the build year). The exact forecast year(s) will be determined for each project.

Other forecast data commonly used to highlight key differences between alternatives include:

- Estimates of the peak-hour percentage and the percentage of trucks for the build and/or design year;
- Average speed, travel time, and/or delay for the build and/or design year;
- Daily VMT for the build and/or design year; and
- Daily vehicle hours of travel for the build and/or design year.

The accuracy needed for this level of alternatives analysis in project planning is the same as that for feasibility studies. The forecasts should be at a level of accuracy such that for any of the alternatives tested, the number of lanes that will ultimately be needed will not be off by more than one.

<u>Evaluation of Selected Alternatives</u>

The goal of evaluating a few alternatives is to select and accurately define the cross-section of the preferred alternative and any reasonable variations for comparison and evaluation with a no-build option in the environmental and public involvement process. Preliminary engineering is at the same level as during the initial evaluation, although more detail with regard to interchanges and ramp access is known. At this point, sufficient knowledge of the travel demand for the corridor or facility should be available such that a detailed representation of the improvements being studied can be coded into the network. The location and configuration of ramps, diamond, split-diamond, directional interchanges, and major cross streets should be included in the network. Additionally, if HOV lanes are an alternative, the network should reflect this facility in as much detail as possible.

Subarea directional daily and peak-hour traffic assignments should be made for evaluating selected alternatives. Unadjusted assignments (checked for reasonableness) are acceptable for use in this evaluation as well. Other forecast data that may be used to evaluate alternatives includes:

- Build and/or design year daily and peak-hour average speed, travel time, and/or delay;
- Build and/or design year daily or peak-hour VMT; and
- Build and/or design year daily or peak-hour vehicle hours of travel.

Peak-hour forecasts of turning movements at major intersections may also be needed to evaluate selected alternatives. Turning movements for the build and/or design year may be needed for various projects including:

- Controlled access highway projects where the volume of interchanging traffic at cross-streets may be so large that a typical diamond interchange cannot provide an adequate level of service (LOS);
- Noncontrolled access highway projects in urban areas where major intersections will be the main factor affecting the LOS;
- Urban arterial street projects;
- Projects where certain intersections and/or interchanges are expected to have a high percentage of heavy trucks relative to the overall facility truck percentage; and
- Projects where several of the selected alternatives will be included in the analysis of environmental impacts.

Because most models do not produce good forecasts of turning movements, adjustments to the turning movement forecasts will have to be performed manually. It is not necessary to produce design level forecasts of turning movements at this stage in the planning process, but these estimates should be as accurate as possible given a minimal level of effort. The local district offices should provide the data to assist the Transportation Planning and Programming Division in estimating turning movements when needed. For projects in developed areas where the travel patterns and development are not expected to significantly change, the existing percentage of turning movements for each intersection approach may be used. For new location projects or projects in rapidly developing areas, local district staff should estimate the percentage of turning movements for each approach based on anticipated development and travel patterns.

For projects where several alternatives are to be included in the environmental and the public involvement processes of project development, the peak-hour forecasts developed during the evaluation of selected alternatives may be used to develop the preliminary schematic design for each alternative for use in determining and analyzing the impacts associated with each alternative. The level of engineering design performed should be the same for each alternative and should include only what is necessary to identify any impacts and reasonable measures of mitigation to determine if an environmental impact statement will be required. The forecast data needed for the environmental documentation are presented and discussed in the section immediately following planning.

The analysis of selected alternatives is not intended to produce design hour volumes. Forecast detail is important, however, because the data developed from these forecasts will be used to select the best improvement(s), to accurately define the facility cross-section, and for some projects may be used in the future for environmental analyses and for preparing design hour volumes.

The accurate definition of the facility cross-section requires that the amount of traffic using the facility during the peak hour be estimated. For some projects this may be prepared by making a 24-hour forecast (ADT) for the facility for the build and design years and applying certain planning factors (the directional distribution of traffic during the design hour (D), the percentage of traffic occurring in the design hour (K-factor), and the design hour percentage of trucks (T)) to determine the design hour volume. The following formula is then applied to determine the number of lanes:

$$\text{Number of Lanes} = \text{AWDT}*[K*D*(1+T)]/\text{Service Flow Volume}$$

Previous research by Walters and Poe (6) has shown that selecting different planning parameters (K, D, T, and service flow volumes) can result in over- or underestimating the facility size by more than 50 percent. The usual range for each of these planning parameters in large urban areas, the variation from the mean, and the possible effect on estimates of facility size is shown in Table 3. Because the values for K, D, and T usually decrease and the service volumes usually increase as an area becomes more urbanized, the errors compound rather than offset one another when the urbanization trend is underestimated. Thus, the size of the design error could be more than 50 percent.

Table 3
Planning Parameter Effect on Design

Planning Factors	Range of Values	Variation from Mean (%)	Effect on Facility Size (%)
K	.08 - .12	±20	±20
D	.50 - .70	±17	±17
T	.02 - .10	±67	±4
Service Volume	1700 - 2000 pcphpl	±8	±8

Source: (6)

Walters and Poe also have shown that the current method used to determine the K and D percentages for use in developing design hour volumes for planning does not take into account the need for:

- Site-specific values: Peaking patterns may differ on various mainlane segments of the same freeway, between mainlane segments and their adjacent ramps, and even between ramp pairs. These patterns are relative to the time of the peak (AM or PM), the percentage of 24-hour volumes, and the directional distribution.

- Evaluation of system effects: When capacity is added on one facility, the K-factor may increase. However, this increase in peak-hour volumes may be constrained by upstream or downstream congestion on facilities that cannot accommodate the increased peak flow. Thus, a lower K will occur on the improved facility.

- K-factors representing average weekday traffic (AWDT): K values are currently developed using ADT which is usually less than AWDT; yet, the design year ADT developed through modeling simulates typical weekday travel. Thus, use of K-factors based on ADT can result in the overestimation of facility size.

- Evaluation of the effects of time: Both K and D tend to decline over time as an area becomes more urbanized and where different land uses exist. Use of current K and D to develop future design volumes must account for the expected trend toward urbanization as well as the development of multiple, different land uses.

All of these factors can impact the estimation of design hour volumes from 24-hour forecasts and, thus, affect the accurate determination of the facility size.

Despite the margin of error associated with using average K, D, and T factors to develop peak-hour volumes for an entire project length, this method is appropriate for use in many urban areas where rapid growth is not anticipated and the existing levels of congestion are not severe. The use of facility-specific K, D, and T factors based on current traffic data and forecast changes in land uses and densities, however, is recommended for project planning for existing highways. New location planning studies should use estimates of K, D, and T factors based on factors for similar facilities in the area and forecast changes in land uses and densities.

Peak-hour assignment models have been developed for forecasting traffic for Houston and Dallas-Fort Worth. Research which focused on developing a peak-period assignment process for TxDOT has shown that a good relationship exists between peak-period volumes and the highest hourly volume within the peak period and that this relationship is much better than that between the peak-hour volumes and the 24-hour volumes. This is largely because peak-period volumes include a directional split that is close to the peak-hour directional split. Using a peak-period assignment process would eliminate the need to estimate the directional distribution of 24-hour assignments and, thus, may reduce the error associated with estimating the directional distribution for design-hour volumes. Use of a peak-hour computer assignment process can save substantial time associated with the manual factoring of assignments for several alternatives during the project planning process (7). Thus, use of a peak-hour model would be advantageous in large and/or rapidly growing urban areas.

Regardless of the method used to prepare peak-hour data, it is important to prepare each alternative forecast using the same design year socioeconomic forecast and network. Only the network changes needed to reflect the specific alternative to be tested should be made.

Preferred Alternative

A travel forecast of the preferred alternative may need to be made upon completion of the alternatives analysis if the selected improvement is a variation of a tested alternative improvement. For example, during the evaluation of selected alternatives it might have been determined that an additional interchange and/or additional ramp access should be provided. Since these were not included in the previous forecast, a new forecast reflecting the desired changes should be prepared. For the purpose of checking these changes, unadjusted daily and peak-hour assignments should be made and the revised alternative analyzed using the evaluation criteria set for the study. This check will verify that the changes made to the alternative have not caused the proposed improvement to become a less attractive solution.

Thus, the forecast data that should be produced for the preferred alternative include:

- Build year and/or design year directional daily and peak-hour volumes for each link;
- Build year and/or design year daily and peak-hour percentage of trucks;
- Build year and/or design year and peak-hour forecast of turning movements at all major intersections along the project;
- Design year daily and peak-hour VMT;
- Design year daily and peak-hour vehicle hours of travel; and
- Design year daily and peak-hour vehicle hours of delay and/or travel time.

The forecast prepared for the selected alternative should be used as the basis for preparing the design-hour volumes for use in design and in preparing the environmental analysis when only one alternative is being evaluated against the no-build. The travel forecast and assignment produced for the preferred alternative should be accurate and detailed enough to allow a reasonably accurate preliminary schematic design to be prepared once turning movements have been adjusted to reflect reasonable estimates. This means that the number of mainlanes required and the location and type of access for the facility should be identified. It may be argued that no design should be prepared from any unadjusted forecast. Too often, however, extensive time and effort is expended preparing DHVs, only to have the design of the facility change, thus, requiring the DHVs to be revised. The use of peak-hour models calibrated for each urban area in conjunction with detailed directional networks should result in relatively accurate volumes (except for turning movements) in sufficient detail to prepare a preliminary schematic. Once the schematic has been prepared and checked for LOS, the final design hour volumes can be prepared from the directional peak-hour assignment. This process should save time and money in preparing DHVs for design and environmental documentation. If, for a certain project, it is not desirable to develop a preliminary design prior to developing the DHVs, then sufficient preliminary engineering should be performed prior to making the final forecast for the preferred alternative to ensure that the facility network being modeled will meet basic geometric design standards.

In summary, the traffic forecast type (regional or subarea, daily or peak hour) and the exact data needed for project level planning studies will vary from project to project and from area to area. Despite the differences, however, Table 4 provides a general overview of the usual type of forecast and data used for the different levels of project planning.

Environmental Documentation

Assessment of the social, economic, and environmental impacts of a transportation project is performed in varying stages of detail during the project planning and design process. A broad assessment of potential social, economic, and environmental impacts may be made during feasibility studies; evaluation criteria set for the analysis of alternatives may include various environmental impacts. Detailed environmental documentation, however, usually is not performed until the later phase of the advanced planning/schematic design stage after directional design hour volumes and a preliminary schematic design have been prepared for the project under consideration.

Traffic data are needed for environmental documentation to describe the project need and to assess the noise and air quality impacts. The type of forecast data needed for these analyses and the sensitivity of the analysis procedures to traffic forecasts are discussed below.

Table 4
General Forecast Requirements for Project Planning[1]

Study Type	Forecast Type			Forecast Data[2]				
	Regional or Subarea	Daily or Peak Hour	Directional Assignment	Percentage Trucks	VMT	Vehicle Hours of Travel	Average Speed	Vehicle Hours of Delay
Feasibility Study	Regional or Subarea	Daily	Preferred but not Required	Yes	Yes	Yes	Yes[3]	Yes[3]
Initial Evaluation of Alternatives	Regional or Subarea	Daily	Preferred but not Required	Yes	Yes	Yes	Yes	Yes
Evaluation of Selected Alternative	Subarea	Peak Hour	Yes	Yes	Yes	Yes	Yes	Yes
Evaluation of Preferred Alternative	Subarea	Peak Hour	Yes	Yes	Yes	Yes	Yes	Yes

Notes:
[1]Forecast needs are dependent on the specific project and may vary from those listed.
[2]Other forecast data may be required depending on the project and the specific evaluation criteria.
[3]Average speed, vehicle hours of delay, or some other measure of benefits may be used in the cost-benefit analysis.

Description of Project Need

A forecast of the design year average weekday traffic is required as part of the description of the project need. The need for a new freeway or additional capacity on an existing freeway is usually based on forecast travel within the corridor or along the facility. Traffic data are used to show that the existing capacity of the facility is not sufficient to handle the forecast traffic or that a new facility is needed to relieve congestion within a corridor. When the need for the project is something other than capacity, other forecast data may be required.

The description of the project need is not sensitive to changes in the forecast of traffic unless changes in the forecast volumes are significant enough to result in lessening the need for added capacity.

Noise Impact Analyses

An analysis of the potential noise impacts resulting from implementation of a project must be made as part of the environmental processing of every project. Most projects will require a noise analysis, regardless of the project type or size. Some projects which are exempt from the noise impact analysis include those for illumination, signing, signalization, landscaping, safety, resurfacing, widening less than a single lane width, and adding shoulders (8).

The general procedures required in this analysis include a measurement and description of the existing noise levels for sensitive receptors along the facility, calculation of the design year sound levels for sensitive receptors, a comparison of the Federal Highway Administration (FHWA) Noise Abatement Criteria (NAC) with the existing and predicted sound levels, and an evaluation of alternative noise abatement procedures for those impacted receptors. Comparing the existing and predicted sound levels with the NAC determines whether a project will have an impact on the identified receivers and whether that impact is considered to be significant. The FHWA NAC are given in Table 5. These criteria represent the maximum acceptable noise level in *Leq* per hour and *L10* per hour for the various land use activity categories. The *Leq* sound level is the equivalent steady state sound level which in a stated period of time (usually one hour) would contain the same acoustic energy as the time-varying sound level during the same time period. The *L10* sound level represents the 90th percentile of sound generation. This level represents the magnitude of sound which indicates the portion of the spectrum most annoying to observers. Either of these sound levels, but not both, can be used in traffic noise impact analyses (8).

The decibel (dB) is the unit of measurement for noise. The decibel scale is a compressed view of the actual sound pressure variations. This means, for example, that a 26 percent change in the energy level changes the sound level only 1 dB. A 1 dB change in sound would not be perceptible to the human ear except in an acoustical laboratory. A doubling of the energy level would result in a 3 dB increase which would be barely perceptible in the natural environment. A tripling in the energy level would result in a noticeable change of 5 dB in the sound level, and a change of 10 times in the energy level would result in a 10 dB increase in sound.

Because the human ear has a non-linear sensitivity to noise, weighing scales are used to define the relative loudness of different frequencies. The "A" weighing scale is widely used in environmental work because it closely resembles the non-linearity of human hearing. Thus, the unit of measurement used in transportation projects is dBA.

