Analysis Procedures Manual

April 2006

Oregon Department of Transportation
Transportation Development Division
Planning Section
Transportation Planning Analysis Unit
Salem, Oregon

ACKNOWLEDGEMENTS

The following individuals were contributors in the preparation of this manual.

Oregon Department of Transportation

Kent Belleque, P.E.
Alexander Bettinardi, P.E.
Rod Cathcart
Don Crownover, P.E.
Brian Dunn, P.E.
Simon Eng, P.E.
Christina Fera-Thomas
Meghan Hamilton
Mark Johnson, P.E.
Chi Mai, P.E.
Christina McDaniel-Wilson
Joseph Meek III, P.E.
Nancy Murphy
Thanh Nguyen, P.E.
Douglas Norval, P.E.
Robert Nova
Gary Obery, P.E.
Peter Schuytema, P.E.
Dorothy Upton, P.E.

DKS Associates, Inc.

John Bosket, P.E.
Carl Springer, P.E.
Bob Schulte

CH2M Hill

Craig Grandstrom, P.E.

 This project was funded in part by the Federal Highway Administration, U.S. Department of Transportation.

Table of Contents

Table of Exhibits

Table of Examples

1 INTRODUCTION

1.1 Overview of Manual Purpose

The Analysis Procedures Manual (APM) was created to provide a comprehensive source of information regarding current methodologies, practices and procedures for conducting long term analysis of Oregon Department of Transportation (ODOT) plans and projects. Although this information is extensive, it is not intended to be exhaustive.

The APM shall be utilized by ODOT staff as well as external consultants and contractors conducting and reviewing plans, projects and/or studies for ODOT. It also applies to work performed under ODOT Grants.

The procedures addressed in this manual have been organized to follow the progression of analysis conducted for a typical transportation plan or project. It begins with project scoping and data collection, and ends with alternatives evaluation and production of the final report. There are examples provided to “walk” the user through a process. These examples are denoted by black bars at the beginning and end of the example.

While the direction provided represents recommended best-practices for producing consistent and accurate results, it should be recognized that every project analysis presents a unique set of problems to address. This manual is not intended to replace the need for sound engineering judgment, which must continue to be a vital part in the process of applying the methodologies to individual studies.

1.2 Manual Structure

Acronyms are shown in parenthesis after a term, phrase or reference is listed the first time. The acronym is used thereafter in the text.

Manuals, papers and other publication titles are italicized.

There are a number of references to web sites, web pages and web accessed documents. Appendix A is a listing of all web links referenced.

Examples are identified with a solid bar the width of the page at the beginning and end of each example.

1.3 Manual Updates

Analysis techniques and project requirements change over time. The ability to immediately incorporate new information into this manual is essential to providing users with the most current resource possible. To accommodate expedient updating, the APM has been designed as an on-line tool, and the on-line version is the official document.

As this is an on-line document, and will not be published and distributed as a traditional publication, there is no user list for update notifications. It is the user's responsibility to verify they are using the most current version of information as their reference.

Update information will be detailed on the web page so that users can identify what has changed and when. Updated pages will have the change date as part of the document footer.

1.4 ODOT Structure

ODOT, through its ten Divisions, is responsible for developing Oregon's:

- System of highways and bridges
- Public transportation services
- Rail passenger and freight systems
- Bicycle and pedestrian paths
- Driver licensing and vehicle registration programs o Motor carrier operations, and
- Transportation safety programs

As part of the Transportation Development Division (TDD), the Transportation Planning Section provides direction for long-term management and improvement of the transportation system through analysis, modeling, research, policy development and coordination with other agencies.

Analysis and modeling for Regions 2-5 are the responsibility of the Transportation Planning Analysis Unit (TPAU), which uses these tools to:

- Provide support and evaluation of corridor and transportation system plans.
- Provide transportation model development guidance to region staff, metropolitan planning organizations (MPO) and local jurisdictions.
- Forecast transportation needs and evaluate the impacts of transportation, economic and land use decisions.
- Help decision-makers find cost-effective ways to manage transportation facilities that alleviate traffic congestion.
- Conduct transportation analysis to support project selection decisions for the Statewide Transportation Improvement Program (STIP).
- Develop traffic data for environmental analysis and design recommendations during project development.

Traffic analysis may also be conducted by Region staff, which is the case in Region 1. Analysis conducted in Region 1 is the responsibility of the Traffic Section, which resides in the Technical Center.

Note: All references in this manual to the Department refer to ODOT, and all references to regions relate to ODOT regions.

2 MANAGING ANALYSIS PROJECTS

2.1 Purpose

In general, transportation analysis projects and planning studies will follow a similar outline of procedures for gathering and analyzing data. This chapter will provide guidance in common procedures and data needs for:

- Scoping
- Reviewing Analysis Work
- Coordinating with Other ODOT Units

2.2 Scoping

2.2.1 How to Determine the Type and Scope of Procedures to Apply

Selection of the appropriate analysis procedures from this manual will often be determined by project-specific characteristics such as the type of project, the surrounding environment and land uses, availability of data and the type of traffic controls present in the field. Generally, similar types of projects will use the same analysis procedures to varying degrees. Exhibit 2-1 lists common analysis procedures used for most projects, and identifies the various types of data needed for each. Depending on the study and the project's purpose and need, additional data may be required.

In general, transportation planning studies involve less detailed analysis, and therefore, will require the use of fewer or less complex analysis procedures to complete the work. Conversely, transportation projects and land development proposals usually involve a more detailed level of analysis, and may require the use of many or more complex analysis procedures. For further information on project scoping and selection of traffic analysis procedures, refer to the Federal Highway Administration's (FHWA) website for Traffic Analysis Tools.

It is important that scopes of work that involve analysis by consultants specify that all analysis input and output sheets shall be provided as the project progresses, in order to expedite the review process.

Exhibit 2-1 Common Procedures and Data Needs for Transportation Analysis Projects

	Vehicle Counts	Seasonal Factor	Growth Rate	Design Hour Volume	Peak Hour Factor	Percent Heavy Vehicles	Geometric Information	Vehicle Speed	Saturation Flow Rates	Signal Timing Sheets	Land Use Trip Generation	Crash Reports
Volumes												
30th Highest Hour Volumes	X	X	X									
Design Hour Volumes	X	X	X									
Trend Line Analysis	X		X	X								
Cumulative Analysis	X		X	X							X	
Forecasting Model	X		X	X		X	X	X[E]			X	
Trip Generation											X	
Trip Distribution/Assignment	X										X	
Operating Conditions												
Signalized Capacity				X	X	X	X		X	X		
Unsignalized Capacity				X	X	X	X					
Preliminary Signal Warrants	X	X	X				X	X[A]				
Segment Analysis				X	X	X	X	X[B]				
Signal Warrants (per MUTCD)	X			X			X	X[A]				X
Turn Lane Criteria Analysis				X			X	X[C]				X
Queuing Analysis				X	X	X	X	X[A]	X	X		
Progression Analysis				X	X	X	X	X[D]	X	X		
Weaving Analysis				X	X	X	X	X[B]				
Merge/Diverge Analysis				X	X	X	X	X[B]				
Passing/Climbing Lanes				X	X	X	X	X[B]				
Simulation Modeling				X	X	X	X	X[D]	X	X		
Arterial Analysis				X	X	X	X	X[D]				
Safety Assessment												
Crash Analysis	X						X					X
Sight Distance Evaluation							X	X[C]				
Access Spacing							X	X[B]				
Throat Design/Site Circulation				X	X	X	X				X	
Planning Studies (e.g., Transportation System Plans, Corridor Plans, and Refinement Plans) do not typically consider signal phasing or progression analysis, and only include general queuing analysis to note blockages and spillbacks.												

[A]85th percentile (operating) speed
[B]Free-flow speed
[C]Design speed
[D]Running speed
[E]Posted speed

2.2.2 Study Scoping Procedures

The purpose of establishing a scope of work (SOW) for a transportation study is to define critical parameters such as the study area boundaries, analysis requirements, data needs and the identification of specific concerns to be addressed. An effective scope of work should always produce a completed study that satisfies the needs of the corresponding project.

Common elements of a SOW for most types of transportation studies include:

- Background or Purpose Statement
- Objectives of the Study
- List of Work Tasks
- Identification of Deliverables
- Project Schedule
- Project Budget

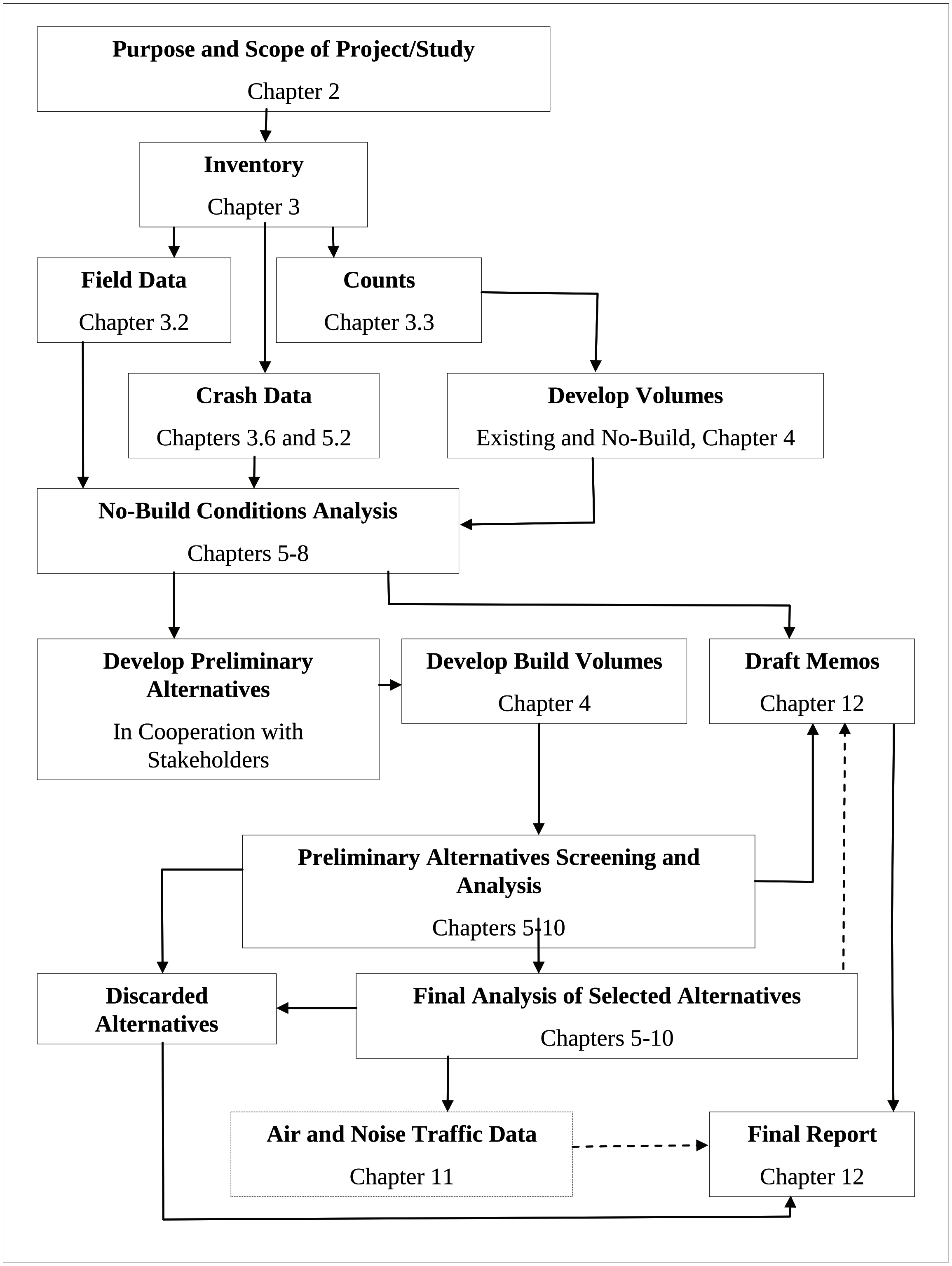
It is important that the work tasks and corresponding deliverables be clearly defined, and that the party responsible for completing them is identified. The distribution list for deliverables should generally include members of the Citizen's Advisory Committee (CAC), the Project Team (PT), the roadway designer, the Region Traffic Manager, and Traffic – Roadway Section (TRS) if signals are involved. Exhibit 2-1 can be used in conjunction with Exhibit 2-2 to assist the analyst in creating the work tasks and deliverables by identifying potential analysis procedures for implementation and their corresponding data needs. It should be recognized that this table and figure are not exhaustive, nor do they require all analysis procedures identified for a given project type to be implemented, and that engineering judgment will be needed to create an appropriate list of work tasks. When completed, the work tasks identified and corresponding deliverables should achieve the objectives of the study.

A useful tool during scoping is TransGIS. It can be used to quickly map and access key data, such as:

- Crashes
- SIP segments
- Congestion
- Volumes
- Rail, rail crossings
- Pavement and bridge conditions
- ATRs
- Posted speed
- STIP projects

As an example, an analyst scoping a study could use TransGIS to map crash locations over the last five years to identify areas where crashes are concentrated. Complete crash data can then be accessed for groups of crashes identified on the map. The analyst may use this information to identify in the SOW specific locations within the study area that need more detailed crash analysis. The TranGIS tool is available on the TDD Transportation Data GIS Unit Website.

Exhibit 2-2 Process of Traffic Analysis



The SOW should include the project title, highway name and number, and a purpose statement to identify the project objectives. Timelines should be defined by number of weeks required to complete each task. Target completion dates for each task should be established. Delays or additional work requests will extend the timelines. A typical analysis will include most, but not necessarily all, of the following tasks.

- **Task 1 – Transportation System Inventory**

  The purpose of this task is to review existing data and collect additional inventory data for the study area. Note: Allow 6-8 weeks from date of request for counts requested from ODOT.

  The methodology for this task should specify the following:

  - Count Request

    - Locations of 16-hour manual classification counts to be taken at major and minor road locations; consider if needed at minor road locations.
    - Locations of 2 or 3-hour manual counts to be taken at major and minor road locations; consider if needed at major accesses. See Chapter 4 to determine the count duration.
    - Locations of 48-hour or 7-day hose tube counts to be taken on free-flow segments including interchange ramps.

  - Field inventory data needed including, but not limited to:

    - Speed Limits
    - Lane Geometrics
    - Intersection/Access Spacing
    - Storage Bay Lengths
    - Intersection Controls

  - Other inventory data needed including, but not limited to:

    - 3-Year Crash Report for roadways through study area
    - Map of area using Geographic Information System (GIS) for figures
    - Functional class and freight/truck route data
    - Comprehensive plan data (zoning code and map)
    - State and local performance measures
    - Saturation flow rates taken at major signalized intersections

  - Other optional inventory data that may be needed, depending on project.

    - Sight Distances
    - Travel Times
    - License Plate Surveys (can be expensive)
    - Origin-Destination (O-D) Surveys (very expensive and require Oregon Transportation Commission (OTC) approval for state highways)

Task deliverables include inventory information, project area map and photo for use in the following tasks.

- **Task 2 – Develop No-Build (Existing and Future) Design Hour Volumes**

  The purpose of this task is to develop base year, build year and future year no- build design hour volumes (30th highest). The base year is the year of the study, or when most of the data was gathered. The build year is the year that has the day of opening of the project. Generally, the build year is one year (for small projects like intersections) to two years (for large projects like interchanges) from the let date shown on the project prospectus. The future (design) year is typically 20 years from the build year. For example, a 2005 interchange project with a let date in 2009 would have a base year of 2005, a build year of 2011 and a future year of 2031.

  The methodology for this task is to use the manual count data to obtain the 30th highest hour volumes (30HV). The seasonal factors developed from local ATRs (Automatic Traffic Recorders) should be used to convert volumes to 30HV. Use yearly factors developed from the historical-based Future Volume Table to adjust counts up or down to the base year. Volumes should be balanced as much as possible between intersections with a single system peak hour.

  The future volume development process should be summarized, whether by historical trends, cumulative analysis, or with a transportation model.

  Task deliverables include balanced base, build and future year 30HV no-build volumes on figures for Technical Memorandum #1.

- **Task 3 – Analysis of No-Build Transportation System**

  The purpose of this task is to evaluate no-build system conditions for the base, build and future years.

  The methodology for this task should use the base year, build year and future year data developed in Task 2 along with current Highway Capacity Manual (HCM)-based analysis software to evaluate the system by performing the following:

  - Use crash data from Task 1 and create segment crash rates to be compared with published crash rates for similar facilities. Analyze the crash data for any patterns and identify any possible countermeasures.
  - Preliminary signal warrants (either TPAU's ADT-based ones or Manual of Uniform Traffic Control Devices (MUCTD) Warrant 1 will be evaluated for unsignalized intersections. The results of any analysis involving signals or intersections that meet preliminary signal warrants should be discussed with Region Traffic prior to distribution.
  - Evaluate the volume to capacity (v/c) and level of service (LOS) for the study area for intersections, merge/diverge/weaving sections, freeway mainlines and

highway segments. ODOT's current analysis software preferences are Synchro, SimTraffic, and HCS.
  - The output no-build v/cs must be compared with the Oregon Highway Plan (OHP) v/c standards for state facilities. Non-state v/c or LOS need to be compared with the appropriate local operational performance measure standard.
  - Turn bay storage lengths will be compared to the 95th percentile no-build queues. Blocking of turn bays and upstream intersections must be noted. Simulation modeling may be needed to evaluate the above if there are multiple signals involved or congested conditions exist.
  - Check if accesses, intersections and interchanges meet OHP and Division 51 spacing requirements.

The task deliverable is Technical Memorandum #1, which includes the crash analysis, preliminary signal warrants, no-build system LOS, v/c, 95th percentile queues and identification of access and spacing issues.

- **Task 4 – Evaluate Preliminary Alternatives (Screening)**

The purpose of this task is to work with the PT, the Region Roadway Section and the TRS to develop and screen the preliminary alternatives.

The methodology for this task is to review goals and objectives with the PT considering identified needs, and evaluate each preliminary alternative by comparing operations at major intersections or other agreed upon key location(s) using the future design year volumes. Models can also be used to screen alternatives effectively if the alternatives have the potential to change traffic patterns beyond the local area. The future no-build alternative needs to be included in the analysis as the baseline that the preliminary alternatives are compared against.

Any comparisons using HCM-based v/c's need to use the 2003 Highway Design Manual (HDM) v/c's. Model-based link v/c's have different methodology and cannot be directly compared to the HDM v/c's. Model-based screening criteria should be based on relative comparisons as much as possible.

The task deliverable is Technical Memorandum #2, with the screening criteria and results shown for each alternative. A summary comparison table that shows how the alternatives and the future no-build alternative perform against the screening-level criteria must be included.

- **Task 5 – Evaluate Build Alternatives**

  The purpose of this task is to work with the PT, the Region Roadway Section and the TRS to develop and completely evaluate the detailed alternatives that satisfy the future transportation needs for this project.

  The methodology for this task is:

  - Develop build and future year Design Hour Volumes (DHVs) for each of the final alternatives. Either distribute the no-build volumes on the build alternatives, or create new build volumes for each alternative if currently diverting traffic that would return with the new alternative is sufficient to invalidate the use of the no-build volumes.
  - Preliminary signal warrants (either TPAU's ADT-based ones or MUCTD Warrant 1) will be evaluated for unsignalized intersections.
  - Evaluate the v/c and LOS for the study area for intersections, merge/diverge/weaving sections, freeway mainlines and highway segments. ODOT's current analysis software preferences are Synchro, SimTraffic, and HCS.
  - The output build v/c's must be compared with the HDM design v/c's for state facilities. Non-state v/c or LOS need to be compared with the appropriate local operational performance measure standard.
  - Determine turn bay storage lengths using the 95th percentile build queues.
  - Blocking of turn bays and upstream intersections must be noted. Simulation modeling may be needed to evaluate the above if there are multiple signals involved or congested conditions exist. If simulation is needed for each of the alternatives, allow approximately 1 week per alternative. More than 1 week per alternative will be needed for alternatives where multiple hours are being evaluated.
  - Work with TRS/Region Traffic if new signals or changes to existing signals are involved. A progression analysis for the study area is needed if more than one signal is included in the alternative.
  - Check if accesses, intersections and interchanges meet or improve over current conditions in the OHP and Division 51 spacing requirements.

  The task deliverables include Technical Memorandum #3 with volumes, LOS, v/c and 95th percentile vehicle queues shown on diagrams for each alternative, a summary comparison of the alternatives (including a table) and how they fared against the evaluation criteria and each other. Also, the design storage lengths and other geometric details need to be forwarded onto the appropriate design staff, either ODOT or consultant.

- **Task 6 – Draft Traffic Narrative**

  The purpose of this task is to select the final solution for the project and write the draft traffic narrative report.

The methodology for this task is to work with the PT and Region Roadway Design and Region Traffic Sections. Use the goals, objectives, and evaluation criteria and information from Technical Memorandums #1 through #3.

The task deliverables include the preferred alternative, draft traffic narrative and all diagrams for both selected and non-selected alternatives. There may be more than one alternative that meets the traffic/transportation related goals and objectives. In these cases the decision making body will need to consider other information and make a decision on the preferred alternative.

- **Task 7a – Air/Noise Traffic Data for No-Build System**

  The purpose of this task is to produce air/noise traffic data for the no-build system conditions. This includes the base year, build year and future no-build volumes. Larger projects may also require the creation of short-term future year (10 year) data. The analyst should contact the air/noise specialist who will be using this information before beginning this task to ensure the correct information is provided. There may be some differences in data requirements form project to project, depending on the needs of the user of the data.

  The methodology for this task is to use the balanced hourly volumes, average daily traffic (ADT), LOS C for the base, build and future years and truck classification data to compute the noise traffic data. Use the TruckSum summary spreadsheets (Chapter 11) and the EISBase program. Work with the noise consultant to confirm years and data results requirements. In congested areas air quality data might be needed, which involves intersection LOS and number of stopping vehicles on each approach.
  The task deliverables include the air/noise traffic data for the base, build and future no-build years delivered to the air/noise consultant. Diagrams are also required as the identification/location key for the data.

- **Task 7b – Air/Noise Traffic Data for Preferred Alternative**

  The purpose of this task is to produce air/noise traffic data for the preferred alternative for the build (opening year) and future design year.

  The methodology for this task is to use the balanced hourly volumes, ADT, LOS C for the build and (short and long-term) future years along with truck classification data to compute the noise traffic data. Use the TruckSum summary spreadsheets (Chapter 11) and the EISBase program. In congested areas air quality data that involves intersection LOS and number of stopping vehicles on each approach might be needed.

  The task deliverables include the air/noise traffic data for the build and future no-build years delivered to the air/noise consultant. Diagrams are also required as the identification/location key for the data.

- **Task 8 – Final Traffic Narrative**

  The purpose of this task is to produce the final traffic narrative.

  The methodology for this task is to use the draft traffic narrative completed in Task 6. Incorporate any changes since draft narrative was completed. Append the air/noise traffic data sheets from Tasks 7a and 7b.

  The task deliverable is the final traffic narrative delivered to project leader or team, Geo/Hydro, Bridge (if necessary) and TRS.

2.2.3 Scoping a Traffic Impact Study

The purpose of providing a SOW for a TIS is to define the study area boundaries, establish the analysis requirements and convey specific concerns to be addressed. The SOW should be created with the goal of identifying the proposed development's impacts to the transportation system, as well as the potential infrastructure improvements necessary to mitigate development impacts. Quite often the effectiveness of the final TIS in evaluating impacts and associated mitigation options is dependent on the quality of the initial scoping.

Chapter 3.3 of ODOT's Development Review Guidelines provides guidance on the preparation and review of TIS's, and should be consulted when creating a SOW for a TIS.

2.3 Reviewing Analysis Work

Often times an analyst may be required to review work conducted by others, whether it was performed within the Department or by a consultant. The following section provides general guidance for reviewing traffic analysis that can be applied to any type of analysis project. Specific guidance for the review of TISs can be found in ODOT's Development Review Guidelines.

When reviewing analysis conducted by others, knowledge of the study area is often very beneficial. The reviewer should first examine all study area mapping and photographs available and should visit the site, if practical to do so.

With any type of technical analysis the proper collection and processing of data is critical to obtaining accurate results. Before reviewing the analysis itself, verify that the data used is appropriate for the analysis conducted. Consider things such as the time of data collection, type of data used and whether any processing of data (e.g., volume balancing) was conducted correctly.

The calculations performed in the study should be checked for computational errors, and procedures used should be appropriate for the given situation and in compliance with accepted ODOT practices. Keep in mind that assumptions made by the analyst performing the work can have a significant effect on analysis results, even if specific analysis procedures are followed correctly. Knowledge of the study area, prevailing traffic conditions and accepted ODOT analysis procedures will aid the reviewer in determining which assumptions are appropriate, and which are not.

In addition to technical accuracy, the results of the analysis should be considered using a "reasonableness" test. The reviewer should compare the subject data, such as the traffic volume counts, lane configurations and traffic controls, and determine whether the conclusions and recommendations of the study seem reasonable. This type of test can often help pinpoint sources of error in analysis, and may address questions likely to arise when the project is presented to the public.

When sources of error are detected in the analysis, the reviewer should not only note the error itself, but should acknowledge the significance of the error to the results of the analysis. There may be times when correcting the error would require a substantial amount of work, but the results of the corrected analysis would not be significantly different and the recommendations of the study would remain unchanged. Noting the significance of the error ahead of time will enable the Department to determine whether correction is necessary or cost-effective.

Once the adequacy of the analysis has been verified, compare the results to ODOT's adopted performance measures. Check any proposed mitigation against ODOT's design standards. Often times the review process will require coordination with other units within the Department that have specific expertise in, or authority over, certain elements of the design or approval of the mitigation proposed.

2.4 Coordinating with Other ODOT Units

The following section describes various units within ODOT that are commonly involved in decision making regarding highway improvements. One or more of these units may need to be contacted for input or to discuss problems and possible solutions regarding a specific project. It is preferable to begin with staff at the Region or District level.