TxDOT uses the FHWA traffic noise prediction computer program, STAMINA 2.0/OPTIMA to model expected noise levels along transportation facilities. The noise levels predicted by this model are a function of:

- Distance of the receiver from the roadway;
- Relative elevations of the roadway and receptors;
- Traffic volume;
- Vehicle mix (percentage of medium and heavy duty trucks);
- Vehicle speed;
- Roadway grade;
- Topographic features; and
- Noise source height of the vehicles. (9)

Noise impact analysis using STAMINA 2.0/OPTIMA requires the following traffic data for each link on the facility, and major cross streets of the alternative(s) under consideration, including the no-build:

- Existing average weekday traffic with K and D factors or existing directional peak-hour traffic;
- Design year average weekday traffic with K and D factors or directional design hour volumes;
- Existing and design year average travel speeds; and
- Vehicle mix volumes or percentages for the peak hour for autos, medium duty trucks (two-axle, six-tire) and heavy duty trucks (three-axle and above).

Table 5
FHWA Noise Abatement Criteria

Activity Category	*Leq*(h)	L*10*(h)	Description of Activity Category
A	57 dBA Exterior	60 dBA Exterior	Lands on which serenity and quiet are of extraordinary significance and serve an important public need and where the preservation of those qualities is essential if the area is to continue to serve its intended purpose.
B	67 dBA Exterior	70 dBA Exterior	Picnic areas, recreation areas, playgrounds, active sports areas, parks, residences, motels, hotels, schools, churches, libraries, and hospitals.
C	72 dBA Exterior	75 dBA Exterior	Developed lands, properties, or activities not included in Categories A or B above.
D	---	---	Undeveloped lands.
E	52 dBA Interior	55 Interior	Residences, motels, hotels, public meeting rooms, schools, churches, libraries, hospitals, and auditoriums.

Source: (10)

Using a user-friendly version of STAMINA 2.0 developed by TxDOT's Design Division, four series of model runs were made to verify the extent to which changes in traffic volumes impact predicted noise levels. The runs were selected to analyze the separate effect changes in auto volumes, heavy-duty truck volumes, and medium truck volumes have on predicted noise levels. The effect of change in total truck volume, using an equal distribution of heavy- and medium-duty trucks was also analyzed. All variables except traffic volumes were held constant, and all runs were made with speeds of 55 mph.

Figure 3 shows the relative increase in predicted *Leq* noise levels for increased volumes of automobile traffic. What is not immediately apparent from the graph, as discussed above, is that doubling the energy level (in this case the volume of autos) resulted in a 3 dBA increase in the predicted noise levels; tripling the volume of autos resulted in an increase of approximately 5 dBA; and an increase of 10 times the volume resulted in a 10 dBA increase in the predicted sound level. When trucks are included in the traffic (regardless of the percentage of the total volume) the same increase in noise occurs relative to the increase in total traffic volumes.

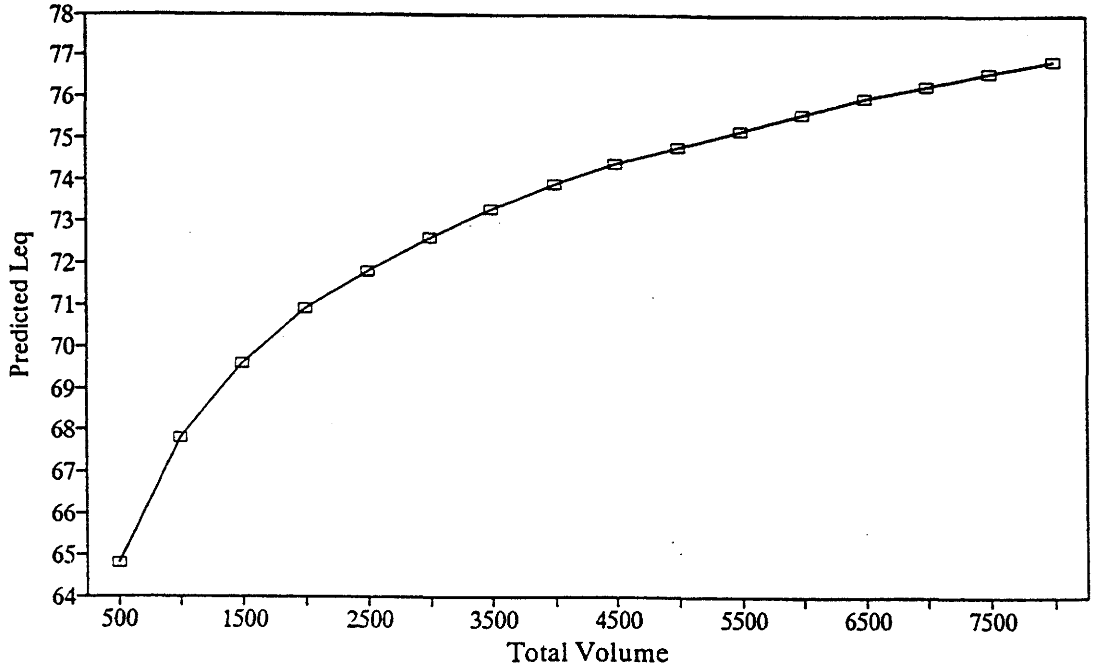


Figure 3. Effect of Traffic Volumes on Noise Levels

To determine the impact of truck volumes on noise, an effort was made to determine a relationship between changes in the percentage of trucks relative to the total traffic volume and changes in the predicted sound level. One source indicated that a 1 dBA increase in the predicted *Leq* can be expected for every 2.5 percent increase in trucks (11). The results of the model runs made with STAMINA 2.0/OPTIMA using an equal distribution of medium- and heavy-duty trucks confirmed that approximately a 1 dBA increase in the predicted sound level (*Leq*) occurs when the percentage of trucks is increased by 2.5 percent. The analysis of medium-duty trucks alone revealed that for every 3.7 percent increase in the percentage of trucks, a 1 dBA increase in the predicted *Leq* can be expected. For heavy-duty trucks alone, a 2.1 percent increase in the percentage of trucks resulted in a 1 dBA increase in the predicted sound level.

Design Division staff indicated that a project is considered to have noise impacts if it exceeds the NAC sound levels for the activity category for the identified receiver or if the project results in a 5 dBA increase over the existing sound level for a receiver. This means that the noise impact analysis procedures may be considered to be both sensitive and not sensitive to changes in traffic volume forecasts. On one hand, traffic volumes must triple to result in a "significant" increase (5 dBA) in the predicted sound level. On the other hand, it takes an increase of only .1 dBA to exceed the NAC for any activity category of receivers. Table 6 shows the expected increase in dBA relative to a range of percentage volume increases in total traffic.

Air Quality Analysis

Analysis of air quality impacts is required for all state and federal projects that have traffic volumes exceeding 1,500 vehicles per day, add capacity or thru lanes, or are new location projects. Those projects which are excluded from air quality analysis include those that have volumes of less than 1,500 vehicles per day and most categorical exclusions outlined in Part II-B of the **Operations and Procedures Manual** (12). A project air quality analysis includes modeled estimates of carbon monoxide (CO) concentrations for the present year conditions, the project build year and design year conditions, and the no-build design year conditions. All projects are modeled using worst-case meteorological conditions. Depending on the project, CO concentrations may be modeled at identified receptors along the project or at the right-of-way line.

Table 6
Predicted Change in *Leq* Relative to Change in Total Traffic Volume

Percentage Change in Total Traffic Volume	Approximate Change in *Leq* (dBA)
10 - 12	0.5
13 - 14	0.6
15 - 17	0.7
18 - 20	0.8
25	1.0
30	1.2
40	1.5
50	1.8
100	3.0
300	5.0

Two models, MOBILE4.0 and CALINE3 are used to estimate CO concentrations for a project. MOBILE4.0 is an EPA model developed to calculate the basic emission rates for an area based on characteristics of the vehicle fleet such as vehicle age and type, the percentage of cold starts, vehicle speeds, and ambient temperatures. These emission rates can be calculated for a designated year using estimated changes in fleet characteristics, speeds, cold starts, and other parameters (13). It should be noted that the EPA now requires that the updated version of this model, MOBILE5.1 be used. The differences between the two versions are important and will affect the results of air quality analyses. Specifically, the CO emission rates produced under MOBILE4.0 decrease consistently as vehicle speeds are increased. With MOBILE5.1, however, the CO emission rates will decrease as speeds are increased up to approximately 48 mph. Once the speed of 48 mph has been exceeded, the emission rates begin to increase. This is important in preparing traffic forecasts for project air quality analyses, because if the proposed improvements result in projected traffic volumes indicating average speeds of 48 mph or better, the emission rates will increase and may cause an adverse effect on estimated CO concentrations, particularly where the traffic volumes have increased significantly as a result of the project.

TxDOT is in the process of implementing MOBILE5.1 for use in air quality analyses; thus, it was not available for use in these analyses. However, once the emission rates under MOBILE5.1 have been produced, the effect of changes in forecast volumes and the resulting estimates of speed on CO concentrations can easily be verified.

CALINE3 is a California line source model that calculates estimates of CO concentrations along transportation facilities based on source strength (emission rates X traffic volume), meteorology, site geometry, and site characteristics. The model can reliably predict CO concentrations for receptors up to 150 meters off the roadway (14).

The traffic data required for air quality analyses include the following for each network link of each alternative (including the existing and no-build) and for major cross roads:

- AWDT with the K and D percentages or directional peak-hour volumes for the estimated time of project completion (ETC), ETC + 10 years and ETC + 20 years;
- Average speeds (existing, ETC, ETC + 10 years, and ETC + 20 years); and
- Vehicle mix volumes or percentages for ETC, ETC + 10 years, and ETC + 20 years :
    - light-duty gas vehicles
    - light-duty gas truck 1
    - light-duty gas truck 2
    - heavy-duty gas vehicle
    - light-duty diesel vehicle
    - light-duty diesel truck
    - heavy-duty diesel vehicle
    - motorcycle

No analysis was performed on MOBILE4.0 to determine the effects of changes in the vehicle classification data on emission rates, because these rates are determined using existing area-specific vehicle registration data and estimates of future vehicle registrations based on anticipated improvements for vehicle emission controls and the expected retirement rate for early model year vehicles.

Documentation for CALINE3 indicates that the source strength input (emission rate X traffic volume) is directly proportional to the predicted concentration. This means that a doubling of the emission rate or the traffic volume will result in doubling the predicted CO concentration (14).

A series of CALINE3 model runs were made to verify the effect of traffic volume changes on estimated CO concentrations at the right-of-way. An emission rate of 27.37 grams/kilometer and a background ambient CO concentration of .4 were used in these runs. The results of these runs are illustrated in Figure 4. As indicated, the predicted CO concentrations in parts per million (ppm) are proportional to the increases in traffic volumes. What is not immediately apparent in the figure is that a 20 percent increase in traffic results in approximately a 20 percent increase in CO concentrations, a 50 percent increase in traffic results in approximately a 50 percent increase in CO concentrations, and a doubling of traffic results in doubling the predicted concentration of CO. (The graph does not exhibit a straight line for the 4-lane road due to rounding by the CALINE3 model).

Several series of CALINE3 model runs were also made to verify the effect of reductions in speed due to increased traffic volumes on the predicted CO concentration. The 1990 MOBILE4.0 emission rates and ambient CO concentration for Dallas County were used in these runs. To perform these runs, average speeds based on volumes per hour per lane were assumed (Table 7), and the appropriate emission rate for the average speed was used. The combined effect of increased volumes/reduced speed and higher emission rates versus increased volumes/constant speed on predicted CO concentrations are shown in Figure 5. By increasing traffic 25 percent from 1,600 vphpl to 2,000 vphpl, thereby reducing speeds from 55 mph to 35 mph, the CO concentration is increased approximately 82 percent. Thus, under MOBILE4.0, once volumes reach the level of 1,600 to 1,700 vphpl, the average speed of the proposed facility is expected to be reduced below 55 mph, and the predicted CO concentration can be expected to increase by a greater percentage than the increase resulting from an increase in traffic volume alone. The precise increase will depend on the emission rates for each project area. Under MOBILE5.1 the optimum traffic volumes for improving air quality