2.4.1 Transportation

Region Traffic Units

Region Traffic Units provide expertise to region and district staff on current traffic policies and procedures. Staff is responsible for overseeing all traffic engineering design (including signal and sign design) for Region projects. Staff actively participate as members of project development teams (PDT) to help insure that traffic related issues are considered early in the process, and to provide traffic information to the team. They also act as the traffic liaison to local agencies on behalf of ODOT.

The Region Traffic Manager/Engineer may authorize standard applications of traffic control devices, and some modifications to existing traffic control devices, provided the applications are in compliance with the principles outlined in the Manual on Uniform Traffic Control Devices for Streets and Highways and applicable ODOT policies and guidelines. A list of items that may be authorized by the Region Traffic Manager/Engineer can be found in the ODOT Traffic Manual.

Traffic–Roadway Section (TRS)

This section prepares specifications, maintains standards for traffic devices and related facilities and provides design expertise in materials, operations and construction support services. TRS consists of five central units under the authority of the State Traffic Engineer: Roadway Engineering, Office of Project Letting (OPL), Geometronics, Traffic Standards and Asset Management (TSAM), and Traffic Engineering.

The State Traffic Engineer has delegated authority to approve the installation of traffic control devices on state highways. This includes the installation of all new signals, major modifications to existing signals and installation of any other traffic control device on state highways. All delegated authority requests for State Traffic Engineer approval should follow roughly the same process.

- Consultation with Region Traffic Unit; and
- A request sent through the Region Traffic Manager/Engineer with supporting documentation.

A list of items that require approval by the State Traffic Engineer for use on state highways can be found in the ODOT Traffic Manual.

Transportation Planning Analysis Unit
The Transportation Planning Analysis Unit (TPAU) provides support and guidance for the development of corridor and urban transportation system plans, transportation model development and use of computer models to forecast transportation needs. TPAU also maintains the Congestion Management System (CMS) and the Highway Economic Reporting System (HERS) that decision-makers use to find cost-effective ways to manage transportation facilities that alleviate traffic congestion. Other duties include supporting ODOT's project development process by conducting transportation analysis to aid project selection for the Statewide Transportation Improvement Program (STIP) and developing data for environmental analysis. TPAU often acts as a resource to Region Traffic Units requesting technical assistance.

2.4.2 Other ODOT Groups

Right of Way Section
The Right of Way (ROW) Section is responsible for the appraisal, acquisition and management of property acquired for public projects, and assists people and businesses in relocating from the acquired right of ways. This section should be consulted when proposed improvements require more highway right of way than is currently available. The ROW Section also conducts research regarding private property access rights when an approach application is submitted.

District Maintenance Offices
The District Maintenance Offices are responsible for the on-going preservation and operation of state transportation facilities and the permitting of all activities (utility, access, miscellaneous) within the highway right of way. They are often very familiar with local issues and the operational and maintenance history of individual highways, and can offer valuable input during the process of identifying needs and mitigation alternatives, in addition to tracking the status of existing permits. Because they will ultimately be responsible for maintaining any proposed improvements, they should be consulted during the selection and design of proposed mitigation as well.

Region Roadway Engineering
Region Roadway Engineering is responsible for the design of new highways and highway features. Early consultation with this unit when evaluating potential improvements can help in identifying fatal flaws and ensure design standards can be met before recommending an alternative for implementation.

Long-Range Planners
ODOT Regions have long-range planners who are familiar with the local government

Transportation System Plans (TSPs) and Comprehensive Plans. TDD also has specialized long-range planners for various travel modes. Coordination with the long- range planners can help identify planned transportation improvements in the study area that could alter future traffic patterns or conflict with proposed improvements.

Rail Division

The Rail Division has jurisdiction over railroad crossings and traffic control devices used within crossing areas. They also have exclusive legal authority over public grade crossings and provide coordination with the railroads for affected private rail crossings. The Rail Division should be contacted any time a project will have an impact directly to or within 500 feet of a railroad or rail crossing.

Bicycle and Pedestrian Program

This program is a group within TRS and provides technical assistance within the Department and to local officials regarding walkway and bikeway design (construction and maintenance), issues grants to local officials regarding bicycle and pedestrian issues and reviews construction plans and TSPs to ensure that bicycle and pedestrian needs are met. They should be used as a resource to use any time bicycle or pedestrian improvements are proposed or existing facilities are impacted by other proposed highway improvements.

Region Environmental/Geo-Environmental Section

Environmental staff is responsible for coordinating environmental regulatory compliance for all transportation improvement programs in the state that use federal funds. Potential issues requiring involvement of the Geo-Environmental Section include:

- Environmental justice
- Threatened and endangered species
- Wetlands
- Historic buildings
- Air quality
- Noise
- Erosion control
- Stream-bank stabilization
- Fish passage
- Storm water quality and quantity

The ODOT Environmental Engineering Acoustical Group is responsible for ODOT’s traffic and construction noise analyses, noise barrier design and noise mitigation recommendations. Their work will often be dependent on data provided by Region traffic analysts and/or consultants as described in Chapter 11.

3 TRANSPORTATION SYSTEM INVENTORY

3.1 Purpose

Before any analysis can begin, data for the study area must be collected from the field or other available sources. This chapter provides guidance in the selection criteria and collection methods of appropriate data for use in transportation analysis. Topics covered include:

- Field Inventory
- Vehicle Count Surveys
- Travel Time Surveys
- Saturation Flow Rate Studies
- Crash Data
- Data Resources from ODOT

3.2 Field Inventory

Specific data related to field conditions that may affect traffic safety and operations shall be collected directly during a visit to the area. In addition, inventory data collected through other sources such as previously conducted studies or databases maintained by the agency with jurisdiction over the roadway should be field verified. There is no substitute for a field visit as an analyst cannot get a good feel for the project area otherwise. Notes, photographs and/or video should be taken of the project area in addition to the inventory data to reference, and possibly include, as graphical elements in the final report. The most common types of field data needed are discussed below.

3.2.1 Geometric Data

Geometric data typically includes:

- Street names
- Lane/shoulder/median widths
- Lane configurations
- Sidewalk widths and locations
- Intersection and access spacing
- Horizontal and vertical roadway alignments
- Storage bay lengths (from stop-bar to start of taper)
- Storage bay taper lengths
- Bike lanes and width
- Parking width and locations

3.2.2 Operational Data

Operational data typically includes:

- Speed limits
- Intersection controls (signalized, stop-controlled, yield, merge, etc.)
- Signal characteristics (timed, actuated, split-phased, protected left turns, etc.)
- Signing (especially turn prohibitions)
- Parking locations, signing, striping, and frequency of parking maneuvers
- Crosswalk location, width, and frequency of use
- Bus stop locations and bus route frequency
- General operations observed during peak period on field visit; note perceived and actual problems, standing queues, poor access/intersection spacing, movements of vehicles and pedestrians/bicyclists
- Saturation flow studies
- Travel time studies
- Rail crossing locations, train frequency and duration of blockages

3.2.3 Simulation-Specific Data

In addition to the geometric and operational field data, additional simulation specific data will be required if a project will require a simulation to be performed. Simulation-specific data typically may include:

- Number of detectors, length, and distances from stop bar or crosswalk
- Locations where vehicles improperly use the shoulder or median to move around blockage points or due to driver confusion
- Turning Speeds (if unusual geometrics or conditions exist)
- Free-Flow speeds on ramps, highway segments, or on intersection discharge legs (can be collected using road tubes or speed guns)
- Floating car travel times and average speeds (see Section 3.4)
- Lane Utilization, are some lanes used more heavily than others in the same approach, what is causing the behavior
- If more than one lane is available to turn into from an approach, indicate the lane that turning vehicles align into (left or right alignment) regardless if the movement is legally proper or not.
- Important travel patterns (OD Data), Example: ~75% of traffic exiting the mall makes left at Main Street
- Lane-change positioning lengths or land marks where vehicles tend to position themselves in a lane to make an upcoming turn or merge
- List the approximate average and maximum queues for each lane
- Note any upstream intersections or bays blocked by congestion from the intersection being observed and what percentage of the site visit hour did blocking occur <5%, 10%, 25%, 50%?
- Arrival type for each approach (i.e. are the vehicles arriving in a platoon, if so, are they arriving on red or green?)

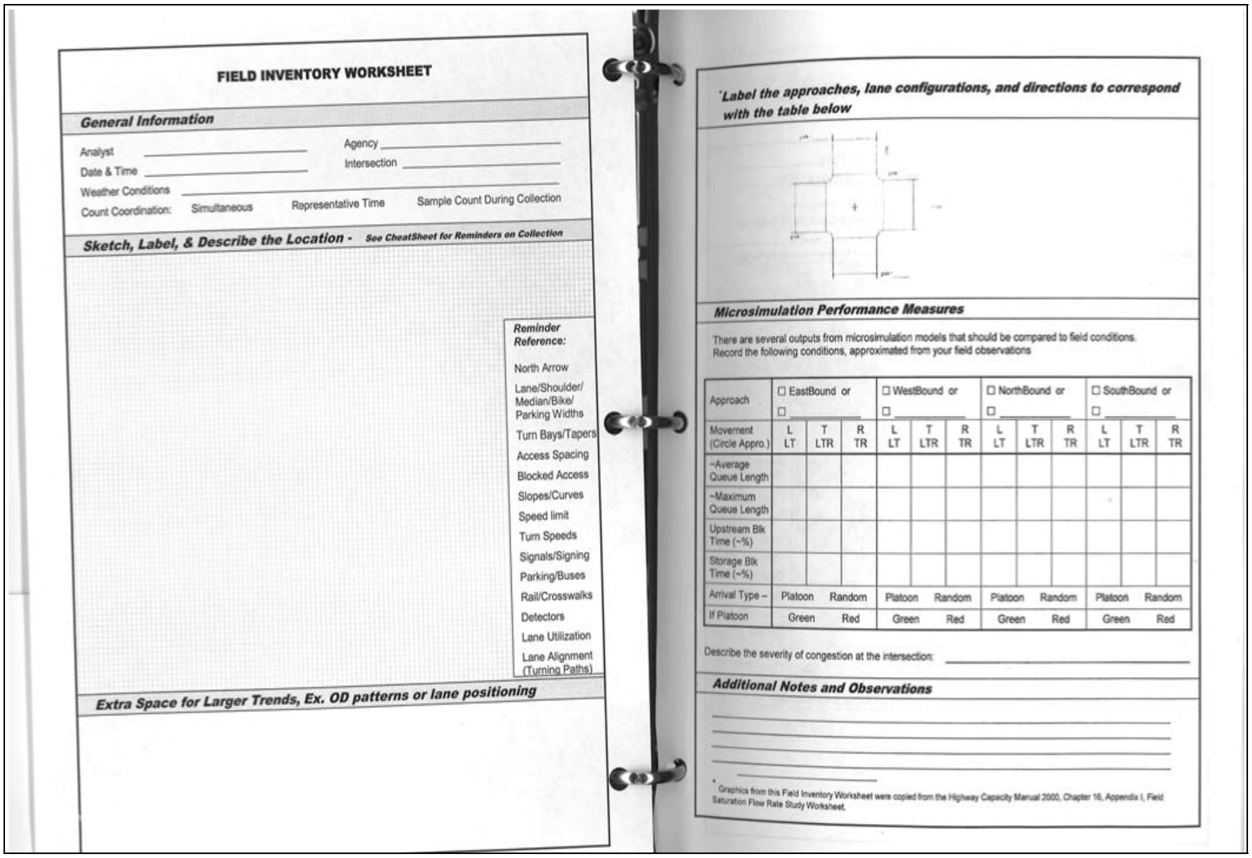
3.2.4 Field Inventory Worksheet

The Field Inventory Worksheet has been designed by TPAU to be generic enough to aid in the collection of field data for all projects at all detail levels from a single intersection realignment project to a highway bypass project that will require a full calibrated simulation. For projects that will NOT involve simulation, it is only recommended but not required that the field inventory occur near the analysis time period. For projects requiring simulation, the simulation specific field data and volumes collected MUST be collected at a time that represents the analysis time period (the 30th highest hour) as closely as possible. The field inventory should be collected by the analyst that will be conducting the work (and/or overseeing the work), and if at all possible, should be done at the time during the vehicle counts being collected for the study (Vehicle Counts described in Section 3.3). See Chapter 8 for guidance and instruction on calibrating simulations.

With large project areas, it may be very difficult or logistically impossible to obtain all counts and inventory on a single day which represents the 30th highest hour. If counts cannot all be collected on the same day (or year), every effort should be made to collect counts and field inventory at primary locations on a day that is on, or closely represents, the 30th highest hour. If it is not possible for any of the counts to occur on a day representing the 30th highest hour and/or the field inventory can not be collected at a time that represents the 30th highest hour for the primary locations in the study area, then short sample counts should be conducted during the field inventory collection to factor the off- peak counts to the day the study area was visited. If this is the case and simulation is required for the study, use the seasonal factor methodology described in Section 4.4 to determine if the vehicle count day is representative of the 30th highest hour. If the primary counts for the study area occurred during a period that is less than 90% of the 30th highest hour for that area seasonal trend type, then sample counts during the field collection (which must occur in that 90% window) will be required for the calibration or "existing" model. These rules are established to help ensure that calibration volumes 1) are near the 30th highest hour and 2) represent conditions that have been witnessed in the field.