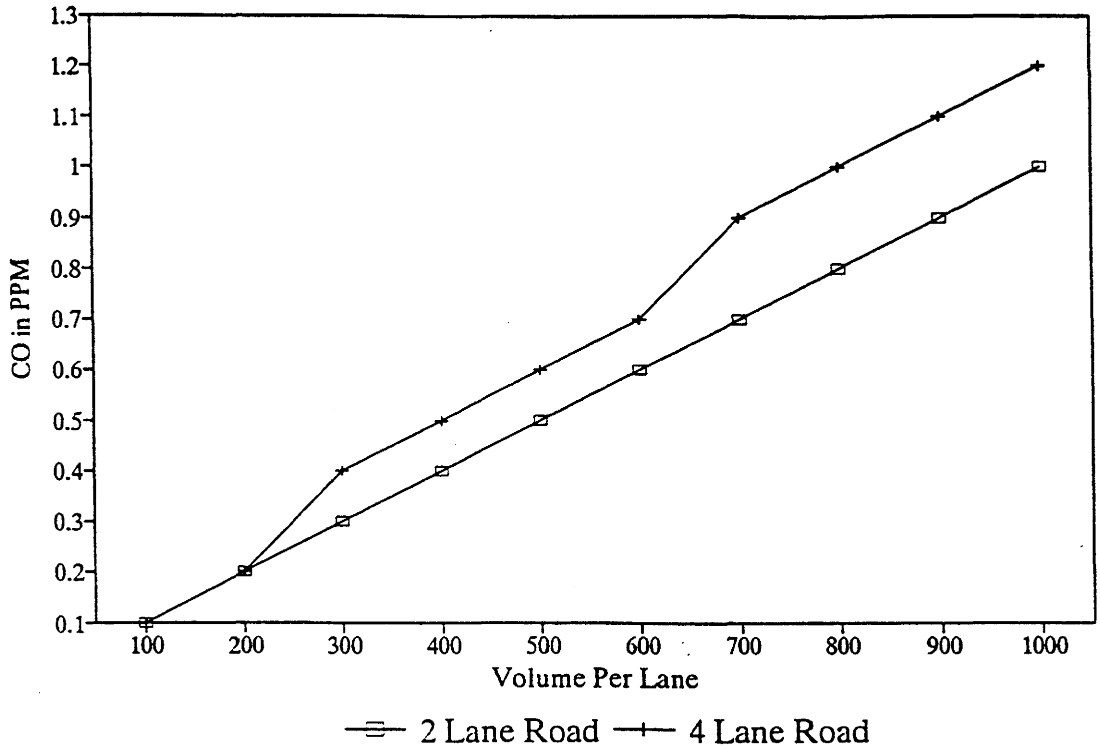


Figure 4. Effect of Volume Change on CO Concentration

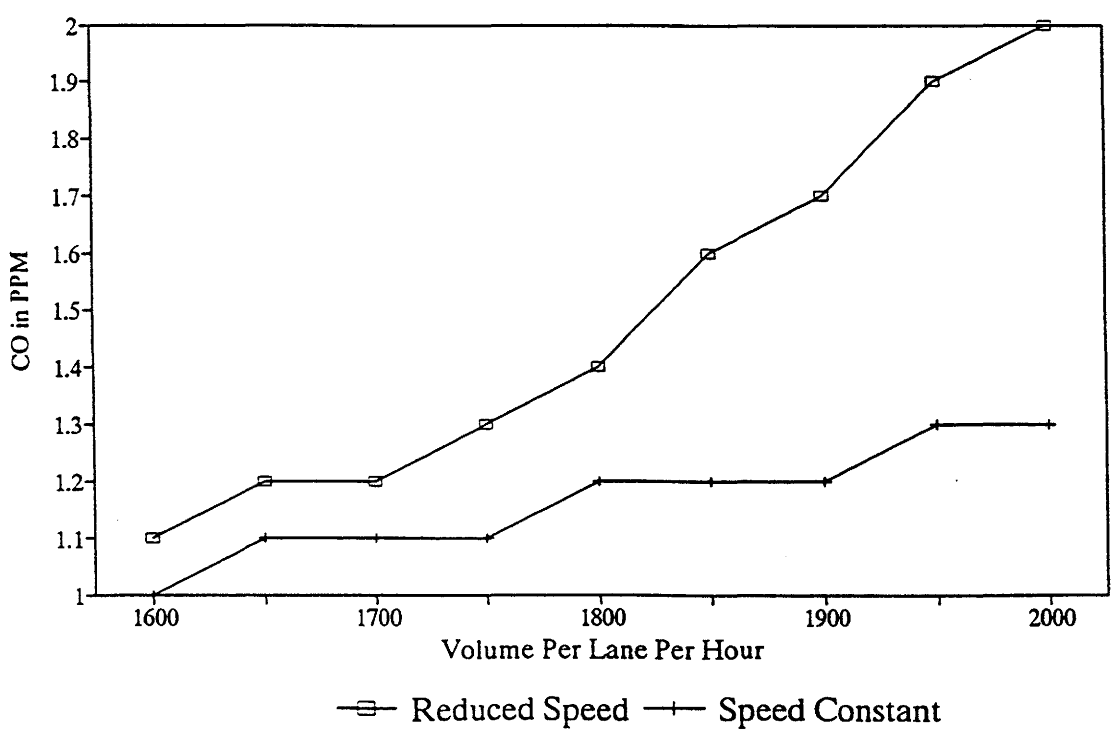


Figure 5. Effect of Volume and Speed Changes on CO Concentrations

Table 7
Assumed Average Speeds and Hourly Traffic Volumes

Average Speed	Volume/Hour/Lane
35	2000
40	1900
45	1800
50	1700
55	1600

are expected to be those where speeds average 48 mph. Any increase or decrease in volumes resulting in higher or lower average speeds will result in an increase in the predicted CO concentrations.

Obviously, changes in forecast traffic do have an impact on the CO concentrations predicted by CALINE3. Conversations with personnel in TxDOT's Design Division indicate that a 5 percent increase in the CO concentration over existing CO levels is considered a significant impact in air quality. Although the emission rates are projected to decrease significantly during the next decade, projects that increase the capacity of a facility to reduce congestion may also allow for an increase in the volume and speed of traffic using the facility. These increases may be significant enough to offset any emission reductions gained through improved vehicle controls, inspection/maintenance programs, and cleaner fuels. Under the 1990 Clean Air Act Amendments' conformity requirement, no particular transportation project may cause or contribute to any violation of any air quality standard or delay their timely attainment. Thus, in areas where reductions in mobile emissions are mandated, forecast traffic volumes and speeds will become increasingly important in gaining project approvals with regard to air quality.

PROJECT DESIGN

Geometric Design

During this stage of project development, the preliminary design of a facility (mainlanes, ramps, interchanges, intersections, driveway access, etc.) is prepared to a sufficient level of detail that the right-of-way requirements can be established and the environmental analyses prepared. For most facility types (controlled access highways, multi-lane noncontrolled access highways, 2-lane rural highways, and urban arterials) certain aspects of the design are controlled by traffic volumes. Those design features controlled by volume are specific to each project type and are usually determined through a process called design analysis. Design analysis procedures for specific facility types and sections are generally the same as those used in an operations analysis. Design analysis is used to select the appropriate laneage and lane configurations to ensure that the facility will accommodate the forecast traffic volume at the desired LOS.

The design analysis procedures and the specific design features affected by volumes are discussed separately for each facility type. Unless otherwise identified, the procedures, concepts, and definitions are from the **Highway Capacity Manual**.

Before proceeding with the discussion of design analysis, the factors affecting the capacity and, thus, influencing the design need to be described. The definitions and descriptions below are taken from the **Highway Capacity Manual**.

Capacity - The capacity of a facility is the maximum hourly rate at which vehicles can reasonably be expected to traverse a point or uniform section of a lane or roadway during a given period of time under prevailing roadway, traffic, and control conditions. The time period used in capacity analysis is usually 15 minutes, although capacity volumes are commonly expressed in an equivalent hourly rate.

As indicated by the definition above, three groups of factors affect the facility's capacity: roadway factors, traffic factors, and control factors. Roadway factors refer to the geometric characteristics of the facility and include:

- The type of facility and the development environment (urban, suburban, CBD, etc.),
- Lane widths,
- Shoulder widths and/or lateral clearances,
- Design speed, and
- Horizontal and vertical alignments.

Traffic factors involve the characteristics of the traffic stream using the facility and include:

- The percentage of vehicles by type (heavy trucks, buses, and/or recreational vehicles),
- Lane distribution,
- Directional distribution,
- Peak-hour factor, and
- Driver population factor.

Control condition factors involve the types and design of traffic control devices and regulations on a given facility. These include:

- The location, type, and timing of traffic signals;
- Stop and/or yield signs;
- Lane use restrictions;
- Turn restrictions; and
- Traffic metering devices.

Much of the data describing capacity in the **Highway Capacity Manual** for each type of facility are based on ideal conditions. Ideal conditions assume that no roadway, traffic, or control factors are affecting the capacity of the facility. Ideal conditions for uninterrupted flow facilities and for signalized intersections are described below.

Uninterrupted flow:

1. Twelve-foot lane widths,
2. Six-foot clearance between edge of travel lanes and the nearest roadside or median obstruction,
3. Seventy mph design speed for multi-lane facilities and 60 mph design speed for 2-lane highways, and
4. All passenger cars in the traffic stream.

Signalized intersections:

1. Twelve-foot lane widths,

2. No curb parking on approaches,

3. Level grade,

4. All passenger cars in traffic stream,

5. All vehicles traveling straight through the intersection,

6. Intersection located in a non-CBD area, and

7. Green signal available at all times.

Conditions are not ideal for most projects; therefore, adjustments to the capacity values must be made according to the specific project characteristics. The adjustment factors affecting the different design analysis procedures will be discussed with each section.

Level of service - Level of service (LOS) is a qualitative measure used to describe the operational characteristics within a traffic stream. Six levels of service, A through F, are defined for each type of facility. The definition for each LOS by facility type usually describes the operating characteristics in terms of speed, density, maximum service flow rate, volume to capacity (v/c) ratio, freedom to maneuver, traffic interruptions, comfort, and convenience. While the levels of service definitions include several parameters that describe the operating conditions, one parameter is used to define the levels of service for each type of facility. Each parameter is called the measure of effectiveness and is shown in Table 8 for each facility type.

Service flow rate - The **Highway Capacity Manual** defines the service flow rate as the maximum hourly rate at which vehicles can reasonably be expected to traverse a uniform section of a lane or roadway during a given time period, usually 15 minutes, under prevailing roadway, traffic, and control conditions while maintaining a designated LOS. Maximum service flow rates are defined for each LOS (except for LOS F in which flow is highly unstable) for each facility type and, thus, define the boundaries between the various levels of service.

Table 8
Measures of Effectiveness for Level of Service Definition

Type of Facility	Measure of Effectiveness
Freeway Basic freeway segments Weaving areas Ramp junctions	 Density (passenger cars/miles/lanes) Average travel speed (mph) Flow rates (passenger cars per hour)
Multi-lane Highways	Density (passenger cars/miles/lanes)
2-Lane Highways	Percentage time delay (%) Average travel speed (mph)
Signalized Intersection	Average individual stopped delay (seconds/vehicle)
Unsignalized Intersections	Reserve capacity (passenger cars per hour)
Arterials	Average travel speed (mph)

Source: (15)

Volume - Volume is the total number of vehicles (either actual counted vehicles or forecast vehicles) that pass over a section of a lane or roadway during a given time interval. The volume may be expressed as annual, daily, hourly, or subhourly.

Rate of flow - The equivalent hourly rate at which vehicles pass over a section of a lane or roadway during a given time interval less than one hour, usually in 15 minutes, is the rate of flow. Thus, the rate of flow for the vehicles passing a given point in 15 minutes would be equal to four times the 15-minute volume.

Peak rate of flow - This is the highest rate of flow (expressed hourly) for volumes observed for consecutive uniform time periods during the period of one hour. For example, volumes observed for four consecutive 15-minute periods resulted in the following counts: 1,200 vehicles, 1,000 vehicles, 1,050 vehicles, and 1,100 vehicles. The total hourly volume observed would be 4,350 vehicles, but the peak rate of flow would be 4 x 1,200 or 4,800 vehicles. Peak rates of flow are related to hourly volumes through the use of a peak-hour factor.

Peak-hour factor - The peak-hour factor (PHF) is the ratio of the total hourly volume to the maximum 15-minute rate of flow within the hour. When the PHF is known, it is used to convert peak-hour or design-hour volumes to a peak rate of flow with the following equation:

$$v = V/PHF$$

Where:

v	=	rate of flow for peak 15-minute period in vehicles per hour
V	=	peak-hour volume in vph
PHF	=	peak-hour factor

Controlled Access Highways

The geometric design of controlled access highways is based on defined standards for horizontal and vertical geometry, safety, and signing as outlined in the Design Division's **Operations and Procedures Manual.** Design elements such as horizontal and vertical alignments, superelevation, lateral clearances, and lane and shoulder widths are governed by standards which are based on the selected design speed. Other design elements such as pavement cross slopes, ditches, roadside obstructions, or drainage facilities are based on standards developed for safety. The elements of controlled access freeway design that are based on or controlled by traffic volumes include the number of lanes and the selection of lane configurations for individual freeway elements.

To determine the number of lanes and to select the appropriate lane configurations for smooth operation of the freeway, a design analysis of the freeway using analysis procedures from the **Highway Capacity Manual** is performed. The objective of the design analysis is virtually the same as the planning objective: to determine the number of freeway lanes needed to achieve the desired LOS for the traffic flows and characteristics. The main difference is that more information and detail are required in the design analysis.

The traffic forecast information needed for the design analysis for controlled access facilities includes:

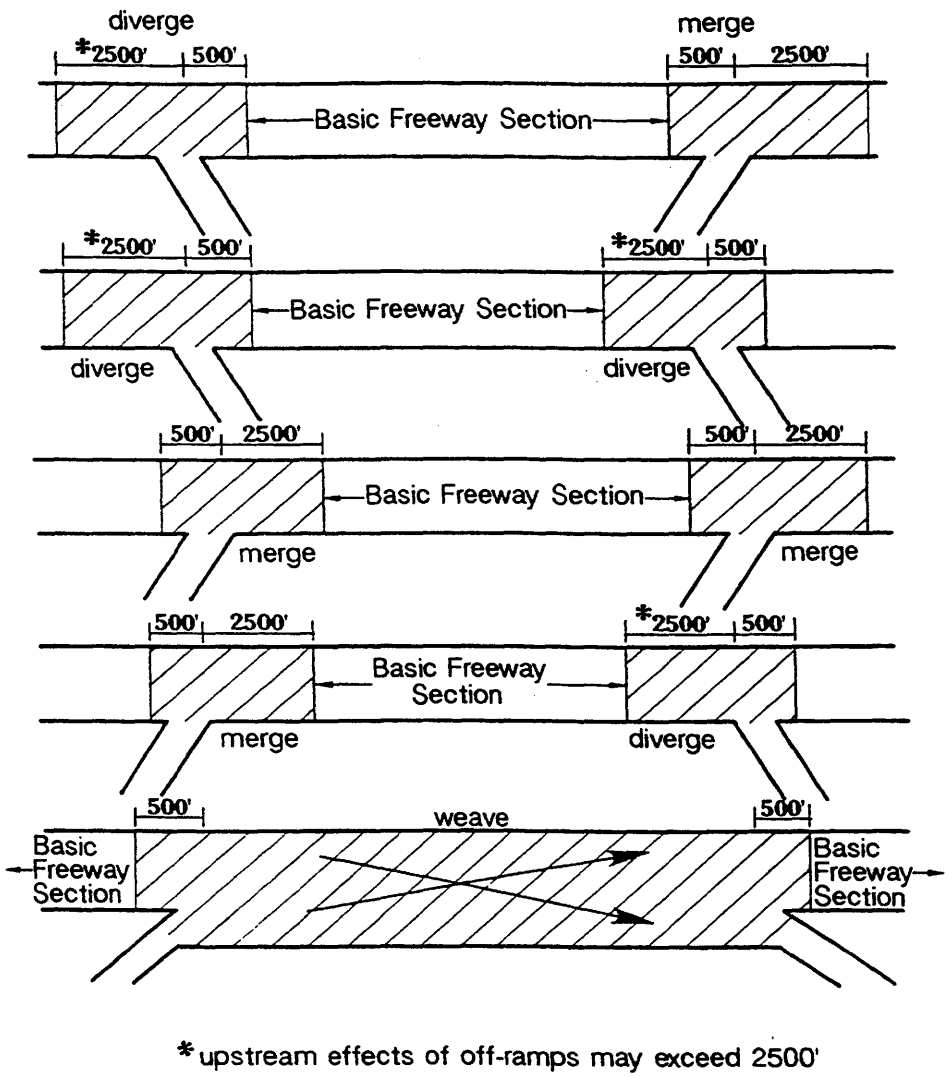
- Directional design hour volumes (DDHV) for each link on the facility (including all ramps and interchanges) and major cross streets (no adjustments should be made to the volumes regarding the percentage of trucks during the design hour because this is handled differently in each analysis);

- Design year percentage of heavy trucks, recreational vehicle and buses;
- Design-year PHF; and
- Design-hour turning movements for all cross street/frontage road and/or ramp intersections (the use of cross street/frontage road turning movements in freeway design is discussed under arterial design analysis).