The Field Inventory Worksheet, Exhibit 3-1 (actual worksheet form found in Appendix H), will help facilitate the field data collection process for the field data listed above. The worksheet can be used for projects where just geometry and observational data is required or for projects requiring simulation where all the data listed above should be addressed. Exhibit 3-2 shows an example of a completed worksheet for a simulation project. Note that the worksheet is intended to be printed multiple times for a given project area. The collection of worksheets can be placed in a three-ring binder providing a hard writing surface. Each copy of the worksheet can be used for each intersection or area of interest in the study and all copies can be neatly organized in a single project binder.

Exhibit 3-1 Field Inventory Worksheet - Intended Setup



FIELD INVENTORY WORKSHEET

General Information

Analyst ____________ Agency ____________

Date & Time ____________ Intersection ____________

Weather Conditions ____________

Count Coordination: Simultaneous Representative Time Sample Count During Collection

Sketch, Label, & Describe the Location - *See CheatSheet for Reminders on Collection*

Reminder Reference:

North Arrow

Lane/Shoulder/ Median/Bike/ Parking Widths

Turn Bays/Tapers

Access Spacing

Blocked Access

Slopes/Curves

Speed limit

Turn Speeds

Signals/Signing

Parking/Buses

Rail/Crosswalks

Detectors

Lane Utilization

Lane Alignment (Turning Paths)

Extra Space for Larger Trends, Ex. OD patterns or lane positioning

*Label the approaches, lane configurations, and directions to correspond with the table below

Microsimulation Performance Measures

There are several outputs from microsimulation models that should be compared to field conditions. Record the following conditions, approximated from your field observations

Approach	☐ EastBound or ☐ ____			☐ WestBound or ☐ ____			☐ NorthBound or ☐ ____			☐ SouthBound or ☐ ____		
Movement (Circle Appro.)	L LT	T LTR	R TR	L LT	T LTR	R TR	L LT	T LTR	R TR	L LT	T LTR	R TR
~Average Queue Length												
~Maximum Queue Length												
Upstream Blk Time (~%)												
Storage Blk Time (~%)												
Arrival Type –	Platoon	Random		Platoon	Random		Platoon	Random		Platoon	Random	
If Platoon	Green	Red		Green	Red		Green	Red		Green	Red	

Describe the severity of congestion at the intersection: ____________

Additional Notes and Observations

* Graphics from this Field Inventory Worksheet were copied from the Highway Capacity Manual 2000, Chapter 16, Appendix I, Field Saturation Flow Rate Study Worksheet.

Exhibit 3-2 Completed Example Field Inventory Worksheet

FIELD INVENTORY WORKSHEET

General Information

Analyst Alex Bettinardi

Agency ODOT-TDD-TPAU

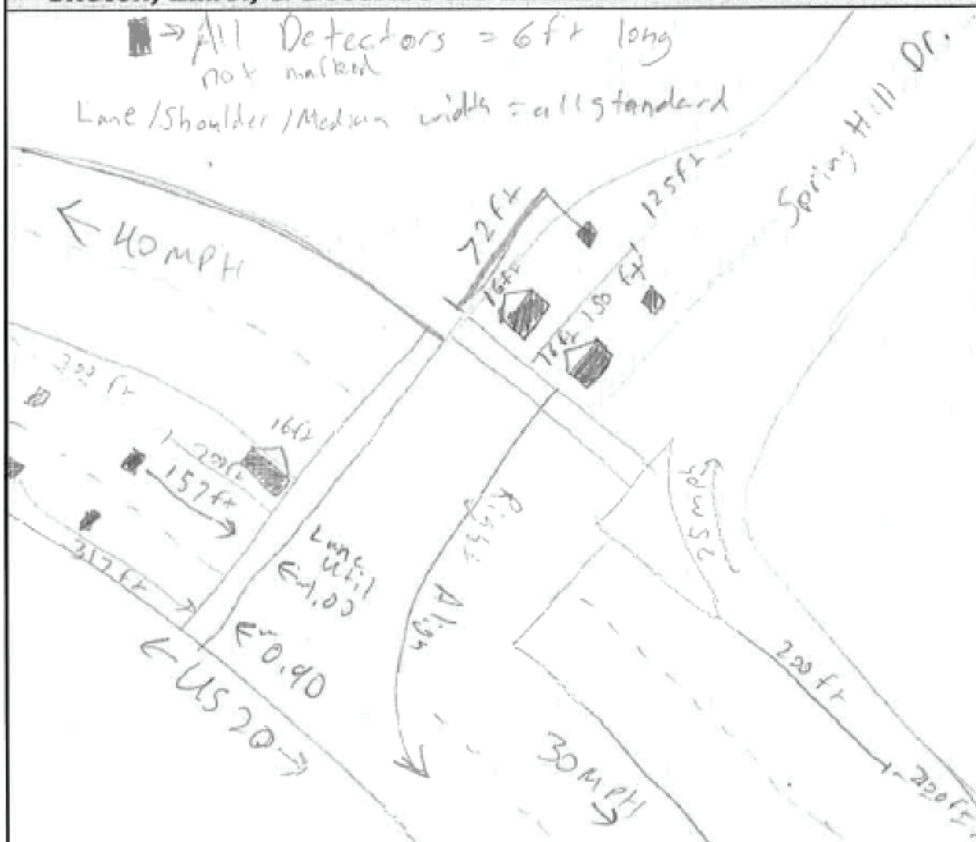
Date & Time July 25th, 2006 4-6:00 PM

Intersection Spring Hill Drive & US 20

Weather Conditions Sunny and Warm

Count Coordination: Simultaneous   Representative Time   Sample Count During Collection

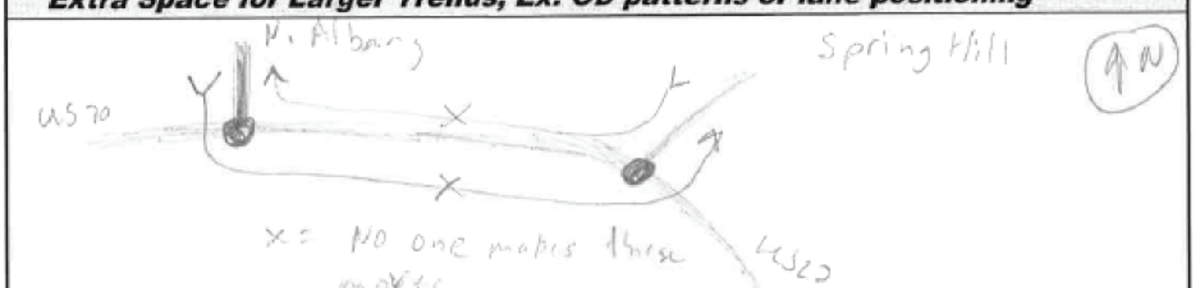
Sketch, Label, & Describe the Location - ***See CheatSheet for Reminders on Collection***



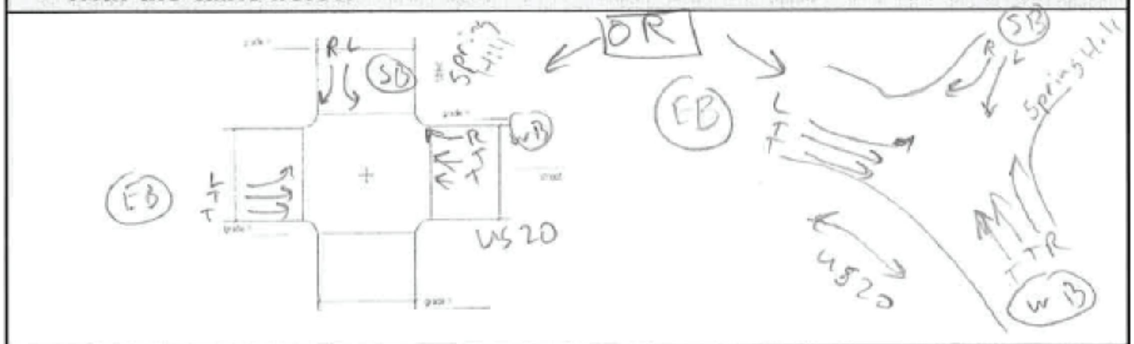
Reminder Reference:

- ✓ North Arrow
- ✓ Lane/Shoulder/Median/Bike/Parking Widths
- ✓ Turn Bays/Tapers
- ✓ ~~Access Spacing~~
- ✓ ~~Blocked Access~~
- ✓ Slopes/Curves
- ✓ Speed limit
- ✓ Turn Speeds
- ✓ Signals/Signing
- ✓ ~~Parking/Buses~~
- ✓ ~~Rail~~/Crosswalks
- ✓ Detectors
- ✓ Lane Utilization
- ✓ Lane Alignment (Turning Paths)

Extra Space for Larger Trends, Ex. OD patterns or lane positioning



***Label the approaches, lane configurations, and directions to correspond with the table below**



Microsimulation Performance Measures

There are several outputs from microsimulation models that should be compared to field conditions. Record the following conditions, approximated from your field observations

Approach	☑ EastBound or ☐ ____			☑ WestBound or ☐ ____			☐ NorthBound or ☐ ____			☑ SouthBound or ☐ ____		
Movement (Circle Appro.)	L LT	T LTR	R TR	L LT	T LTR	R TR	L LT	T LTR	R TR	L LT	T LTR	R TR
~Average Queue Length	50	150			200	50				200		50
~Maximum Queue Length	150	400			Back to Bridge	400				300		300
Upstream Blk Time (~%)												
Storage Blk Time (~%)					5%					25%		<5%
Arrival Type –	Platoon	Random		Platoon	Random		Platoon	Random		Platoon	Random	
If Platoon	Green	Red		Green	Red		Green	Red		Green	Red	

Describe the severity of congestion at the intersection: Little to no congestion most of the hour, except pk 5 min

Additional Notes and Observations

This Collection seemed to represent a peak hour/season and commuter driving characteristics. This data should be applicable to counts taken on days representing the 30th highest hour for this area.

* Graphics from this Field Inventory Worksheet were copied from the Highway Capacity Manual 2000, Chapter 16, Appendix I, Field Saturation Flow Rate Study Worksheet.

3.3 Vehicle Count Surveys

The data collected from vehicle count surveys is used in nearly all types of analysis procedures, and can include information regarding volumes of vehicles, types of vehicles, vehicle speeds and directions of vehicle flow. When such information is needed, the analyst must determine the appropriate time and method of data collection to obtain the desired results.

How many counts of what type is dependent on the context of the plan or project goals and objectives. For outsourced projects and plans, a draft scope/workplan with a completed objective section is critical for efficient use of time, money and data for all involved parties. The level of count detail required will be dictated by the level of detail in the plan or project. For example, Transportation System Plans (TSP) will be less detailed than a TSP Refinement Plan.

3.3.1 Vehicle Count Types and Durations

Intersection Classification Counts

Intersection classification counts provide vital information for project development. They provide peak hourly volumes (PHV), Average Daily Traffic (ADT) and vehicle classifications such as cars, pickups, buses and trucks for each approach and movement. Additionally, the K-factor (percent of ADT in the peak hour) and the D-factor (percent of traffic in a single direction) can be derived from count data. These are then used to convert PHV to ADT. For further explanation of traffic volume characteristics, refer to the HCM – Part I: Overview.

Intersection classification counts are typically 16-hours in duration, so average daily traffic (ADT) and other relationships can be created. These counts are used at signalized intersections, intersections that may become signalized, and other important major intersections, such as interchange ramp terminals. A 16-hour count is needed when requirements exist such as multiple peak periods, truck classifications, signal warrants, air quality and/or noise studies in environmental documents. Sixteen–hour counts can also be easily used for other purposes such as pavement design or other plans or projects. Typical count costs are variable depending on travel, duration, and other specifics, but are around $1,100.

Intersection classification counts group 13 different types of vehicles, pedestrians and bicycles. Refer to the FHWA vehicle classification descriptions in Chapter 11. Classification counts can either be done manually in the field or by use of video cameras. ODOT typically uses video cameras as it does not require the presence of a field technician throughout the duration of the count, may have less influence on driver behavior in some situations, allows for more flexibility in scheduling and processing counts, and provides a database that can be easily revisited if more information is desired at a later time or if an error in the count is detected. Data is recorded in the field and is then sent to the Transportation Systems Monitoring (TSM) Unit for processing. Passenger and other two-axle vehicles are tabulated both with and without trailers. The number of axles for single-unit trucks and for all single, double and triple trailer trucks is recorded along with buses and motorcycles.

A number will be given to each count so that it can be accessed easily. A hardcopy will be stored in TSM's files. Counts are sent to the requestor either electronically or hardcopy by mail. The first page of the ODOT intersection count provides a sketch of the intersection counted, the date, location, count number and the ADT for each movement. The second page provides a summary of movements broken down into 1-hour increments. Some intersection counts will break the peak periods into 15-minute intervals instead of 1-hour intervals. Specify 15-minute count intervals for any period when peak hour factors are needed. The rest of the pages show individual turning movements with the vehicle classifications, a summary of the bicycle and pedestrian counts and a summary of the movement volumes. Sample intersection and tube counts have been included in Appendix D for reference.