Additionally, design standards such as design speed, lane widths, and lateral clearances must be specified; the horizontal and vertical alignment of the freeway and the approximate location of ramps and interchanges must be available; and the design LOS which determines the v/c ratio must be selected. Because the controlled access design standards prescribed in the Design Division's **Operations and Procedures Manual** (16) require a design speed of 70 mph, 12-foot lane widths, and shoulder widths exceeding the 6-foot lateral clearance requirement (except the minimum inside shoulder on 4-lane facilities) and because the emphasis of this report is on traffic requirements, no discussion on the effect of these roadway factors is included. Additionally, the effect of grades on capacity is considered only in determining the effect of heavy vehicles on traffic.

To conduct a design analysis, the freeway must be divided into segments that have uniform characteristics. Segmenting the freeway usually includes dividing the freeway into basic freeway or mainlane sections, ramp and interchange junctions, and weaving sections. An example of the various segments and areas of influence are shown in Figure 6. Additionally, major terrain changes should be identified to determine any grades that need separate consideration. Each type of freeway segment will be analyzed separately using different procedures and measures of effectiveness to determine the number of lanes and the lane configuration to provide the desired LOS.

The following sections briefly describe the procedures, LOS criteria, and volumes associated with the design analysis of the basic freeway segments, ramps and ramp junctions, and weaving areas. The procedures associated with design analysis are based on precise concepts and definitions in the **Highway Capacity Manual**. A thorough understanding of the concepts found in the **Highway Capacity Manual** is needed to perform the design analyses. The intent of these sections is to identify the general traffic factors affecting the design of



Source: (15)

Figure 6. Freeway Components and Areas of Influence

each type of freeway section and to attempt to quantify the influence these factors have on design in an effort to describe the accuracy needed in forecasting for design. Because there are numerous factors involved in any one of these procedures, sufficient information and detail are not provided to cover all conditions that may occur in the design analysis of each type of freeway section. Nor are all the equations, tables, nomographs, or figures used in the computations of these procedures included.

Basic freeway section. The goal in the design analysis of basic freeway segments is to determine the number of lanes needed in each direction along the identified section of the facility. Basic freeway segments are defined in the **Highway Capacity Manual** as those "sections of the freeway that are unaffected either by merging or diverging movements at nearby ramps or by weaving movements." The following equation is used to calculate the number of lanes required:

$$N = SF/(MSF \times f_W \times f_{HV} \times f_P)$$

Where:

SF	=	service flow rate determined by dividing the forecast DDHV by the PHF
MSF	=	maximum service flow for the selected LOS or v/c ratio
f_W	=	factor to adjust for the effects of restricted lane widths and/or lateral clearances
f_{HV}	=	factor to adjust for heavy vehicles
f_P	=	factor to adjust for effect of driver population

As indicated by the above equation, the traffic forecast information and factors required to perform the design analysis for basic freeway sections include the design-hour volume, the percentage of heavy vehicles by type, the PHF, and the driver population factor. A selected design LOS is also needed to complete the analysis.

This procedure produces the number of mainlanes required for each direction, although judgment is needed in using the results. Often successive segments of the facility produce a different value for N. Specific requirements and conditions should be followed for lane additions or deletions. Additionally, solving for N often results in a fractional number; judgment must be made on whether to use the next full integer or to raise the v/c ratio and use the next smaller integer.

Table 9 shows the LOS criteria for basic freeway sections for design speeds of 70, 60, and 50 mph. The capacity of basic freeway sections under ideal conditions is equal to the maximum service flow rate for LOS E (2,000 pcphpl for 70 and 60 mph design speeds, and 1,900 pcphpl for 50 mph design speeds). The maximum service flow rates represent the maximum volume in passenger cars per hour per lane (pcphpl) which can be accommodated under ideal conditions while maintaining the specific LOS.

It is desirable to design controlled access facilities for a LOS C or better. In large urban areas, however, the impacts and costs of right-of-way often prohibit design for any LOS better than D or E. The service flows for LOS A and B are quite large and are less sensitive to changes in forecast traffic volumes. Thus, for the purpose of this discussion, LOS ranges of C, D, and E will be used for design.

As indicated by the equation for N, the maximum service flow values in Table 9 must be factored to reflect the project roadway and traffic conditions. The two traffic factors are the percentage of heavy vehicles by type and the driver population factor.

The factor to account for heavy vehicles (f_{HV}) is determined through two steps. The first step is to determine the passenger car equivalent (pce) for each truck, bus, and/or recreational vehicle for the roadway conditions (level, rolling, or mountainous terrain). The pce values for trucks (E_T), buses (E_B), and recreational vehicles (E_R) for the roadway conditions are given in Table 10 and represent the number of passenger cars that would use the same capacity as one truck, bus, or recreational vehicle on the designated terrain. Grades of less than 3 percent and more than 1 mile in length or grades of more than 3 percent and 1/2 mile or more in length should be considered as separate sections in determining the pce and a separate equivalency table used.

Table 9
Levels of Service for Basic Freeway Sections

LOS	Density (pc/mi/ln)	70 mph Design Speed			60 mph Design Speed			50 mph Design Speed		
		Speed[b]	v/c	MSF[a]	Speed[b]	v/c	MSF[a]	Speed[b]	v/c	MSF[a]
A	≤ 12	≥ 60	0.35	700	---	---	---	---	---	---
B	≤ 20	≥ 57	0.54	1100	≥ 50	0.49	1000	---	---	---
C	≤ 30	≥ 54	0.77	1550	≥ 47	0.69	1400	≥ 43	0.67	1300
D	≤ 42	≥ 46	0.93	1850	≥ 42	0.84	1700	≥ 40	0.83	1600
E	≤ 67	≥ 30	1.00	2000	≥ 30	1.00	2000	≥ 28	1.00	1900
F	> 67	< 30	[c]	[c]	< 30	[c]	[c]	< 28	[c]	[c]

[1] Maximum service flow rate per lane under ideal conditions
[2] Average travel speed
[3] Highly variable, unstable
Source: (15)

Table 10
Passenger Car Equivalents for Basic Freeway Segments

Factor	Type of Terrain		
	Level	Rolling	Mountainous
E_T for Trucks	1.7	4.0	8.0
E_B for Buses	1.5	3.0	5.0
E_R for RVs	1.6	3.0	4.0

Source: (15)

The second step is to determine the adjustment factor for heavy vehicles (f_{HV}) using the equation:

$$f_{HV} = 1/[1 + P_T(E_T - 1) + P_R(E_R - 1) + P_B(E_B - 1)]$$

Where:

f_{HV} = adjustment factor for the effect of heavy vehicles in the traffic stream

$E_{T,B,R}$ = passenger car equivalents for trucks, buses, and recreational vehicles, respectively

$P_{T,R,B}$ = percentage of trucks, recreational vehicles, and buses, respectively, in the traffic stream

For most controlled access facilities in Texas the percentage of buses and RVs in the traffic stream during the design hour is so small that they do not need to be handled separately. Where trucks represent the majority of heavy vehicles (usually when the percentage of trucks is five times the total percentage of buses and RVs together), it is reasonable to consider all heavy vehicles as trucks. When this is the case, the equation for the adjustment factor becomes

$$f_{HV} = 1/[1 + P_T(E_T - 1)]$$

Where:

P_T = percentage of heavy trucks, buses, and RVs on the freeway

E_T = pce factor for each truck

Using the above equation, the pce factor for each truck from Table 10, and the usual range of percentage trucks during the design hour, adjustment factors for heavy vehicles were calculated. These adjustment factors are shown in Table 11.

Table 11
Adjustment Factor for the Effect of Trucks

PCE[a] E_T E_B E_R	Adjustment Factor for Heavy Vehicle (f_{HV})								
	Percentage of Trucks (P_T)								
	2	3	4	5	6	7	8	9	10
1.7	.99	.98	.97	.97	.96	.95	.95	.94	.93
4	.94	.92	.89	.87	.85	.83	.81	.79	.77
8	.88	.83	.78	.74	.70	.67	.64	.61	.59

Source: (15)

The second traffic factor used to adjust the maximum service flow (MSF) found in Table 9 is the driver population factor. The values in Table 9 represent service flows for regular weekday commuter drivers in the traffic stream. Although data are scarce, it is generally acknowledged that weekend, recreational, and, in some instances, off-peak drivers use the freeway less efficiently. Thus, the service flows should be factored according to the characteristics of the driver population expected to use the facility. A range of values is given in Table 12 for driver population for weekday, commuters, and other (weekend, recreational, off-peak) drivers. The analyst must use judgment in determining the appropriate value.

Table 12
Driver Population Factors

Traffic Stream Type	Factors, f_P
Weekday or Commuter	1.0
Other	0.75 - 0.90[a]

Source: (15)

In addition to the MSF value adjustments needed in Table 9, the DDHV must be adjusted by the PHF. This adjustment is required to make the DDHV reflect the peak 15-minute flow so that the facility design will have sufficient capacity to avoid breakdown during the peak flow of the design hour. Where the PHF for a facility is known, that value should be used. If the PHF is unknown, Table 13 provides estimates of these values based on the area's population in which the project is located. As discussed in the planning analysis, different sections of the same facility can have different peaking characteristics. Assuming that all components and sections of a facility have the same PHF may produce a more conservative design of the facility if the overall PHF used is appropriate.

Table 13
Peak Hour Factors

Metropolitan Area Population (000s)	PHF
< 500	0.77
500 to 1,000	0.83
Over 1,000	0.91

Source: (16)

Ramps and ramp/freeway junctions. The goal of the ramp and ramp junction analysis is to determine the number of lanes required on the ramp itself and the freeway/ramp lane configuration required at the ramp/freeway junction. Ramps are defined in the **Highway Capacity Manual** as a "length of roadway providing an exclusive connection between two facilities." They are designed to allow high-speed merging and diverging movements with minimal disruption to the freeway traffic flow. The influence area of ramps on freeway operations is illustrated in Figure 6.

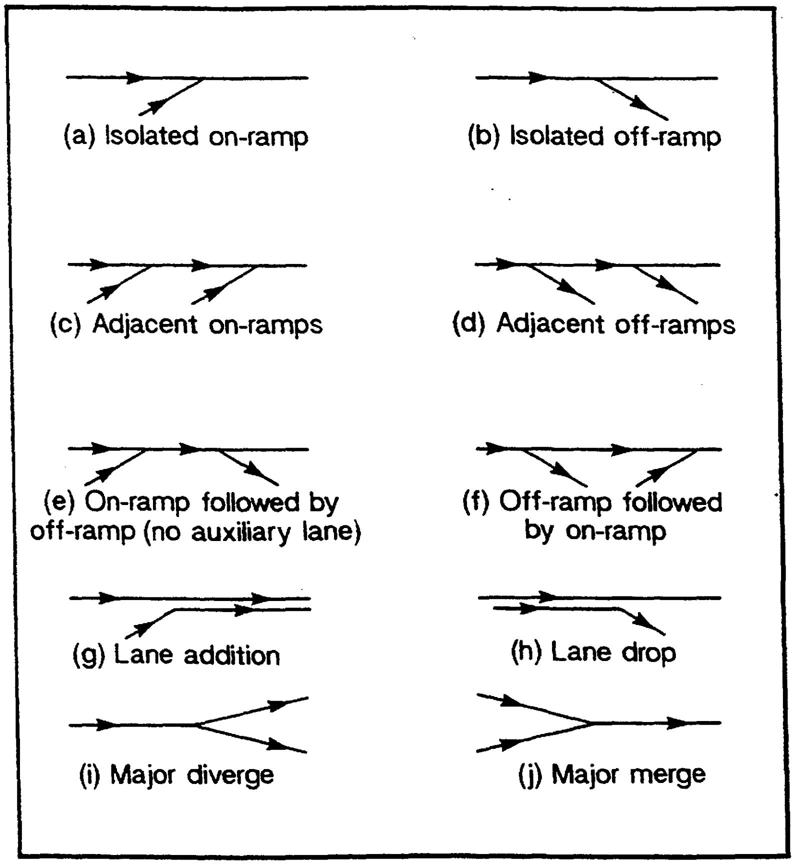
There are numerous possible design configurations for ramps along freeways. Those that are covered by analysis procedures in the **Highway Capacity Manual** are shown in Figure 7. The design analysis for ramps and ramp/freeway junctions is the same procedure used for operations analysis of ramps and ramp junctions. That is, the procedure used is a trial-and-error process. A ramp or series of ramps may initially be designed to meet the standard design criteria for ramps as set forth in the **Operations and Procedures Manual** (16). However, the design analysis of that ramp/freeway configuration may indicate an LOS below the designated design LOS. Thus, either the ramp/freeway design must be altered or the selected LOS criteria lowered for the project.

The elements included in the analysis of ramp junctions are discussed below and illustrated in Figure 8. Merge volume (V_m) is applicable to on-ramps and represents the total a volume of the traffic streams that will join or merge together. Thus, V_m is equal to the sum of the ramp volume and the lane volume to which it will merge. Diverge volume (V_d) applies to off-ramps and is the total volume in the traffic stream that will separate. Thus, for a right-side, 1-lane off-ramp, the diverged volume is equal to the volume in the right freeway lane (Lane 1) immediately upstream of the ramp. Freeway volume (V_f) is the volume on the freeway where it would be at its highest level relative to the ramp type. Thus, for on-ramps it is the volume immediately after the merge; for off-ramps it is the volume immediately prior to the diverge.

These volumes, V_m, V_d, and V_f, are the volumes to which the LOS criteria for ramps, shown in Table 14, are applied. The LOS criteria are in peak rate of flow which requires the DDHV to be converted using the PHF as previously described for basic freeway segments. The criteria given for the checkpoint volumes represent the flow rates that can be accommodated while allowing the freeway to operate at the designated LOS.