Peak Period Counts

Peak period counts capture the individual vehicle movements at a location. These counts are typically used to capture the in/out turning movements at driveway accesses or to count all movements at minor or unsignalized intersections that are not being considered for signalization. Generally, separate truck percentages are not available. Peak period counts cannot be used to create daily traffic volumes or perform any signal warrant work. Use of turning movement counts are limited to counting in a single peak period. Typical peak periods are morning (6:00 AM – 9:00 AM), mid-day (11:00 AM – 1:00 PM), and evening (3:00 PM – 6:00 PM). A three-hour count is a typical duration to capture the peak hour. A four-hour afternoon peak period count can be obtained to capture both school and commuter peaks. For count durations of more than four hours or when more than one peak period is needed, it is more practical to collect a 16-hour count. Count durations less than three hours make it difficult to capture the peak hour and should be avoided. Typical count costs are variable depending on travel, duration, and other specifics, but are in the $600 range.

Road Tube Counts

Road tube counts are often employed when the details provided by intersection counts are not needed or practical given the data needs. These count individual vehicles only or can be setup to capture vehicle classifications. These counts are used to capture mid-block volumes on streets and for segment volumes on most highways and interchange ramps. Road tubes are subject to vandalism or damage, and should not be done where vehicles may stop on the tube (in congested areas or near intersections) or cross the tube at an angle (near intersections or driveways) because under or over-counting may occur. Tubes are also susceptible to be damaged on roadways with speeds at or above 40 mph, and for safety reasons, cannot be placed on high-volume expressways and freeways. Road tube counts are typically done in a 48-hour format so an entire 24-hour period can be obtained. A 7-day count can also be done if daily fluctuations over a week are necessary to be captured and are only done on roadway segments. Typical road tube count costs are around $200.

3.3.2 Other Sources of Count/Volume Information

Frequently, existing or alternative count sources are overlooked so these should be reviewed before completing the initial count list. This can, in some cases, substantially reduce the number of new counts, save on data collection costs, and cut down the number of SOW review iterations.

Transportation System Monitoring (TSM) Unit Data

- **Previously Collected Counts**

  Besides obtaining new counts there are some other sources of count information which may be used to reduce the overall new count requirement needs. ODOT has a large quantity of traffic volume data and previously collected counts. Before any new counts are ordered, the Transportation System Monitoring (TSM) Unit should be contacted to determine if any previous usable counts are available for the study area.

  In general, counts in the study area should be three years old or less. Older counts between three and five years old can sometimes be used if they are the correct type and no significant changes, such as new roads or developments, have occurred to influence traffic flows. A newer count may not accurately represent the traffic flows on a roadway section even if less than the three years old if recent development has occurred within or near the study area since the count was taken.

  If the count was completed earlier, not per request, provide the TSM data analyst with the electronic number (refers to year and number) found in the lower right corner of the cover sheet for the count printout. The TSM Unit will send either an electronic or hardcopy version of the count.

- **State Highway Vehicle Classification Data**

  State Highway vehicle classification information is available through the TSM Unit's Internet page at: http://www.oregon.gov/ODOT/TD/TDATA/tsm/tvt.shtml.
  With this information, the daily and hourly volumes can be obtained along with truck classifications which will substantially reduce the need for 48-hour road tube counts.

- **Automatic Traffic Recorders/Automatic Vehicle Classifiers (ATR/AVC) Sites**

  ATR and AVC sites can be used as substitutes for classification and regular road tube counts. Every ATR site is counted with a 24-hour classification count every three years which is available from the TSM Unit. AVC's continually classify data so classification data will be available throughout a given year at these locations. AVC's are currently replacing ATR's so soon all recorder sites will have classification abilities. ATR/AVC "Critical Hour" listings are also available which breakdown a year's worth of data down to the hour level so a 30 HV can be easily obtained at that location.

- **Ramp Volume Diagrams**

  While 16-hour counts at an interchange ramp terminal are preferable, the ramp volume diagrams in the Transportation Volume Tables and on the TSM Unit webpage can be used to substitute if a count is not available and intersection turn movements or intersection operations are not desired. Free-flow ramp volumes (i.e. between two Interstate highways) can be obtained from the diagrams if a 48-hour tube count is not available or practical.

Other Jurisdiction's Counting Programs
In addition, some counties (i.e. Deschutes) and larger cities (i.e. Medford, Portland) may have traffic counting programs in place. The TSM Unit webpage also has links to many of these jurisdiction's Internet traffic data pages. These counts are typically daily volumes and can be used to supplement the local system and can reduce the need for 48-hour road tube counts. Sometimes intersection counts are available, but differing classification breakdowns and durations from ODOT standards can make these difficult to use except for a source for local peak period counts.

Traffic Signal Controller Counts
The 170-type and later traffic signal controllers have the ability to store loop detection information that can be downloaded at a later date. This data is attractive to the end user as there are a large number of usable installations available. However, experience with use of this data shows that the count data is unreliable with a large undercount of turning vehicles. These counts are better utilized for establishing relationships (i.e. seasonal adjustments for local streets).

Special Vehicle Counts (short duration)
Frequently on projects, there is a need to collect additional peak hour data for driveways, for a check count, or other overlooked spot. Sometimes these counts are done for specific purposes such as capturing headways, weaving movements, or saturation flow rates for simulation calibration. These counts typically are collected by the project analyst rather than region or consultant counting staff. Counts longer than an hour or in many locations should be done by region staff or traffic count contractors.

These counts can be manually tabulated in case of a number of small adjacent driveway counts, or use of a video camera or electronic count board. Video cameras can be useful assuming that a good vantage point is available that will provide a clear view of all movements being counted. When using video cameras to collect count surveys, be sure to have an adequately charged battery and enough video tape to collect the amount of data needed.

Typically, when counts are not done by video, some sort of handheld electronic count device is used. One of these is the Jamar board which has been used by ODOT in the past to collect counts. These are now only used at the project analyst level. Limitations of the Jamar boards prevent using these to do a full 13-class count. However, the boards can be used for volume-only or limited class counts. These counters are necessary for saturation flow data capture or other simulation/operational data (see Chapter 8). The Jamar traffic count is in raw form and can be downloaded as text files. The data will need to be placed into a spreadsheet or run through another program (the included software is no longer compatible with today's operating systems) to get the data to a useable form.

3.3.3 Vehicle Count Periods

For most traffic studies, the 30th highest hour volumes (30 HV) should be used to represent future volumes. The 30HV is the target hour based on the concept that designs are not done to the absolute highest hour of the entire year, but to design to meet most of the needs. Solutions in plans and projects need to be done to the 30HV to be consistent with accepted analysis methods and for comparisons with mobility standards.

To get a typical traffic mix of the 30 HV for the analysis, the counts should be taken as close to the likely 30th highest hour as possible. This typically requires collecting counts on a weekday afternoon (usually in summer) in most larger urban areas, but may include weekends for high recreation areas (the coast or Central Oregon), or areas experiencing lunch hour peaks or high reverse direction flows during the day. In fully developed portions of Metropolitan Planning Organization (MPO) areas, the 30th highest hour is generally assumed to be represented by the weekday afternoon peak hour. Outside of fully developed MPO areas, a seasonal adjustment will be required to convert the counts to 30 HV.

Seasonal adjustments should not be more than 30% because the traffic flow characteristics are most likely NOT represented by the count information. A seasonal adjustment greater than 30% indicates that the count was taken at the wrong time of year. Turn movement patterns may be so different they cannot be adequately represented by a seasonal adjustment. Count timing is critical especially if the project/plan SOW will not be complete until after October. Please refer to Section 4.4 in the Analysis Procedure Manual or contact the Transportation Planning Analysis Unit (TPAU) for advice.

Counting Considerations to Minimize Seasonal Adjustments

- Coastal or summer recreational areas should be counted during the traditional summer period (Memorial Day to Labor Day). Outside of coastal/recreational areas, most areas can be counted from March to October. Larger MPO areas or commuter-based corridors can be counted most months, but should generally avoid December to February as these are the lowest traveled months, have a number of holidays, and have the most weather-related problems. Winter recreation areas (i.e. Mt. Hood area) should be counted in the December to February timeframe to capture the peak periods. Recreational areas (or routes that travel to or between recreational areas) may require counting on the weekends.
- Road tube count placement is limited to the April to October period because of studded tire damage potential.
- In general, days potentially influenced by state or federal holidays or other significant events (such as local festivals) that may alter normal traffic patterns should be avoided.
- It is also common to avoid Monday and Friday counts when weekday data is desired, as the trip characteristics on these days generally differ from the remainder of the week.
- Consideration should also be given to the presence of generators such as schools and major employers or attractions that experience significant peaks in generated trips that may or may not occur during the other peaks because of shift changes or event scheduling.
- In agricultural areas, truck traffic may be highly seasonal and have a substantial impact on the system. Counts may have to be timed carefully to balance the overall peak months with the harvest periods.
- In the Portland Metro area, while infrequent, there may be times when additional data must be collected to capture the 2nd hour, needed to evaluate the adopted 2-hour OHP

mobility standard. This is generally only necessary when the operational threshold for the second hour of the peak period is lower than the threshold for the first hour of the peak period and the analysis shows the first hour operating below its threshold, but above the threshold for the second hour.

3.3.4 Vehicle Count Locations

Vehicle count locations should be identified in the project workplan/SOW, and should be determined based on the needs of the subject plan or project. For example, planning efforts that are expected to generate potential highway projects within three years will require more detailed counts than a standalone or long-range plans, such as TSP's.

For planning projects it is important to correspond with the local jurisdiction and TPAU/Region Traffic to make sure that count needs cover the system to be analyzed at the appropriate level of detail and address areas of concern. The grant/project manager should meet with TPAU/Region Traffic staff to discuss traffic count requirements after the objective section of the SOW (or project prospectus) is completed as this section provides the context for the plan/project. For Transportation Growth Management (TGM) grants, it is usually more efficient to arrange a meeting with the appropriate TPAU/region staff to go over multiple studies at once. For construction projects, the project team or at least the region traffic engineer/manager, the environmental lead, and/or project leader should be consulted.

While differences of opinion may exist on the number and type of counts versus the available budget, remember that the ultimate goal will be to have enough data to answer the questions, address the needs, and cover the level of detail in the plan/project as described by the project objectives and the local jurisdiction(s). Staff will need to come to an agreement whether the data collection budget and/or the number/type of counts need to change.

The following plan/project-specific count location guidelines do not cover every possibility or combination of elements, but are intended to help generate a reasonable starting point for discussion. The location guidelines are generally laid out in an increasing level of detail.

County Transportation System Plan (TSP)
The arterial and major collector system needs to be documented (counted). It is generally unnecessary to count lower functional class roads as these usually carry very little traffic, and possibly are unpaved unless the county government wants a specific roadway included because of operational concerns. Analysis at the County level is more system-based with a higher emphasis on ADT rather than peak hour and many of the analysis tools require ADT as an input.

- Need to have at least ADT-level count coverage of the arterial and major collectors. Acceptable previously taken counts may exist at the state or local level.
- Major arterial intersections with other arterial and major collector intersections should be counted where operational concerns exist. State highway segments (between major intersections) should use the TSM Unit's vehicle classification data to capture volumes and truck classifications.
- The TSM Unit's ramp volume diagrams (where available) should be used to capture any

free-flow ramp connections.

- County arterials and major collectors should have at least a 48-hour classification tube count performed so truck traffic can be captured and ADT can be calculated.
- Peak period counts should be obtained at signalized intersections, unsignalized highway to highway junctions, and county arterial – highway intersections. If this is a TSP Update, refer to the old TSP to find the critical intersections that should be counted.

City Transportation System Plan (TSP)

The arterial and collector system needs to be documented (counted). It is generally unnecessary to count lower functional classes unless the roadway is area-significant, provides an alternate path for trips to bypass congested areas (as in a parallel local street), or the local government has previously identified operational concerns. Analysis at the City level is more centered on the peak periods and individual facilities/intersections which require more detail.

- Need to have at least ADT-level count coverage of the arterial and collectors. Acceptable previously taken counts may exist at the state or local level.
- Major cities (i.e. MPO's) need to have at least the arterial system counted.
- Medium cities (10,000 – 49,999 pop.) need to have the arterial and representative/significant collectors counted.
- Small cities (<10,000 pop.) need to have the arterial and significant collectors counted.
- Major arterial intersections with other arterial and significant collector intersections should be counted. Peak period counts should be obtained at minor arterial/collector signalized intersections, unsignalized highway to highway junctions, city arterial – highway intersections, and major private development accesses (i.e. regional shopping mall). If this is a TSP Update, refer to the old TSP to find the critical intersections that should be counted.
- Significant collectors extend across the city for a considerable distance, are a direct route, or extend outside the city.
- If multiple signals exist, it may not be necessary to have a count at every one, but a reasonable representation of the system needs to be counted.
- Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Sixteen-hour counts should be obtained at interchange ramp terminals and signalized major arterial intersections. The TSM Unit's ramp volume diagrams (where available) should be used to capture any free-flow ramp connections.
- State highway segments (between major intersections) should use the TSM Unit's vehicle classification data to capture volumes and truck classifications.
- City arterials and collectors should have at least a 48-hour tube count performed so ADT can be calculated. Larger cities may already have this count data.
- If detailed refinement plans and/or actual highway projects are expected out of the TSP within three years and plan to use the TSP data, then the counted major intersections should be 16-hour counts with the lesser unsignalized intersections or access points using peak period counts.

Interchange Area Management Plan (IAMP)

The roadway system needs to be counted within at least a half-mile radius of the interchange.

Analysis at the IAMP level can be close to a project-level of detail (see project section) depending on whether it is standalone or not. If the IAMP is part of a project, then the IAMP should be using the project counts and volumes and no new counts should be necessary unless the counts are very old (greater than three to five years). It may be necessary to obtain a few “check counts” to see if volumes are substantially different before replacing all or most of the counts. If the IAMP is a standalone plan but it is anticipated that a project may occur within three years, then the IAMP needs a project-level count request. If the IAMP is a standalone plan but no project is anticipated within three years:

- Major arterial intersections with other arterial and major collector/collector intersections should be counted.
- Sixteen-hour counts should be obtained at the ramp terminal intersections, other arterial/arterial intersections, or unsignalized intersections that may need to be signalized.
- Peak period counts should be obtained at other existing signalized and unsignalized intersections and unsignalized intersections.
- If multiple signals exist, it is unnecessary to have 16-hour counts at every one. Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Most, if not all, driveway accesses should be counted with peak period counts as many of these will be rerouted to new connections.
- State highway segments (between major intersections) should use the TSM Unit’s vehicle classification data to capture volumes and truck classifications.
- The TSM Unit’s ramp volume diagrams (where available) should be used to capture any free-flow ramp connections.

Refinement/Management/Facility Plans

The arterial and collector system needs to be counted within the defined study area limits. It is generally not necessary to count lower functional classes unless the roadway is area-significant, provides an alternate path for trips to bypass congested areas (as in a parallel local street), or the local government has previously identified operational concerns. If it is anticipated that a project may occur within three years, then a project-level count request is needed. If no project is anticipated within three years:

- Major arterial intersections with other arterial and major collector/collector intersections should be counted.
- Facilities parallel to the subject arterial should be counted.
- Longer roadway sections without intersections should use road tube counts.
- Sixteen-hour counts should be obtained at signalized intersections and major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic or where larger scale improvements may be needed.
- Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Unsignalized intersections or major accesses should be counted with peak period counts.
- If an Interstate Highway or statewide expressway exists in the study area, the mainline shall be counted by direction between interchanges in addition to any interchange ramp terminals. Road tube counts may be necessary to capture movements on ramps or connections.

Local Street Network (LSN) /Downtown Plan
These kinds of plans are generally trying to identify new roadway or multimodal connections to control congestion on the state highway or make limited improvements in the downtown area. The arterial and collector system need to be counted. It is generally not necessary to count lower functional classes unless the roadway is the only access to a neighborhood, provides an alternate path for trips to bypass congested areas (as in a parallel local street), or the local government has previously identified operational concerns. Larger numbers of peak period counts may be necessary with a few 16-hour counts at major intersections.

- Major arterial intersections with other arterial and collector intersections should be counted.
- Sixteen-hour counts should be obtained at signalized intersections and major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic or where larger scale improvements may be needed.
- If multiple signals exist, it is unnecessary to have 16-hour counts at each one. Each major roadway should have truck traffic captured on it in at least one location.
- Unsignalized intersections or major accesses should be counted with peak period counts.
- Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- State highway segments (between major intersections) should use the TSM Unit's vehicle classification data to capture volumes and truck classifications.
- The TSM Unit's ramp volume diagrams should be used to capture any free-flow ramp connections.

Pedestrian Plans/ Trail Plans
Generally, counts are only needed if the state highway system will be affected by removing or narrowing through travel lanes or if new crossings are to be added. Count requirements in the lane reduction areas should follow the LSN/Downtown Plan recommendations above.

Plans with proposed mid-block trail crossings of state highways or local arterials should have a 48-hour classification road tube count performed at the crossing location. For plans with existing pedestrian crossings (formally defined or not) where the number of crossing pedestrians is desired, the crossing count should be replaced with a 16-hour video classification count.

Traffic Impact Studies (TIS)
For TIS's, the analysis area and study intersections are typically selected from estimates of anticipated impacts from added traffic based on site trip generation and distribution, and existing intersection operations. Count requests need to be developed with the guidance of the Region Access Management Engineer or appropriate region staff and the Development Review Guidelines.

- Sixteen-hour counts should be obtained at major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic; obtain the basis for signal warrants, or where larger scale improvements may be needed.
- Signalized intersections may use a 16-hour count or a peak period count depending on the

particular study area.

- If multiple signals exist, it may not be necessary to have 16-hour counts at each one. Bracketing peak period counts with 16-hour counts is an acceptable practice. Each major roadway should have truck traffic captured on it in at least one location.
- Unsignalized intersections and accesses should be counted with peak period counts.
- The Interstate/expressway/highway mainline shall be counted by direction in addition to any interchange ramp terminals. Road tube counts may be necessary to capture movements on ramps or connections.

Construction Projects
For most other project types (modernization, safety, operations, etc) the analysis area and study intersections are selected by considering the problem that is being addressed by the project and the information that is required to fully assess the problem and propose appropriate solutions. Project analysis is needed to support roadway and intersection control improvements, pavement and bridge design, air quality, and noise mitigation. Larger projects especially those with required environmental studies (such as noise and air quality) may require many full 16-hour classification counts.

- Sixteen-hour classification counts should be obtained at signalized intersections and major unsignalized intersections (i.e., ramp terminals, four-way stops) to capture truck traffic or where larger scale improvements may be needed.
- Truck classification data needs to be captured on each roadway segment in the study area.
- Minor unsignalized intersections and accesses should be counted with peak period counts.
- Significant driveway accesses should be counted as many of these will be rerouted to new connections.
- If an Interstate Highway or grade-separated highway exists in the study area, the mainline shall be counted by direction between interchanges in addition to any interchange ramp terminals. Road tube counts may be necessary to capture movements on ramps or connections.

3.3.5 Count Requests

When ordering counts, the request must contain the name of the contact person (requestor), the person to whom the data will be sent, the locations, time periods, dates, types of counts and collection methods must be clearly communicated to those conducting the counts. Count requests should group different count types (classification, peak period, road tube) separately for clarity. The count request also lists any special requests, count intervals, count time windows (start and finish dates) and an expense account charge number. A couple examples of a special request would be counting only on a specific day or counting certain intersections at the same time at complex intersections.

A map showing the count locations, durations and other special requirements should also be provided to help eliminate misunderstandings since often times the text is separated from the map. Please keep in mind that the field counting staff usually only has the map in hand so all pertinent information (count locations, durations, types, intervals, and special requests) needs to be on the map. See the sample count request memo and map in Appendix D.

When ordering intersection counts, be sure to specify the duration and type for each location. Fifteen-minute intervals must be specified for at least the standard morning, noon and evening peak periods in 16-hour counts. Peak period counts should always be done in 15-minute intervals. It is not required, but very helpful if 48-hour road tube counts are counted in 15-minute intervals as well.

Specify the latest acceptable date by which the count is needed for analysis. Keep in mind that it can take at least five weeks from the date of the request date to get the count scheduled (not including weather restrictions) and then another three to four weeks to have the count processed, recorded and distributed. Therefore, counts need to be requested about nine weeks ahead (or more if weather is a factor) of when they will be needed for the analysis work.

Count requests should be sent to the Region Traffic Manager and a courtesy copy (cc) to the TSM Unit to alert them that these counts are requested and need to be scheduled. If Region does not have the resources to do the count, it will be contracted out to a consultant. Region requires the same count request information/format for a consultant to do the work. In addition, most of the raw counts are processed into a readable format in the TSM Unit before being released to the requestor. The TSM Unit needs know what counts are out there so staff resources can be allocated. The TSM Unit coordinates the counting schedules of all Region traffic counting staff. Keeping the TSM Unit in the loop allows for these counts to be added to the count databases which can limit counting needs by others and help limit project delays.

3.4 Travel Time Surveys

Travel time surveys measure the duration of time taken for a vehicle to travel from one point to another along a designated route, and are often used to quantify congestion over a corridor. The data collected from travel time surveys works well with statistical analysis, and the results are often more easily understood by the public than other methods used for measuring congestion.

3.4.1 Data Collection

The most common method used for collection of this data involves the use of a "floating car." The elapsed time is measured from a car driven along the designated route maintaining an average travel speed relative to other cars on the roadway. Other methods include license plate matching and the use of various intelligent transportation system technologies. Travel time data is collected at the beginning and end of a designated route, and can be collected between predetermined points along the route as well, depending on the level of information desired.

When collecting travel time data, all runs should be taken under good weather conditions and during a time that is representative of the time period of interest for the study. Also, be sure to distribute the travel time runs over several days of the week and over multiple weeks (collections over weeks, still need to be representative of the time period of interest for the study).

3.4.2 Applications

The data collected during travel time surveys can be used to measure congestion in several ways, but the most common application involves before and after comparisons, usually measuring the changes in performance resulting from a transportation improvement. Because travel time surveys can not be used to predict changes in performance, a sample of data must be collected for every scenario of interest under actual conditions or during the actual time period of interest.

Other common uses of travel time data include the calculation of average travel speed, which is used to define the level of service for urban streets, and the calculation of delay by comparing measured travel speeds to desired travel speeds. Additionally, travel time data can be used for calibration and validation of micro-simulation models.

Travel time data can also be used in a cumulative analysis procedure. If travel time data is collected for all roadways in the study area, the shortest time method can be used to assign traffic volumes to the roadway network.

3.4.3 National Cooperative Highway Research Program Report 398

National Cooperative Highway Research Program (NCHRP) Report 398[1] represents a comprehensive source of information regarding travel time surveys. It provides guidance on the uses of travel time data, includes comparisons of travel time surveys to other congestion

[1] Lomax, T., Turner, S., and Shunk, G. "Quantifying Congestion, Volumes 1 & 2", NCHRP Report 398, TRB, National Research Council, National Academy Press, Washington, D.C., 1997.

measuring techniques, identifies the strengths and weaknesses of travel time surveys and provides recommendations for proper data collection and analysis. All travel time surveys conducted should be performed in accordance with the recommended practices in this report.

When collecting travel time data, all runs should be taken under good weather conditions and during a time that is representative of the time period of interest for the study. Also, be sure to distribute the travel time runs over several days of the week and over multiple weeks. The number of runs required to obtain a statistically representative sample size with a good confidence level can be determined from the following tables from NCHRP Report 398.

Exhibit 3-3 Suggested Sample Sizes for Arterial Streets

Arterial Street Signal Density Group	Number of Test Sections	Average Coefficient of Variation % [a]	Minimum Number of Runs – 90% Confidence, 10% Relative Error[b]
Low – less than 3 signals per mile	320	9%	2 (6) runs[c]
Medium – 3 to 6 signals per mile	433	12%	4 (6) runs[c]
High – greater than 6 signals per mile	109	15%	6 runs

[a] Coefficient of variation (c.v.) = mean speed (mph)/speed standard deviation (mph).
[b] Sample size for 90% confidence level, 10% relative error was used as an example. Precision levels should be set by local agencies in accordance with uses of data and desired precision.
[c] Four runs considered practical minimum on arterial streets.
Source: NCHRP Project 7-13, Reference (2).

Exhibit 3-4 Suggested Sample Sizes for Freeways/Expressways

ADT per Lane Stratum	Average Coefficient of Variation %[a]	Minimum Number of Runs - 90%Confidence, 10% Relative Error[b]
Low – less than 15,000 ADT per lane	9%	2 (5) runs[c]
Medium – 15,000 – 20,000 ADT per lane	11%	3 (5) runs[c]
High – greater than 20,000 ADT per land	17%	8 runs

[a] Coefficient of variation (c.v.) = mean speed (mph)/speed standard deviation (mph).
[b] Sample size for 90% confidence level, 10% relative error was used as an example. Precision levels should be set by local agencies in accordance with uses of data and desired precision.
[c] Five runs considered practical minimum on freeways.
Source: NCHRP Project 7-13, Reference (2).

3.5 Saturation Flow Rate Studies

The saturation flow rate is a critical component in the analysis of signalized intersection capacity and can be defined as the flow in vehicles per hour that can be accommodated by a lane group assuming that the green phase is displayed 100 percent of the time. Saturation flow rate data is collected on an ongoing basis. Copies of saturation flow rate studies should be sent to TPAU so that the work on developing the default values can continually be improved. To date, this research has shown that a default saturation flow rate of 1750 passenger cars per hour of green per lane is appropriate in most cases, with some exceptions as noted below[2]. See the Transportation Analysis webpage on the TDD Planning Section website for the latest information on saturation flow rates.

3.5.1 Field Measurements of Saturation Flow Rates

Field measurement of saturation flow rates is preferred over estimation. Using default values and adjustment factors will produce more accurate results and does not require further modification. If possible, saturation flow rates should be collected at no less than one major intersection on each main study area roadway. When using these values in analysis be sure to set all of the adjustment factors to 1.0. The measurement of the saturation flow rate in the field shall be in accordance with methodology described in Appendix H in Chapter 16 of the Highway Capacity Manual 2000 (*HCM*) and submitted on the *HCM* Field Saturation Flow Rate Study Worksheet.

Once the field saturated flow rate is obtained, the ideal (unadjusted) saturation flow rate should be back-calculated by applying adjustment factors to account for the influence of lane widths, heavy vehicles, approach grades, on-street parking, bus stops, area type, lane utilization, turning movements and bicycle and pedestrian conflicts. Heavy vehicles, parking maneuvers, turning movements, and bicycle and pedestrian conflicts must be collected during the same period as the field saturation flow study to be able to back- calculate an accurate value.