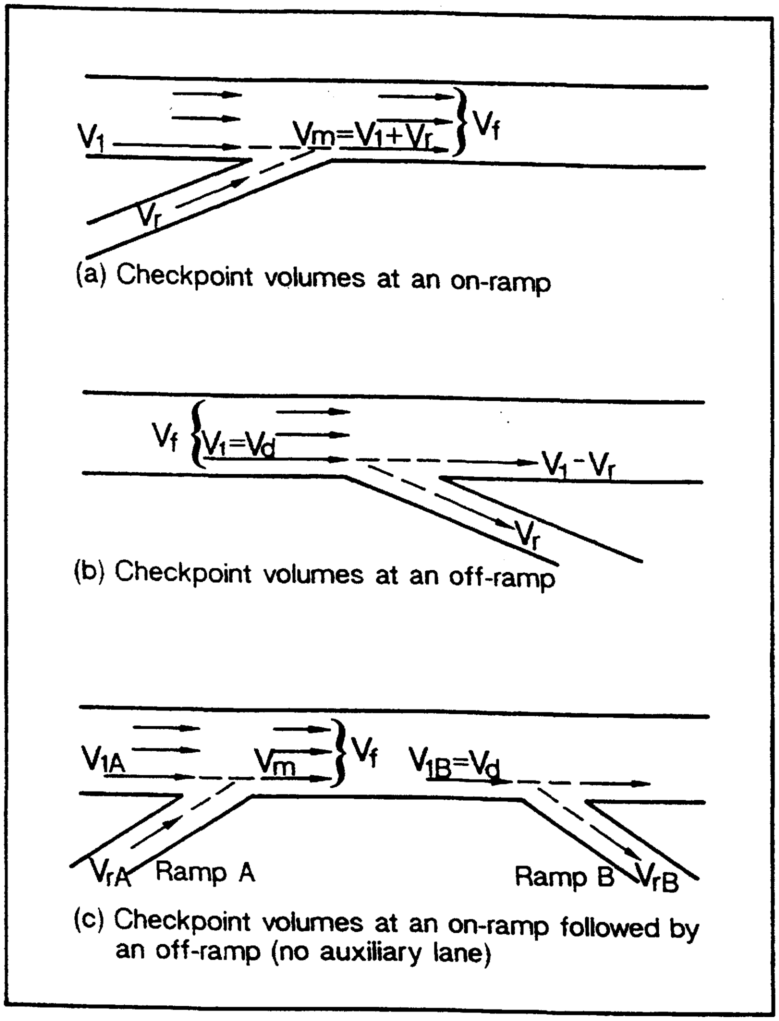
The procedures for analyzing ramp/freeway junctions are briefly described below. The first step is to compute the volume in Lane 1 of the freeway immediately upstream of the ramp. Because the merge and diverge maneuvers associated with ramps occur in the freeway lane adjacent to the ramp, the amount and type of traffic in this lane is important in this analysis.

Lane 1 volumes are dependent on the total ramp volume, the total freeway volume upstream of the ramp, the distance to the adjacent upstream or downstream ramp, the volumes on the adjacent upstream and/or downstream ramps, and the type of ramp and number of lanes. The **Highway Capacity Manual** uses a series of equations and nomographs to compute Lane 1 volumes for different freeway sizes and various ramp configurations. For freeway/ramp configurations which the nomographs do not cover, an approximation procedure is provided.



Source: (15)

Figure 7. Ramp Terminal Configurations



(a) Checkpoint volumes at an on-ramp

(b) Checkpoint volumes at an off-ramp

(c) Checkpoint volumes at an on-ramp followed by an off-ramp (no auxiliary lane)

Source: (15)

Figure 8. Ramp Terminal Checkpoint Volumes

Table 14
Level of Service Criteria for Ramp/Freeway Terminals
Checkpoint Flow Rates

Level of Service	Merge Flow Rate (pcph)[a] V_m	Diverge Flow Rate (pcph)[b] V_d	Freeway Flow Rates (pcph)[c], V_f								
			70 mph Design Speed			60 mph Design Speed			50 mph Design Speed		
			4 Lane	6 Lane	8 Lane	4 Lane	6 Lane	8 Lane	4 Lane	6 Lane	8 Lane
A	≤ 600	≤ 650	≤ 1400	≤ 2100	≤ 2800	d	d	d	d	d	d
B	≤ 1000	≤ 1050	≤ 2200	≤ 3300	≤ 4400	≤ 2000	≤ 3000	≤ 4000	d	d	d
C	≤ 1450	≤ 1500	≤ 3100	≤ 4650	≤ 6200	≤ 2800	≤ 4200	≤ 5600	≤ 2600	≤ 3900	≤ 5200
D	≤ 1750	≤ 1800	≤ 3700	≤ 5550	≤ 7400	≤ 3400	≤ 5100	≤ 6800	≤ 3200	≤ 4800	≤ 6400
E	≤ 2000	≤ 2000	≤ 4000	≤ 6000	≤ 8000	≤ 4000	≤ 6000	≤ 8000	≤ 3800	≤ 5700	≤ 7600
F	Widely Variable										

[a] Lane 1 flow rate plus ramp flow rate for 1-lane right-side on-ramps
[b] Lane 1 flow rate immediately upstream of off-ramp for 1-lane right-side ramps
[c] Total freeway flow rate in one direction upstream of off-ramp and/or downstream of on-ramp
[d] LOS not attainable due to design speed restrictions
Source: (15)

Once Lane 1 volumes have been computed, the truck presence in Lane 1 is determined. Like other vehicles, heavy trucks are not distributed equally among the freeway lanes. Trucks and other heavy vehicles tend to concentrate in the shoulder lanes; thus, Lane 1 generally has a higher percentage of trucks compared to the other lanes.

The third step is to convert all volumes to passenger car equivalents per hour. Volumes are converted using the heavy vehicle factor (f_{HV}) as determined for basic freeway segments.

The last step is to compute the checkpoint volumes for the merge, diverge, and freeway using the equations below:

$$V_m = V_{1A} + V_{rA}$$

Where:

V_m = merge volume immediately downstream of an on-ramp

V_{1A} = total volume in lane 1 immediately upstream of the on-ramp

V_{rA} = total volume on the on-ramp

$$V_d = V_{1B}$$

Where:

V_d = diverge volume immediately upstream of the off-ramp

V_{1B} = total volume in Lane 1 immediately upstream of the off-ramp

$$V_f = V_{fl} + V_{rA}$$

Where:

V_f = total freeway volume either upstream of an off-ramp, downstream of an on-ramp, or between an on-ramp and an off-ramp

V_{fl} = total volume in Lane 1

V_{rA} = total volume of the on-ramp traffic remaining in Lane 1 based on distance from the ramp

Once the checkpoint volumes have been computed, the volumes must be converted to peak-flow rates by dividing by the PHF. These volumes are then compared to the LOS flow rates as shown in Table 15. The results of this comparison will indicate whether the freeway/ramp configuration will operate at the selected design LOS. If the LOS for any of the checkpoint volumes is lower than for the freeway as a whole, then that LOS will be the controlling factor for the freeway operation; and improvements to the design will be explored. Options to improve the design might include adding an auxiliary lane where an on-ramp is followed by an off-ramp; making a ramp two lanes and adding or subtracting a lane to the freeway; eliminating a ramp; or braiding or channelizing ramp pairs. Any of these improvements can affect the number of lanes needed on the freeway as well as the lane configuration.

Little data exist regarding the operational characteristics of ramp roadways. Ramps are notably different from freeway mainlanes with regard to length, design speed, effect of heavy vehicles, and the effect of ramp/roadway terminals making it difficult to adjust the LOS criteria for freeways for use with ramps. Based on data from other research (17) estimates of capacity for ramps were developed in the **Highway Capacity Manual**. These are shown in Table 15.

The volumes in Table 16 apply only to the ramp roadway and do not account for the effect the ramp terminal may have on the operation of the ramp. As a general rule, when ramp volumes exceed 1,500 passenger cars per hour (pcph) a 2-lane ramp and 2-lane ramp/freeway terminal should be provided.

Weaving areas. The third design element of controlled access facilities affected by traffic volumes is the weaving area. Weaving areas exist where two or more streams of traffic traveling in the same direction along the facility must cross. Weaves occur when a merge area is closely followed by a diverge area or where an on-ramp is followed by an off-ramp and joined by an auxiliary lane.

Table 15
Service Flow Rates for Single-Lane Ramps[a] (pcph)

Level of Service	Ramp Design Speed (mph)				
	≤ 20	21 -30	31 -40	41 - 50	≥ 51
A	b	b	b	b	600
B	b	b	b	900	900
C	b	b	1100	1250	1300
D	b	1200	1350	1550	1600
E	1250	1450	1600	1650	1700
F	Widely Variable				

[a] For two lane ramps, multiply the values by:
1.7 for ≤ 20 mph
1.8 for 21 -30 mph
1.9 for 31 -40 mph
2.0 for ≥ 40 mph
[b] LOS not attainable due to restricted design speed
Source: (15)

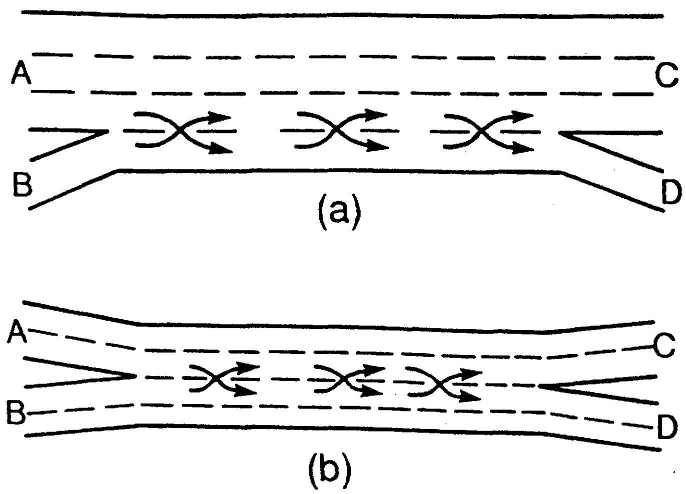
In addition to traffic volumes there are three geometric parameters that affect the operation of the weaving area: the number and configuration of lanes and the length of the section. The lane configuration refers to the type of weave. There are three types of weaves: Type A, Type B, and Type C. Type A weaves require that each weaving vehicle make one lane change in order to make the desired movement. Figure 9 shows two examples of Type A weaves. Figure 9a shows an on-ramp followed by an off-ramp and joined by an auxiliary lane. Figure 9b illustrates a major weave where two 2-lane roadways join to a 4-lane roadway and then diverge into two 2-lane roadways. Both of these weaves are similar in that each has a crown line, a lane line connecting the nose of the entrance ramp gore area to the nose of the exit ramp gore area; and each weaving vehicle must cross that crown line.

Type B weaves involve multi-lane entrance or exit legs and are characterized by two features; one weaving movement may be accomplished without making a lane change, and the other weaving movement requires at most one lane change. Figures 10a, b, and c show examples of Type B weaves.

Type C weaves are similar to Type B weaves in that one weaving movement does not require a lane change. Unlike Type B, however, the second weaving movement requires more than one lane change. Examples of Type C are shown in Figures 11a and b.

The second geometric parameter affecting the operation of weaving areas is the length of the weave section. The length affects the operation by limiting the time and space available for making the desired lane changes. Generally, as the length of the section decreases, the intensity of the lane changes increases causing more disruption to the traffic flow on the facility. Based on the procedures in the **Highway Capacity Manual**, the maximum weave length is 2,500 feet. Beyond this length, the merge and diverge movements tend to separate.

The number of total lanes within the weave section, as well as the number of lanes available for use by weaving vehicles, make up the third geometric parameter affecting the operation of the section, the weave width. Weaving vehicles use more space than nonweaving vehicles. The amount of space used is dependent on the total traffic volume, weaving and nonweaving, and the number of lane changes required in the weave. Under normal conditions the weaving vehicles will make use of the available lanes in such a way that all freeway component flows will achieve the same average running speed, with weaving speeds being



Source: (15)

Figure 9. Type A Weaving Sections

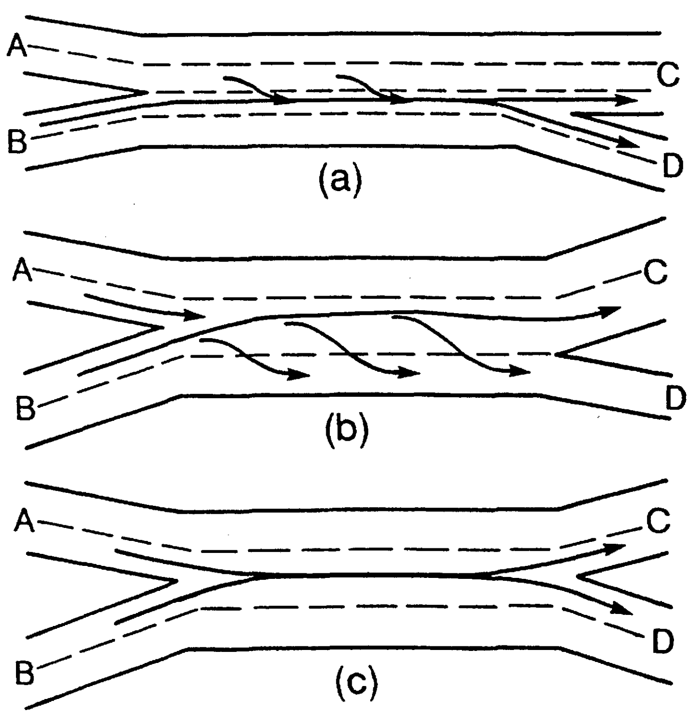


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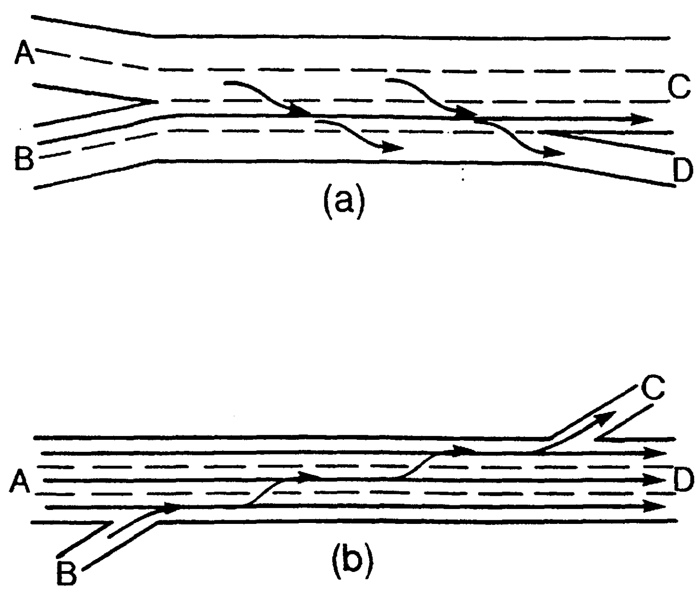
Figure 10. Type B Weaving Sections



Source: (15)

Figure 11. Type C Weaving Sections

slightly less than nonweaving speeds. In some cases the lane configuration limits the number of lanes that the weaving vehicles can utilize, and the area becomes constrained. In constrained areas the speed of nonweaving vehicles will be much higher than that of the weaving vehicles.

The geometric characteristics described are used in conjunction with various traffic flow data to analyze the operation within the weave section. These parameters and the symbols representing them in the analysis procedures are given in Table 16.

Table 16
Parameters and Symbols Used in the Analysis of Weaving Operations

Parameter Symbol	Parameter Definition
L	Length of weaving area in feet
L_H	Length of weaving area in hundreds of feet
N	Total number of lanes in the weaving area
N_w	Number of lanes used by weaving vehicles in the weaving area
N_{nw}	Number of lanes used by nonweaving vehicles in the weaving area
v	Total flow rate in the weaving area in passenger car equivalents (pcph)
v_w	Total weaving flow rate in the weaving area in pcph
v_{w1}	Weaving flow rate for the larger of the two weaving flows in pcph
v_{w2}	Weaving flow rate for the smaller of the two weaving flows in pcph
v_{nw}	Total nonweaving flow rate in the weaving area in pcph
VR	Volume ration v_w/v
R	Weaving ratio v_{w2}/vw
S_w	Average running speed of weaving vehicles in the weaving area in mph
S_{nw}	Average running speed of nonweaving vehicles in the weaving area in mph

Source: (15)

The measure of effectiveness for weaving areas (Table 8) is the average travel speed. The equations used in weave analysis determine the average speed for weaving and nonweaving vehicles under constrained and unconstrained conditions. The general equation format is given below and the constants used under constrained and unconstrained conditions are given in Table 17 for each type of weave:

$$Sw \text{ or } Snw = 15 + \frac{50}{1 + a\,(1 + VR)^b\,(v/N)^c\,/\,L^d}$$

To determine if the weave area is operating constrained or unconstrained, the number of lanes required for weaving vehicles to operate unconstrained must be determined and compared with the maximum number of weaving lanes that may be used by weaving vehicles. The equations for determining the number of lanes needed and the maximum number of lanes available are given in Table 18. For cases where $N_w \leq N_w$ (max), the weaving area will be unconstrained. Where $N_w > N_w$ (max) the operation of the weaving area will be constrained.

Table 17
Constants for Estimating Speeds for Weaving Areas

Type of Weave Configuration	Constants for Weaving Speed S_w				Constants for Nonweaving Speeds S_{nw}			
	a	b	c	d	a	b	c	d
Type A								
Unconstrained	0.226	2.2	1.0	0.90	0.020	4.0	1.30	1.00
Constrained	0.280	2.2	1.0	0.90	0.020	4.0	0.88	0.60
Type B								
Unconstrained	0.100	1.2	0.77	0.50	0.020	2.0	1.42	0.95
Constrained	0.160	1.2	0.77	0.50	0.150	2.0	1.30	0.90
Type C								
Unconstrained	0.100	1.8	0.80	0.50	0.015	1.8	1.10	0.50
Constrained	0.100	2.0	0.85	0.50	0.013	1.6	1.00	.050

Source: (15)

Table 18
Criteria for Determining the Operating Conditions of a Weave Area[1]

Type of Weave Configuration	Number of Lanes Required for Unconstrained Operation, N_w	Maximum Number of Weaving Lanes, N_w (max)
Type A	$2.19\,N\,VR^{0.571}\,L_H^{0.234}\,/\,S_w^{0.438}$	1.4
Type B	$N\,\{0.085 + 0.703\,VR + (234.8/L) - 0.018(S_{nw}\text{-}S_w)\}$	3.5
Type C	$N\,\{0.761 - 0.011L_H - 0.005(S_{nw}\text{-}S_w) + 0.047\,VR\}$	3.0[2]

[1] All variables are as defined in Table 16
[2] For 2-sided weaving areas, all freeway lanes may be used as weaving lanes

Source: (15)

The design analysis of weave areas, like that of ramps, is performed for a certain "trial" design to predict the operation with the design hour volumes. Thus, all the roadway factors, such as number of lanes, lane configuration, weave area length, lane widths and terrain, and traffic factors such as directional design hour volumes, PHF, the percentage of trucks and driver population factor, must be known.

The first step is to convert the traffic volumes to peak-hour flows in pcph for ideal conditions using the PHF and adjustment factors for heavy vehicles, driver population, and lane widths. Using these volumes, the unconstrained weaving and nonweaving vehicle speeds are determined using the equation and constants from Table 17. Based on these speeds, the equations from Table 18 are used to estimate the number of lanes needed by weaving vehicles to obtain unconstrained operation and are compared to the maximum number of lanes that may be used by weaving vehicles (also in Table 18). If the computed number of lanes is less than or equal to the maximum number of lanes that can be used, then the speeds computed for unconstrained operation are considered accurate. If, however, the number of lanes needed is greater than the maximum lanes used, the speeds should be recalculated using the values for constrained operation from Table 17.

Once the average speeds for weaving and nonweaving vehicles under constrained or unconstrained conditions have been determined, the weave area should be checked against the limitations presented in Table 19. These values generally represent the maximum values for weaving flow rates, total flow rate per lane, volume ratios, and weaving ratios which are within the range of the methodology used. When the weaving capacity is exceeded, breakdown in the operation is likely. Where the volume ratio and weaving ratio are exceeded, breakdown may not occur; but the speeds occurring would likely be lower than those computed by the equations presented. Where the length limitations are exceeded, the area should be analyzed as separate merge and diverge areas.

Finally, the speeds should be compared to the LOS criteria in Table 20 to determine the prevailing LOS for the design. If the LOS indicates that the design will not operate at an acceptable LOS, then changes to the design must be considered and additional analyses performed. Options to improve the design might include increasing the weave length, changing the weave type, braiding the ramps, or removing a ramp to eliminate the weave section.

Table 19
Limitations on Weaving Area Analysis

Type of Configuration	Weaving Capacity Maximum V_w	Maximum v/N	Maximum Volume Ratio VR		Maximum Weaving Ratio R	Maximum Weaving Length L
Type A	1800 pcph	1900 pcphpl	N	VR	0.50	2000 ft.
			2	1.00		
			3	0.45		
			4	0.35		
			5	0.22		
Type B	3000 pcph	1900 pcphpl	0.80		0.50	2500 ft.
Type C	3000 pcph	1900 pcphpl	0.50		0.40	2500 ft.

Note: Type C limitations do not apply to two-sided weaves.

Source: (15)

Table 20
Level of Service Criteria for Weaving Sections

Level of Service	Minimum Average Weaving Speed, S_w (mph)	Minimum Average Nonweaving Speed, S_{nw} (mph)
A	55	60
B	50	54
C	45	48
D	40	42
E	35/30[1]	35/35[1]
F	< 35/30[1]	< 35/30[1]

[1]The 35 mph boundary for LOS E/F is used when comparing to computed speeds using the equations of Table 18. The 30 mph boundary is used for comparison to field-measured speeds.

Source: (15)

Multi-Lane Noncontrolled Access Facilities

Multi-lane highways differ from freeways in that they may not be divided and/or they may not have full control of access. Traffic on these facilities operates unimpaired between points of interruption such as intersections (signalized or unsignalized) or driveway access. The geometric design of noncontrolled access multi-lane highways, like controlled access facilities, is based on defined standards for horizontal and vertical geometry and safety as outlined in the **Operations and Procedures Manual**. The design elements for noncontrolled access multi-lane highways that are based on traffic volume include the facility size and certain elements in the design of access/egress and intersections.

Table 22 gives the design year ADT and DDHV ranges used to determine the facility size (4-lane divided or undivided or 6-lane). The selection of the facility size then determines the design standards for the median width, inside shoulder, and bridge width. The other design standards listed do not vary by size or volume.

Traffic volumes are also used in designing intersections and median deceleration and storage lanes serving left-turning traffic on divided highways. The design criteria for deceleration lanes as provided in the **Operations and Procedures Manual** is given in Table 22.

Table 21
Standards of Design for Multi-Lane Rural Highways
(Noncontrolled Access, All Functional Classes)

HIGHWAY CLASS		CLASS 6L		CLASS 4L		CLASS 4L (undivided)[1]	
Average Daily Traffic (ADT)[2]		20,000 or more		5,000 to 20,000		Up to 7,500	
Design Hourly Volume (DHV)[3]		1,600 to 2,400		400 to 1,600		Up to 600	
Design Speed (Arterials)[4]		Des.	Min.	Des.	Min.	Des.	Min.
Flat		70	70[5]	70	70[5]	70	70[5]
Rolling		70	60[6]	70	60[6]	70	60[6]
Lane Width (Ft.)		12					
Median Width (Ft.)	Narrow (Surfaced)	16	4	16	4	0	
	Depressed	76	48	76	48		
Shoulder Outside (Ft.)		10	8[7]	10	8[7]	10	8[7]
Shoulder Inside (Ft.)		10	2[8] 4[9]	4	2[8]	Not Applicable	
Bridge Width (Ft.)	Narrow Med.	108	92	84	68	68	64[10]
	Depressed Med.	50	42[11]	38	30[11]		

Notes:

1 Undivided section may be used on betterment projects of 2-lane highways to improve passing opportunities. Most appropriate for rolling terrain and/or restricted right-of-way conditions.

2 ADT at design year (equivalent passenger vehicles per day, flat terrain, ideal conditions, 20 years from date of construction).

3 One-way DHV (equivalent passenger vehicles per hour, flat terrain, ideal conditions, 20 years from date of construction).

4 For multi-lane collectors, minimum design speed values are 10 mph less than tabulated.

5 55 mph acceptable for heavy betterment under unusual circumstances. Otherwise, 70 mph should be minimum for rural design.

6 50 mph acceptable for heavy betterment under unusual circumstances. Otherwise, 60 mph should be minimum for rural design.

7 Applies to collector roads only. On Class 4L undivided highways, outside surfaces, shoulder width may be decreased to 4 feet where flat (10:1), sodded front slopes are provided for a minimum distance of 4 feet from the shoulder edge.

8 Applicable only to 4-foot flush median.

9 Minimum 4-foot surfaced for depressed medians.

10 Bridge width of 56 feet may be retained; all new or widened bridges to be width of approach roadway including shoulders.

11 Pertains only to existing bridges to be retained. All new or widened bridges to be width of approach roadway including shoulders.

Source: (16)

Table 22
Lengths of Median Speed Change Lanes
Multi-Lane Rural Highways

Turning ADT, vpd Minimum Storage length, feet	150 50	300 100	500 175	750 250
Design Speed Taper length, feet	50 180	60 245	70 320	

Note: **For low volume median openings, such as those serving private drives or U-turns, taper length of 180 feet may be used regardless of mainlane design speed.**

Source: (16)

Design year average daily traffic volumes are used to determine the minimum storage length. for the deceleration lanes. The use of traffic volumes in the design of intersections on multi-lane highways is discussed under urban arterial streets.

In some cases the number of lanes required on a multi-lane, noncontrolled access facility may be determined using the design analysis procedure from the **Highway Capacity Manual**. This procedure is the same as that used for basic freeway segments on controlled access facilities except that one additional factor for the type of surrounding development (rural or suburban) is used in the equation to adjust the maximum service flow rate. Thus, the equation used to determine the number of lanes required on multi-lane, noncontrolled access highways is:

$$N = SF/(MSF \times f_W \times f_{HV} \times f_P \times f_E)$$

Where:

SF = service flow rate determined by dividing forecast DDHV by PHF

MSF = maximum service flow for selected level of service or v/c ratio

f_W = factor to adjust for effects of restricted lane widths and/or lateral clearances

f_{HV} = factor to adjust for heavy vehicles

f_P = factor to adjust for effect of driver population

f_E = factor to adjust for development environment and highway type

As described, the traffic forecast data needed in the design of multi-lane, noncontrolled access highways are identical to those needed for controlled access facilities including:

- Design year average daily traffic for each link on the facility and for major cross streets,
- Directional design hour volumes for each link on the facility and for major cross streets,
- Design year percentage of heavy vehicles by type,
- Design year PHF, and
- Design year average daily turning movements and design hour turning movements for all major intersections.

Table 23 gives the LOS criteria for multi-lane, noncontrolled access highways for design speeds of 70 mph, 60 mph, and 50 mph. As with the other LOS criteria, the maximum service flow rates represent the maximum volume in pcphpl which can be accommodated under ideal conditions while maintaining the specific LOS.

Urban Arterial Street

The design of urban streets is based on standard geometric design criteria as prescribed in the **Highway Operations and Procedures Manual** (16). Most of the basic design criteria are not dependent on traffic volumes or functional class. The design of signalized intersections is the only element of urban streets that is volume dependent.

The design of signalized intersections involves consideration of traffic, roadway, and signalization conditions. TxDOT utilizes the critical lane analysis technique to determine if a proposed intersection design will provide an acceptable LOS in the design year. Thus, the design of signalized intersections is a trial-and-error process in that an initial design and signal plan must be assumed in order to determine if the intersection will provide satisfactory service.

Table 23
Level of Service Criteria for Multi-Lane Highways

LOS	Density (pc/mi/ln)	70 mph Design Speed			60 mph Design Speed			50 mph Design Speed		
		Speed[a]	v/c	MSF[b]	Speed[a]	v/c	MSF[b]	Speed[a]	v/c	MSF[b]
A	≤ 12	≥ 57	0.36	700	≥ 50	0.33	650	---	---	---
B	≤ 20	≥ 53	0.54	1100	≥ 48	0.50	1000	≥ 42	0.45	850
C	≤ 30	≥ 50	0.71	1400	≥ 44	0.65	1300	≥ 39	0.60	1150
D	≤ 42	≥ 40	0.87	1750	≥ 40	0.80	1600	≥ 35	.0.76	1450
E	≤ 67	≥ 30	1.00	2000	≥ 30	1.00	2000	≥ 28	1.00	1900
F	> 67	< 30	[c]	[c]	< 30	[c]	[c]	< 28	[c]	[c]

Source (15)

The traffic forecast data required for testing the design of signalized intersections include:

- DDHVs for each movement (thru traffic, right and left turns) on each approach. The DDHVs should be in pce; thus, the adjustments for trucks and/or other heavy vehicles should be made to the volumes; and

- Design year peaking factor.