3.5.2 Default Values for Base Saturation Flow Rates

Except in larger urban areas, field conditions generally do not allow the *HCM* saturation flow study procedures to be met. A roadway approach may not have long enough queues during the study or intersection spacing may be so tight that long enough queues without gaps are not possible. In these cases, a default ideal unadjusted saturation flow is determined as follows:

- Outside of the Portland, Salem and Eugene MPO urban areas the unadjusted saturation flow rate is 1750 passenger cars per hour of green per lane (pcphgl).
- Inside the Portland, Salem and Eugene MPO urban growth boundaries an unadjusted saturation flow rate of 1900 pcphgl may be used, unless one or more of the following conditions is present, in which case 1750 pcphgl shall be used. Conditions indicating use of lower base saturation flow rate inside urban growth boundaries:
    - On-street parking
    - Greater than 5% trucks

[2] Based on NCHRP Report 599, Default Values for Highway Capacity and Level of Service Analysis, TRB, 2008, and Metropolitan Statistical Area (MSA) population estimates from the U.S. Census Bureau as of July 1, 2007.

- Roadways intersect at severe skew angle (i.e., greater than 20 degrees off perpendicular)
- One or more driveway approach(es) with a combined volume in excess of 5 vph, are present downstream of the intersection within the functional area (see Section 7-3) or upstream within the length of the standing queue
- Poor signal spacing or observed queue spillbacks between signals during the peak hour
- Less than 12-foot travel lanes

The ideal (unadjusted) saturation flow rate is converted to an actual flow rate by applying adjustment factors to account for the influence of lane widths, heavy vehicles, approach grades, on-street parking, bus stops, area type, lane utilization, turning movements and bicycle and pedestrian conflicts. Theoretically, once adjusted, the result would be equivalent to the field measured value.

3.6 Crash Data

Crash data can come from a variety of sources, and is useful for identifying problem areas of the highway experiencing an above-average frequency of crashes or reoccurring crash patterns. The analysis procedures that use this data are described in Chapters 5-7, while the data itself is described below.

3.6.1 Safety Priority Index System

The Safety Priority Index System (SPIS) is a method developed by ODOT in 1986 to identify hazardous locations on state highways. Major revisions in the reporting were made in 1999. The SPIS score is based on three years of crash data, and considers crash frequency, crash rate and crash severity. ODOT bases its SPIS on 0.10-mile segments to account for variances in how crash locations are reported. To become a SPIS site, a location must meet one of the following criteria:

- Three or more crashes have occurred at the same location over the previous three years.
- One or more fatal crashes have occurred at the same location over the previous three years.

The use of this information is discussed in Chapters 5-7, and the documentation on how the SPIS is calculated can be found at the Safety Priority Index System website.

3.6.2 Sources of Crash Data

Oregon Motor Vehicle Traffic Crash Database

ODOT's Crash Analysis and Reporting (CAR) Unit provides motor vehicle crash data through database creation, maintenance and quality assurance, information and reports and limited database access. Crash data since 1985 is maintained at all times. Vehicle crashes include those coded for city streets, county roads and state highways. The CAR Unit website offers a variety of publications containing information on monthly and annual crash summaries.

Depending on the level of analysis needed, there are various reports that can be obtained. It should be noted that even though this database often represents the most current data available, data for a given year is typically not available until at least 6 months into the following year.

Detailed information for individual crashes can also be obtained by contacting the CAR Unit and specifying the segment of highway (roadway) and time period of interest. Because crashes are reported by milepoint, a concern or cause may be located just outside of the analysis area. Therefore, the area of requested crash data should always be greater than the area of analysis. Internal ODOT users can access the crash data reports via the Intranet Transviewer application, except for non-state highway crashes, which are obtained from the CAR Unit. While several years of data may be available for any given roadway segment, it is common practice to analyze only the most recent 3 to 5 years of data as factors such as traffic volumes, environmental conditions and roadway characteristics may change with time.

Although there are other sources of information, such as police departments and local groups that collect information, the ODOT CAR Unit data is the standard source. Oftentimes these other sources include reporting calls, not the actual investigation/report, and groups often include near-misses and/or don't have all the facts, so the report is erroneous. The CAR data has the checks and balances built into the data collection that matches "both sides of the reports" for a crash, treating all areas the same so comparisons can be made. This is also the database used to calculate all the comparison rates published in the crash rate books.

Accident Summary Database

Produced annually since 1990, this database/software combination for use on a desktop computer is useful to generate quick summary reports that are often sufficient to answer questions when there is not time to do a detailed analysis. The accident summary database uses three years of crash data, the middle year traffic volume estimate and the annual SPIS numbers to generate a report that includes an estimated crash rate.

TransGIS Mapping Tool

TransGIS is a mapping tool that provides a graphical display of many types of safety, volume and crash data on a state map. The user can choose the information that is displayed, and can zoom into the map to increase detail as well as display city and county maps behind this data.

Collision Diagrams

The CAR Unit can create a crash diagram that depicts the crashes on a given roadway section. However, this is typically used to evaluate operational issues, not analytical ones.

Crash Rate Tables

Crash Rate Tables have been published annually by the CAR Unit since 1948. Tables in the front of the book list statewide crash rates for several categories of the State Highway System. More tables list the crash rates for selected sections of each state highway, as well as a rural/urban break out. Additional tables list intersection crash data and fatal crash data. The use of these tables is discussed further in Chapter 5.

Statewide Transportation Improvement Program – Safety Investment Program (STIP – SIP)

The Statewide Transportation Improvement Program - Safety Investment Program (STIP-SIP) is a process to selectively place safety countermeasures on roadways with a history of fatal and serious injury crashes, and perform minimal safety upgrades on roadways with low fatality and severe injury crash history. Because of its operational nature, this information is typically furnished by either Region Traffic or the TRS. The use of the STIP-SIP information in identification of crash patterns is discussed in Chapter 5.

3.7 Data Resources from ODOT

ODOT, through its Transportation Data Section, collects substantial data for system inventory, volumes and crash information that can be used for the purpose of conducting traffic studies. Because much of this data is collected and processed by different units within the Department, clear and frequent communication between units regarding what is desired and what is available is critical for ensuring these resources are readily accessible. Furthermore, good communication between units will help to obtain the right data in a timely manner, which is important for maintaining project schedules. Coordinate with the appropriate ODOT department or staff as noted below.

3.7.1 Timelines

- Traffic count request letters should be submitted at least five weeks before counts need to be taken. Allow one additional month for the Transportation Systems Monitoring Unit to process the counts, total lead time nine weeks.
- Crash data requests from the CAR Unit should be made two to four weeks before the information is needed.
- Allow about one month to gather remaining inventory data from the Intranet Transviewer application, TransGIS (Internet or Intranet), or the Roadway Inventory and Classification Services (RICS) Unit.

3.7.2 Personnel

- Transportation Systems Monitoring or Region Traffic staff provide count data.
- RICS Unit staff provide inventory log data; Region as, necessary.
- Region Traffic staff provide signal timing sheets.
- Crash Analysis and Reporting Unit staff provide crash data.
- The project analyst will be responsible for all other system inventory data.

3.7.3 Product

Typical project area inventories may include:

- Project area map
- Photos for use during the project
- Complete record of geometric and operational data for each study area roadway
- Copies of all vehicle count surveys taken
- Travel time survey data, where applicable
- Saturation flow rate data and calculations
- Crash data

4 DEVELOPING DESIGN HOUR VOLUMES

4.1 Purpose

DHVs are used for ODOT planning and project level analyses. The DHV is defined as the future year 30 HV. The following procedure outlines the development of the DHV for a single intersection based on the application of seasonal factors and growth rates to manual counts.

Daily traffic count volumes cannot be used alone for design or operational analysis of transportation projects. This chapter will outline the procedure for developing the DHVs used for ODOT planning and project level analysis. Topics covered include:

- General Considerations
- Peak Hour Selection
- Seasonal Factors
- Volume Development for Sketch Planning
- Forecasting
- Comprehensive Example

4.2 <u>General Considerations</u>

4.2.1 Rounding

The 30 HV or DHV need to be rounded before the network is balanced. The traffic volumes are not that precise to go down to one vehicle, especially beyond the existing year. Balancing the network is easier if the network is not down to the individual vehicle. Round volumes to the nearest five for the existing, build year and any short-term future years. Twenty-year future volumes can either be rounded to the nearest five or ten vehicles. Volumes less than five vehicles should use the “<5” symbol instead of using zero.

4.2.2 Need for Balancing

The 30 HV and the DHV networks need to be balanced. Balancing is simply, “what goes into an intersection or segment needs to come out.” Without balancing, it is possible to have two intersections with nothing between them with the volume that leaves one intersection and enters the next one be 200 vph or more different. Interstates and expressways with interchanges and no accesses need to balance perfectly from one intersection or interchange to another. Roadways with accesses probably will not balance perfectly, but should be consistent from intersection to intersection.

The timing of the traffic counts can help determine how easy a network is to balance. Counts that are spread throughout the allowable three-year span taken at different time of the year will be harder to balance than counts all taken on the same day or within a week of each other.

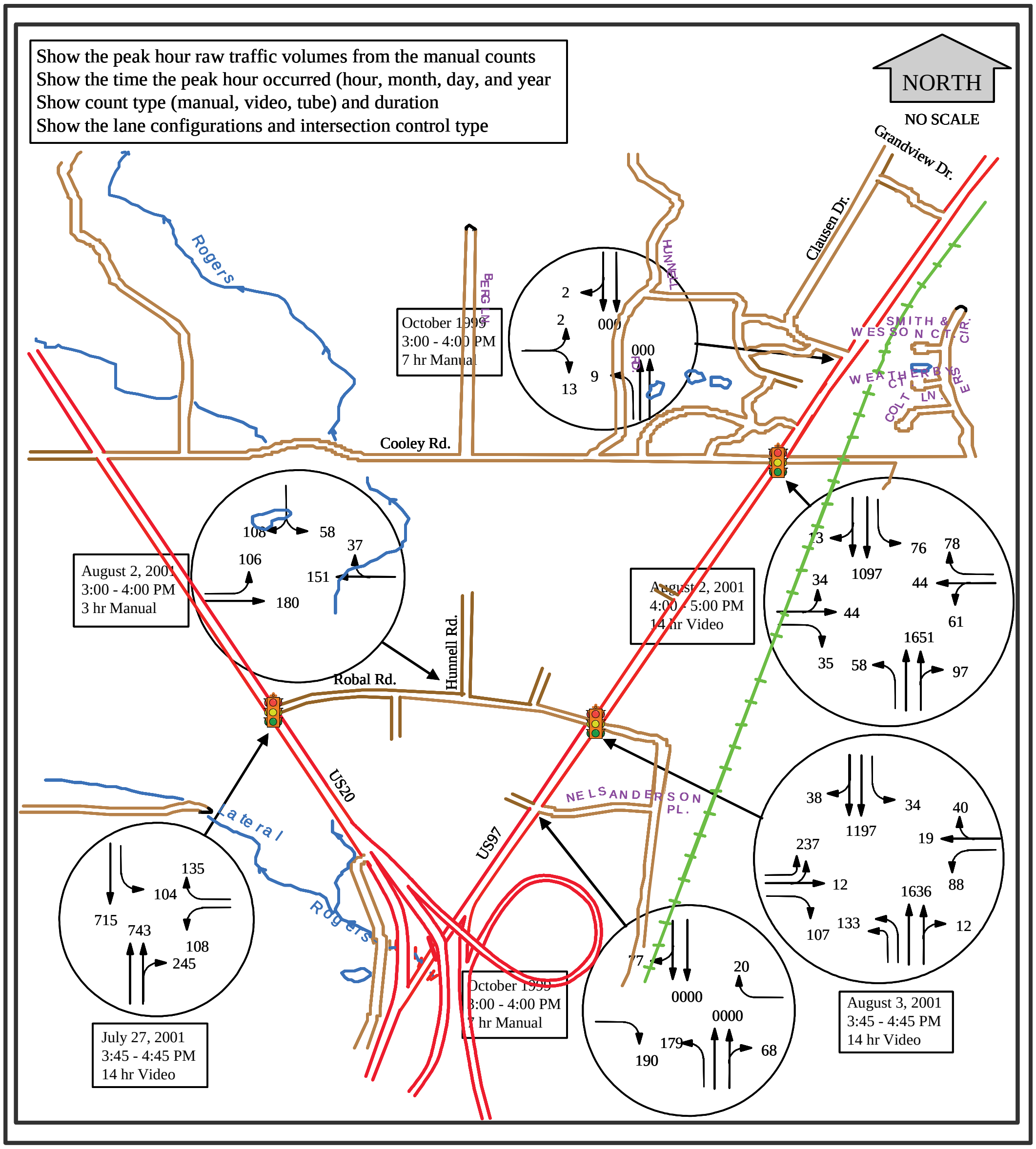
4.2.3 Documentation

It is critical that after every step in the 30 HV and DHV processes that all of the assumptions and factors are carefully documented, preferably on the graphical figures themselves. Seasonal adjustments, ATR 30 HV adjustments, yearly growth factors, 20- year growth factors, ATR’s used, peak hour assumed are some of the items that need to be documented. If all is documented then anyone can easily review the work or pick up on it quickly without questioning what the assumptions were. The documentation figures will eventually end up in the final report or in the technical appendix. The volume documentation should include:

- Figure showing raw traffic volumes with hour, month, day and year that the peak hour occurred. Also show the lane configurations and the intersection control type. See Exhibit 4-1 and Exhibit 4-2
- Figure showing raw traffic volumes for the system peak hour. See Exhibit 4-3 and Exhibit 4-4. Figure showing unbalanced base year 30 HV. Show any yearly growth factors to adjust counts to base year plus any seasonal factors used. See Exhibit 4-5 and Exhibit 4-6.
- Figure showing balanced base year 30 HV. See Exhibit 4-7 and Exhibit 4-8.
- Figure showing balanced future year DHV. Note on the figure how future volumes were developed. If historic trends were used, cite the source. If the cumulative method was

used, include a land use map, information that documents trip generation and through movement growth. If a model was used, attach the base and future year model runs. See Exhibit 4-9 and Exhibit 4-10.