The peaking factor is similar to the PHF used in the design analysis procedures previously described in that it is used to adjust the design hour volume to the peak 15-minute flow rate within the hour. The peaking factors used by TxDOT are given in Table 24 for the design year population of the city. Peaking factors are applied to the DDHVs in the following manner:

$$PPV = DDHV \times PF$$

Where:

PPV = the 15-minute peak-period volume or flow rate

DDHV = the directional design hour volume

PF = the peaking factor from Table 24

Table 24
Peaking Factors for Design Year City Population

Design Year Population	Peaking Factor
< 100,000	1.35
100,000 - 300,000	1.30
300,000 - 500,000	1.25
> 500,000	1.18

Source: (16)

Once the DDHVs have been adjusted to reflect the peak-flow rate, an intersection design and signal phasing scheme must be selected for the initial analysis. Because this process is iterative the intersection design most likely to accommodate the design year traffic should be used. Using the selected intersection design and signal phasing, the peak-period traffic flow volumes are combined to reflect the traffic movement volumes (MV_m). For example, the thru and right-turn volumes would be combined unless a free right-turn lane is provided in the design. If a left-turn bay is not provided, then the left-turn volumes would be added to the thru-right movement.

The next step in the analysis is to adjust all separate movement volumes to reflect design and operational features of the intersection with the following equation:

$$Q_m = U \times W \times TF \times MV_m$$

Where:

Q_m	=	adjusted volume for movement(s) $_m$, vph
U	=	lane utilization factor
W	=	lane width factor
TF	=	turning movements factor
MV_m	=	volume of appropriate traffic movement(s), vph

As the number of lanes provided on an approach increases, there is a tendency for one lane to become more highly utilized than the others. The lane utilization factor, shown in Table 25, adjusts for this effect. The lane width factor, also shown in Table 25, adjusts for the effect of lane widths under 10 feet. .

Table 25
Adjustment Factors for Lane Utilization and Lane Width

Factor	Conditions	Value
Lane Utilization, U	Number of Lanes	
	1	1.0
	2	1.1
	3	1.2
Lane Width, W	Average Lane Width, Feet	
	9.0 - 9.9	1.1
	$\geq$3.05 ($\geq$ 10.0)	1.0

Source: (16)

The turning movements factor adjusts for the effect of left- and right-turning vehicles on traffic flow and is determined by the following equation:

$$TF = 1.0 + L + R$$

Where:

TF	=	turning movements factor
L	=	left-turn movement adjustment factor
R	=	right-turn movement adjustment factor

The equation used to determine the left-turn movement factor depends on whether or not a left-turn bay is provided. Where a left-turn bay is not provided, the adjustment for left-turning vehicles (L) is determined using:

$$L = P_L\ (E - 1.0)$$

Where:

P_L	=	decimal fraction of the total approach volume turning left
E	=	appropriate equivalence factor from Table 26

Where a left-turn bay is provided, the left-turn adjustment factor (L) is determined using:

$$L = \frac{1700 \ x \ E}{S} - 1.0$$

Where:

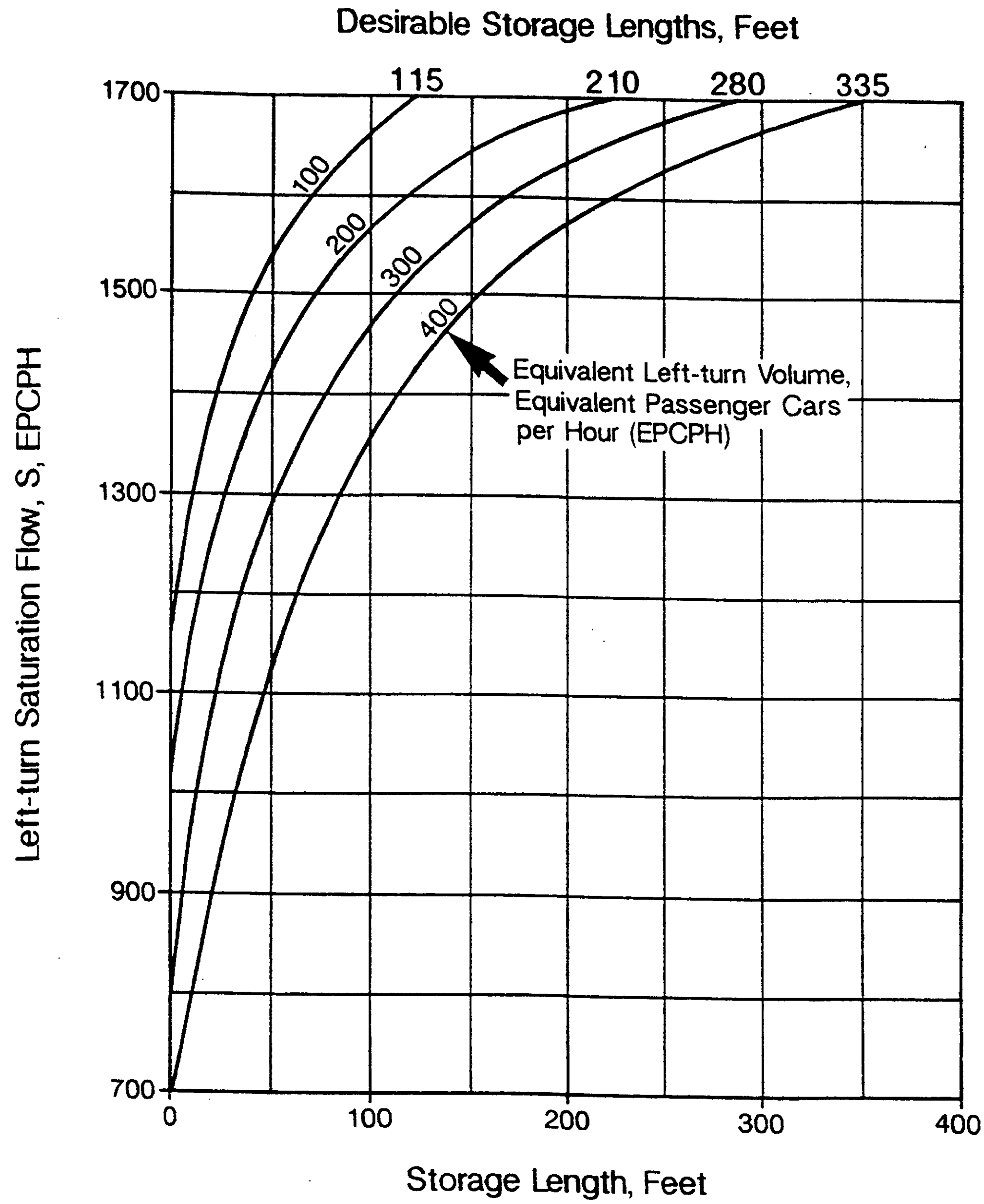
L	=	left-turn movement adjustment factor
S	=	saturation flow of the left-turn bay determined from a figure in the **Operations and Procedures Manual**; based on the volume of left-turning vehicles and the storage bay length
E	=	appropriate equivalence factor from Table 26

If the left-turn storage bay length is not long enough to result in a saturation flow rate of 1,700 vph, then the thru-right movement must be adjusted for the effect of blockage due to inadequate left-turn storage. The equation used to adjust the thru-right movement is:

$$\frac{L = 1700 - S}{1700 \ (N - 1) + S}$$

Where:

S	=	saturation flow of the left-turn bay determined from Figure 12
N	=	number of lanes serving the thru-right movement



Source: (16)

Figure 12. **Saturation Flow of Left-Turn Phase as a Function of Bay Storage Length and Turning Volume**

Table 26
Factors for Left-Turn Equivalents, E

Intersection Signal Phasing	Traffic Movement	Number of Opposing Lanes	Opposing Volume				
			200	400	600	800	1000
Unprotected Turning							
Two Phase • No Bay	Left & Thru	1	2.0	3.3	6.5	**	**
	Left & Thru	2	1.9	2.6	3.6	6.0	**
	Left & Thru	3	1.8	2.5	3.4	4.5	6.0
• With Bay	Left	1	1.7	2.6	4.7	**	**
	Left	2	1.6	2.2	2.9	4.1	6.2
	Left	3	1.6	2.1	2.8	3.6	2.8
Three or More Phases • No Bay	Left & Thru	1	2.2	4.5	**	**	**
	Left & Thru	2	2.0	3.1	4.7	**	**
	Left & Thru	3	2.0	2.9	4.2	6.0	**
• With Bay	Left	1	1.8	3.3	**	**	**
	Left	2	1.7	2.4	3.6	5.9	**
	Left	3	1.7	2.4	3.3	4.6	6.8
Protected Turning							
• No Bay	Left	Any	1.2	1.2	1.2	1.2	1.2
• With Bay	Left	Any	1.03	1.03	1.03	1.03	1.03

Source: (16)

When free right-turn lanes are provided, the right-turning vehicles are subtracted from the approach volume; and no right-turn adjustment factor is needed. For intersections where no free right-turn lane is provided, a right-turn adjustment may be developed when a detailed analysis of the intersection is required. For these cases, the right-turn adjustment factor is determined using two equations. First, the equation below is used to adjust the thru-right movement as follows:

$$R = \frac{5 \times P_r}{c}$$

Where:

R	=	the right-turn adjustment factor
P_r	=	the decimal fraction of movement combination turning right
c	=	the curb return radius, in feet

Next, the number of vehicles turning right on red is estimated and subtracted from the adjusted thru-right movement volume. The right-on-red vehicles are estimated using the following equation:

$$ROR = 50 \left(\frac{P_e}{1 - P_e} \right)$$

Where:

ROR	=	right-on-red vehicles
P_e	=	estimated decimal fraction of the traffic in the curb lane turning right

In most cases, these right turn adjustments cancel each other out. Thus, the right-turn adjustment factor (R) in the equation used to adjust the volumes for right and left turns can usually be set to zero where right turns on red are allowed.

After the adjustments have been made to the movement volumes for each approach, the critical lane analysis procedure is used to determine whether the proposed design is acceptable. First, the adjusted movement volumes (Q_m) are divided by the number of lanes available for that movement to obtain a design volume per lane. Then, the critical lane volumes for each street are determined by adding together design volumes per lane according to the phasing scheme initially selected. The critical lane volumes for each street are then added together to determine the critical lane volume to be compared with the selected LOS volume for critical lane analysis given in Table 27.

Table 27
Level of Service Maximum Sum of Critical Lane Volumes at Signalized Intersections

Level of Service	Traffic Flow Condition	Volume to Capacity Ratio	Maximum Sum of Critical Lane Volumes, Sum of Volume for Intersection[1]		
			Two Phase[2]	Three Phase[3]	Multi-Phase[4]
A	Stable	≤ 0.6	900	855	825
B	Stable	≤ 0.7	1050	1000	965
C	Stable	≤ 0.8	1200	1140	1100
D	Unstable	≤ 0.85	1275	1200	1175
E	Capacity	≤ 1.0	1500	1425	1375

Notes:
For typical 4-leg intersection:
[1] Sum of critical lane volume for both streets.
[2] One-phase operation on both streets.
[3] Two-phase operation on one of the street and one-phase operation on the other street.
[4] Two- or multi-phase operation on both streets or multi-phase operation on one street and one-phase operation on the other street.
Source: (16)

Two-Lane Rural Highways

Two-lane rural highways when classified as arterials are designed to provide mobility by connecting major links in the state and interstate highway system. Low volume 2-lane rural highways may be classified as collectors or local roads which serve primarily to provide access for various activities. The functional class and traffic volume of 2-lane rural highways will determine the appropriate design speed for the facility (Table 28). The design speed in combination with current and/or design year traffic volumes then determines minimum design criteria such as the width of travel lanes and shoulders as shown in Table 29. Other design standards for 2-lane rural highways such as horizontal and vertical alignments, lateral clearances, cross slopes, and others are based on defined engineering standards governed by design speed, sight distance, and safety. Current traffic also determines the minimum width for bridges that will remain in place on rural highways. Table 30 gives the roadway clear widths required for rural structures to remain in place for the determined volume ranges.

As described by the tables above, the traffic forecast data needed for the geometric design of 2-lane rural highways are less detailed than what are needed for the other roadway types. Typically, the forecast data needed include:

- Current or base year average daily traffic and
- Design year average daily traffic.

Table 28
Minimum Design Speed[1] Related to Functional Class Terrain and Traffic Volume for Rural 2-Lane Highways

Functional Class	Terrain	MIN. DESIGN SPEED (mph) FOR TRAFFIC VOLUME OF			
		Current 0-250 ADT	Current 250-400 ADT	Future 750-1500 ADT	Future 1500 + ADT
Arterial	Level	70			
	Rolling	60			
Collector	Level	50[2]	50[2]	50	60
	Rolling	40[3]	40[3]	40	50
Local[4]	Level	30	40	50	50
	Rolling	30	30	40	40

Notes:
[1] Applicable to projects on new location or at regrading of existing highways.
[2] A 40 mph minimum design speed may be used where roadside environment or unusual design considerations dictate.
[3] A 30 mph minimum design speed may be used where roadside environment or unusual design considerations dictate.
[4] Applicable only to off-system routes that are not functionally classified at a higher classification.

Source: (16)

Table 29
Width of Travel Lanes and Shoulders on Rural 2-Lane Highways

Functional Class	Design Speed (mph)	Lane/Shoulder	MIN. WIDTH[1,2] (ft.) FOR TRAFFIC VOLUME OF: Current 0-250 ADT[3]	Current 250-400 ADT[3]	Future 750-1500 ADT	Future 1500-3000 ADT	Future 3000 + ADT
Arterial	All	LANES	12				
Bridges	All	SHOULDERS	4[4] 34[8]	4[4] 34[8]	6[4] 38[8]	8-10[4] 40-44	10[4] 44
Collector	30	LANES	10	10	10	11	12
	40		10	10	11	11	12
	50		10	10	11	12	12
	60		11	11	11	12	12
Bridges	All	SHOULDERS	2[5,6] 28-30[5]	2[5,6] 28-30[5]	4[6] 28-30[5]	8-10[6] 38-44[8]	8-10[6] 40-44
Local[7]	30	LANES	10	10	10	10	12
	40		10	10	11	11	12
	50		10	10	11	11	12
Bridges	All	SHOULDERS	2 24	4 28	4 28-30	8 36-38[8]	8 40

[1] Minimum surfacing width is 22' for all highway system routes.
[2] On high riprapped fills thru reservoirs, a minimum of two 12' lanes with 8' shoulders should be provided for both roadway sections and bridges. For arterials with 3,000 or more future ADT in reservoir areas, two 12' lanes with 10' shoulders should be used.
[3] Future ADT should be less than 750.
[4] On arterials, shoulders should be fully surfaced.
[5] On collectors, use minimum 4' shoulder width at locations where roadside barrier is utilized.
[6] For collectors, shoulders fully surfaced for future 3,000 or more ADT. Shoulder surfacing not required but desirable even if partial width for collectors with lower volumes and all local roads.
[7] Applicable only to off-system routes that are not functionally classified at a higher classification.
[8] To maximize use of currently available bridge standard details, use a 34' bridge for a 32' approach roadway, a 38' bridge for a 36' approach roadway, and a 44' bridge for a 42' approach roadway; and widen crown on bridge approaches to accommodate guard fence.

Notes:
1. The minimum width of new or widened structures shall accommodate the approach roadway including shoulders.
2. See Table 26 for minimum structure widths that may remain in place.

Source: (16)

Table 30
Minimum Structure Widths for Bridges to Remain in Place on Rural 2-Lane Highways

Current ADT (vpd)	Roadway Clear Width[1] (ft.) Locals & Collectors	Roadway Clear Width[1] (ft.) Arterials
Under 400	22	
400 to 4,000	24	Traveled Way Plus 6'
Over 4,000	30	

[1]Clear width between curbs or rails, whichever is less, is considered to be at least the same as approach roadway clear width.

Source: (16)

Forecast Accuracy and Roadway Design

The design engineer must use the forecast data and design analysis procedures to determine the most efficient and cost-effective design for each facility. The forecast DDHV and the design/operational analysis results form the basis for justifying the multitude of design decisions made with regard to the number of lanes, ramp/freeway terminal design, and weave area design. And, it is the forecast volumes and the design/operational analysis results that are used to obtain federal approval, if required, for the facility design.

The DDHVs used for the schematic design of urban freeways should be sufficiently accurate to prevent the over- or underdesign of a facility by even one lane and to avoid merge/diverge and/or weaving areas that result in a breakdown of operations on the facility. The design analysis procedures are only as accurate as the data used as input. As discussed under project planning, the use of K, D, and T factors to develop DDHV, in conjunction with the variation in design service flows that may be selected, has the potential to result in the over- or underdesign of a facility by more than 50 percent if care is not taken in estimating these factors for the design year. If the forecast data are accurate to within ± 10 to 15 percent, the design analysis results will be accurate to ± 10 to 15 percent. When performing the design analysis for the various freeway segments, at some point it takes only one additional vehicle (or passenger car equivalent) to cause the results to indicate the need for more or less capacity. Although the addition of one vehicle or passenger car equivalent can change the results of the design analysis to a poorer or better LOS, it is not reasonable to assume that the design analysis procedures or the forecast volumes are accurate to that level. Because the service flow volumes provided by the **Highway Capacity Manual** for the various analyses have been rounded to the nearest 50 vehicles, the results of any design analysis should not be considered to be more accurate than ±50 vehicles.

Of the traffic factors used in the design analysis procedures, only the DDHV and percentage of heavy vehicles are forecast by the Transportation Planning and Programming Division. The other factors (driver population, PHF, and selected design service volumes) are determined by the local district office or by the Design Division. However, these factors affect the ultimate design of the facility. The PHF is used to adjust the DDHVs to reflect the peak rate of flow and, depending on the value used, serve to increase the DDHV by 10 to 30 percent. The driver population factor is used to adjust the MSF rates for the effects of driver population on a facility's capacity. The selection of different service flow rates (from the maximum for LOS C to the maximum for E) can raise or lower the capacity needs by 14 percent. Thus, given the many factors used in design analysis, providing project-specific traffic factors, as well as accurate traffic forecast data, is crucial in sizing and designing the facility.

The effect of changes in the DDHV forecast on determining the number of lanes needed is linear. That is, ± 10 percent in the DDHV will result in a need for 10 percent more or less capacity. Whether or not a change in the DDHV forecast will produce a different result in the design analysis depends on the initial forecast volume relative to the selected design LOS as well as the analyst's judgment as to whether the design analysis results indicate satisfactory operation on the facility. For example, if the initial DDHV is 20 percent below the capacity of the facility at the selected LOS, a 15 percent increase in the volume will probably not result in the need for more capacity; whereas, if the initial DDHV is only 1 or 2 percent below the capacity, any increase in the volume may result in a change in the design.

The effect of changes in the forecast of the percentage of trucks is similar to changes in the overall volumes; that is, a 2 percent increase or decrease in the percentage of trucks will result in a 2 percent increase or decrease in the DDHV and, thus, a corresponding change in the need for more or less capacity.

The ramp/freeway terminal and weaving area design analyses are more sensitive to changes in the DDHV because the maximum service flows that can be accommodated in these type operations are less than for mainlane freeway operations. Whether a change in the DDHV will impact the design depends on the many conditions discussed for each type of operation.

Pavement Design

The traffic forecast components, techniques, and the need for forecast accuracy in pavement design are presented in a separate report, **Traffic Load Forecasting for Pavement Design, Research Report 1235-1** (18), prepared for this project. The results of that report relative to forecasting requirements for pavement design are summarized in this section.

A primary component of pavement design is forecasting traffic loading that will be applied to the pavement during its design life (usually 20 to 30 years). Traffic loading for pavement design is determined by converting the forecast traffic volume into a load equivalence factor. This load equivalence factor is defined as the ratio of the number of 18,000-pound single-axle repetitions which are required to cause a given level of pavement damage to the number of repetitions of an axle of some other weight and/or configuration which are required to cause the same given amount of damage. The load equivalency factor is a function of variables such as axle weight, axle configuration, pavement type, pavement thickness, tire contact area, tire contact pressure, environmental conditions, and soil support. In traffic forecasting, however, the load equivalence factor for each specific axle weight and configuration, pavement type and thickness, tire contact area, and other factors is taken as a set value.

The goal in traffic forecasting for pavement design is to develop the best estimate of the number of axles of each configuration in each weight category for the project over the design period. The load equivalence factor for each axle configuration and weight category is used to convert the axle weights to equivalent single axle loads (ESALs). The ESALs for each axle configuration and weight category are then added together to determine a total ESAL forecast for use in pavement design.

The forecast data needed for pavement design include:

- Average daily traffic growth rate for the design year;
- Base year ADT;
- Percentage trucks, including dual-rear-tire pickups and buses, for each classification category;
- Directional distribution for the design period; and
- Lane distribution factor for the design period.

Pavement life is influenced by numerous factors including the traffic loadings and environmental conditions that occur during the design period and the properties of the materials used to construct the pavement. Thus, the uncertainty regarding pavement life can be attributed to two groups of factors, traffic factors and pavement factors.

The effects of variance between the forecasts for the traffic components used in pavement design and the actual traffic that occurs are well documented. Forecasts which cause the underestimation of the design period ESALs can cause pavement to fail years earlier than planned, resulting in significant opportunity costs. Likewise, forecasts that are too high can result in major opportunity costs due to the increased cost of unnecessary pavement.

The relationship between the design ESAL and the traffic volumes is linear. That is, if the total traffic volume forecast increases by 20 percent proportionately in every vehicle class, then the design ESAL would increase by 20 percent.

The traffic component having the greatest effect on pavement design is the truck vehicle type and weight classification data. Numerous studies have shown that there can be a large variation in the total volume of trucks and the volume of trucks by type and weight on a given facility between seasons, days, and time of day and that there can also be major variations between sites on the same facility. The truck classification and weight data used in most projects, however, are not project-specific. Typically, these data are collected on similar highways in the same general geographic area. Research has shown that the assumption that truck classification and weights are similar for like highways and areas can introduce errors in magnitude of 3 or 4 into the ESAL estimates. Additionally, research has shown that variation in the traffic factors of pavement design is largely due to variation in the equivalency factors which are taken as given.

Current site-specific truck classification and weight data should be collected for each project. Further research has shown that traffic load forecast accuracy could be improved by 30 percent from current levels by conducting 24-hour manual vehicle classification counts at specific pavement project types and improved by more than 85 percent by conducting week-long weigh-in-motion sessions at specific project sites.

BRIDGE PROJECT FORECASTING REQUIREMENTS

TxDOT's Design Division provides guidance and assistance to the local district offices in the preliminary planning and design of all types of structures. Elements of assistance provided may include preliminary studies of structure types, estimates and economic comparisons, preliminary field inspections and location studies, and structure layout and presentation of preliminary data. Guidance on bridge design is based on the current **Bridge Division Operation and Planning Manual** (19) and on current standards and interim specifications for highway bridges published by the American Association of State Highway and Transportation Officials (AASHTO) (20).

Generally, project planning and design for bridges is performed in conjunction with the project planning and design of the approach roadway. Thus, the traffic forecast requirements and level of accuracy for bridge project planning and design are generally the same as those for the different types of roadways previously discussed.

Except for the lowest volume roads, bridge widths conform to the roadway widths as determined by traffic volumes in the **Highway Design Division Operations and Procedures Manual.** The specific roadway and structure widths for the different roadway types for the forecasted design year ADT (and/or current ADT) are listed in Table 31 (19). As indicated, the standard roadway and structure widths for controlled access freeway mainlane sections are not varied according to current or design year volumes. That is, if design year volumes for a controlled access freeway require six lanes to be built, the 6-lane cross-section shown for the desired median type will be used and will not be varied according to the current or forecast traffic volumes. In all cases, the structure width for controlled access facility mainlanes matches that of the approach roadway. This is also true of ramps and direct connections except that at-grade roadway ramps without traffic barriers require a 2-foot inside shoulder, and ramps and direct connections on structures require a 4-foot minimum inside shoulder.

The minimum roadway and bridge design width of the various types of multi-lane, noncontrolled access facilities, rural frontage roads, 2-lane rural highways, and F.M. and R.M. roads is, however, determined by the design year and/or current year ADT. Depending on the road type, the design year volume (and, in some cases, the current volume) determines the number of lanes, the lane widths, the shoulder widths and the type of cross-section (undivided or divided).

- For multi-lane noncontrolled access facilities, bridge widths match the approach roadway including the usable shoulder. The roadway widths are determined by design year ADT volumes and the type of cross-section.
- For rural frontage roads, the lane width and shoulder width are determined by design year ADT volumes. Bridges match the roadway width.
- Bridge widths match the approach roadway widths on F.M. and R.M. roads. The design year ADT determines these widths.
- For 2-lane rural highways other than F.M. and R.M. roads, the roadway and bridge widths are based on design year and current ADT. Bridge widths match the approach roadway except on low volume roads where the design speed is over 50 mph. In this case the bridge width is 4 feet wider than the roadway.
- Bridges on urban arterial streets match the curb and gutter dimensions with a 4- to 6-foot sidewalk provided on each side.
- Off-system bridge rehabilitation or replacement should conform to state design standards and the Secondary Road Plan when applicable.

Discussions with TxDOT staff revealed that traffic volumes and truck volumes are no longer considered in determining the design load of structures. Currently, TxDOT designs all on-system structures and the majority of off-system structures for an HS20 load. Some low volume off-system structures, however, may be designed at H20 loads. As a result, traffic forecasts do not affect the design of bridges except with regard to geometrics as discussed above.

Table 31
Roadway and Bridge Widths Based on Traffic Volumes

Facility Type	Design Year ADT	Roadway Width	Structure Width
Multi-lane Controlled	N/A		
Access Freeway			
Mainlanes			
4-Lane Divided	N/A	4+24+10 = 38	38
6-Lane Divided (deprssd. med.)	4+36+10 = 50	50	
6-Lane Divided (flush median)		10+36+10 =56	56
8-Lane Divided		10+48+10 = 68	68
1-Lane Direct Connection		2*+14+8 = 24	4+14+8 = 26
2-Lane Direct Connection		2*+24+8 = 34	4+24+8 = 36
Ramps other than Dir. Conn.		2*+14+6 = 22	4+14+6 = 24
		(*4 ft. min. to traffic barrier)	
Rural Frontage Roads	Less than 400	4+20+4 = 28	28
	400 to 750	6+22+6 = 34	34
	More than 750	8+24+8 = 40	40
Multi-lane Noncontrolled Access			
Undivided	7,500 or less (4L)	10+48+10 = 68 des.	68
		8+48+8 = 64 min.	64
Narrow Median (4 ft.)	5,000 to 20,000	10+24+4+24+10 = 72 des.	72
		8+24+4+24+8 = 68 min.	68
Narrow Median (16 ft.)	5,000 to 20,000 (4L)	10+24+16+24+10 = 84 des	84
		8+24+16+24+8 = 80 min.	80
Depressed Median	5,000 to 20,000 (4L)	4+24+10 = 38 des.	38
		4+24+8 = 36 min.	36
Narrow Median (4 ft.)	20,000 or more (6L)	10+36+4+36+10 = 96 des.	96
		8+36+4+36+8 = 92 min.	92

Facility Type	Design Year ADT	Roadway Width	Structure Width
Narrow Median (16 ft.)	20,000 or more (6L)	10+36+16+36+10 = 108 d	108
		8+36+16+36+8 = 104 min.	104
Depressed Median	20,000 or more (6L)	4+36+10 = 50 des.	50
		4+36+8 = 48 min.	48
2-Lane Rural			
Low Volume	Current ADT ≤ 400	4+20+4 =28	28[1]
Less than 2200 Design		4+22+4 = 30	34[2]
Year ADT and:	Current ADT 400-750	6+22+6 = 34	34
	Current ADT ≥ 750	8+24+8 = 40	40
	Current ADT > 750	10+24+10 = 44 des.	44
		8+24+8 = 40 min.	40
High/Moderate Volume	2,200 to 7,500	10+24+10 = 44 des.	44
		8+24+8 = 40 min.	40
F.M. or R.M. Roads			
	0 to 50[3]	3+18+3 = 24	24
	400 or less	4+20+4 = 28	28
	400 to 750	6+22+6 = 34	34
	750 or more	8+24+8 = 40	40

Notes:
[1] Applicable when design speed is 50 mph or less.
[2] Applicable when design speed is over 50 mph.
[3] Applicable only for projects involving 100 percent state funds.

Source: (19)

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