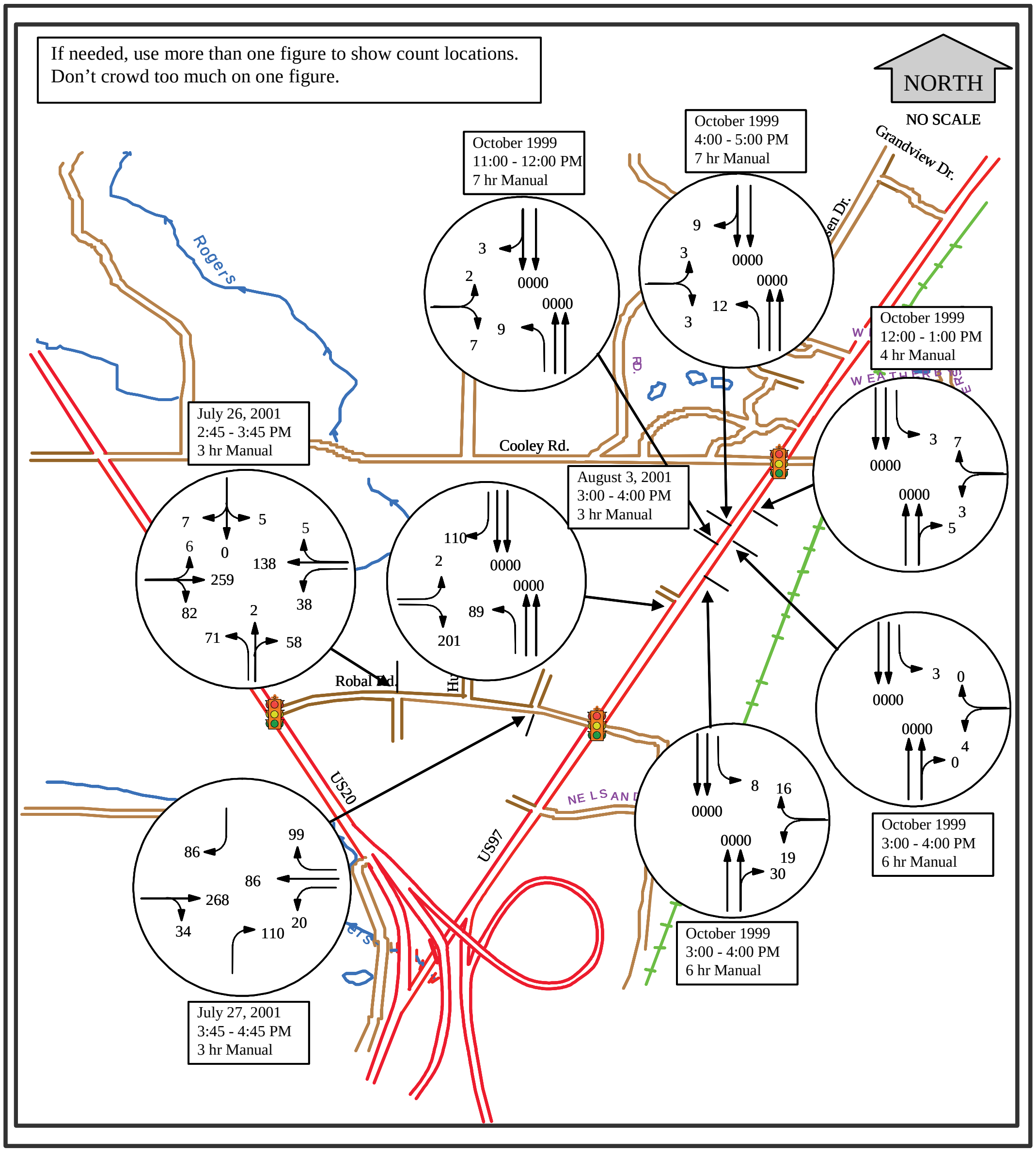
Exhibit 4-1 Raw Traffic Volumes



OREGON DEPARTMENT OF TRANSPORTATION T P A U **TRANSPORTATION PLANNING ANALYSIS UNIT**

Project Title **Raw Count Data**	**File : .ppt**	**Prepared By:**	FIGURE A1
	Date :	**Reviewed By:**	

Exhibit 4-2 Raw Traffic Volumes (Sheet 2)



OREGON DEPARTMENT OF TRANSPORTATION T P A U **TRANSPORTATION PLANNING ANALYSIS UNIT**

Project Title **Raw Count Data**	**File : .ppt**	**Prepared By:**	FIGURE A2
	Date :	**Reviewed By:**	

Exhibit 4-3 Raw Traffic Volumes During System Peak Hour

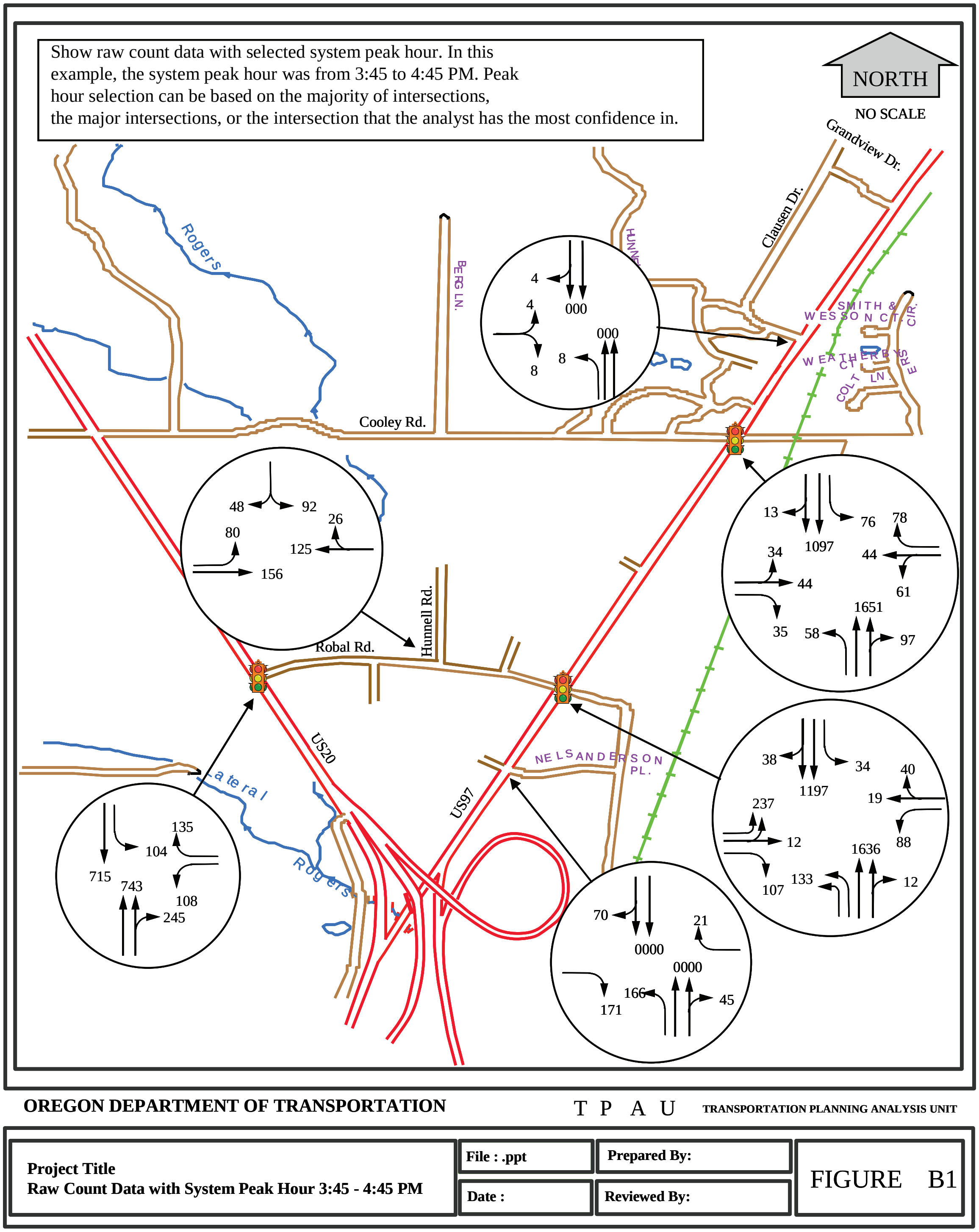
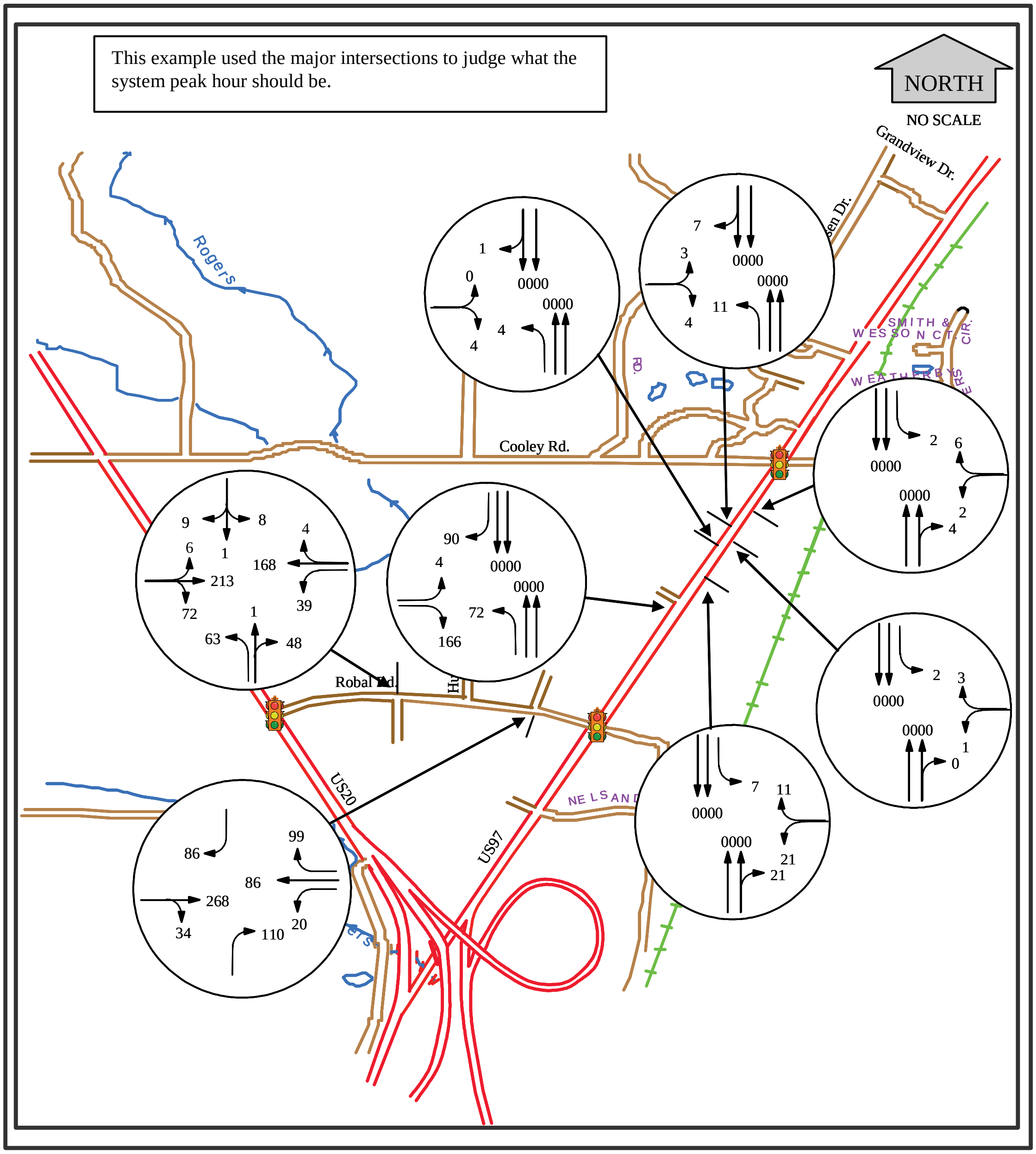


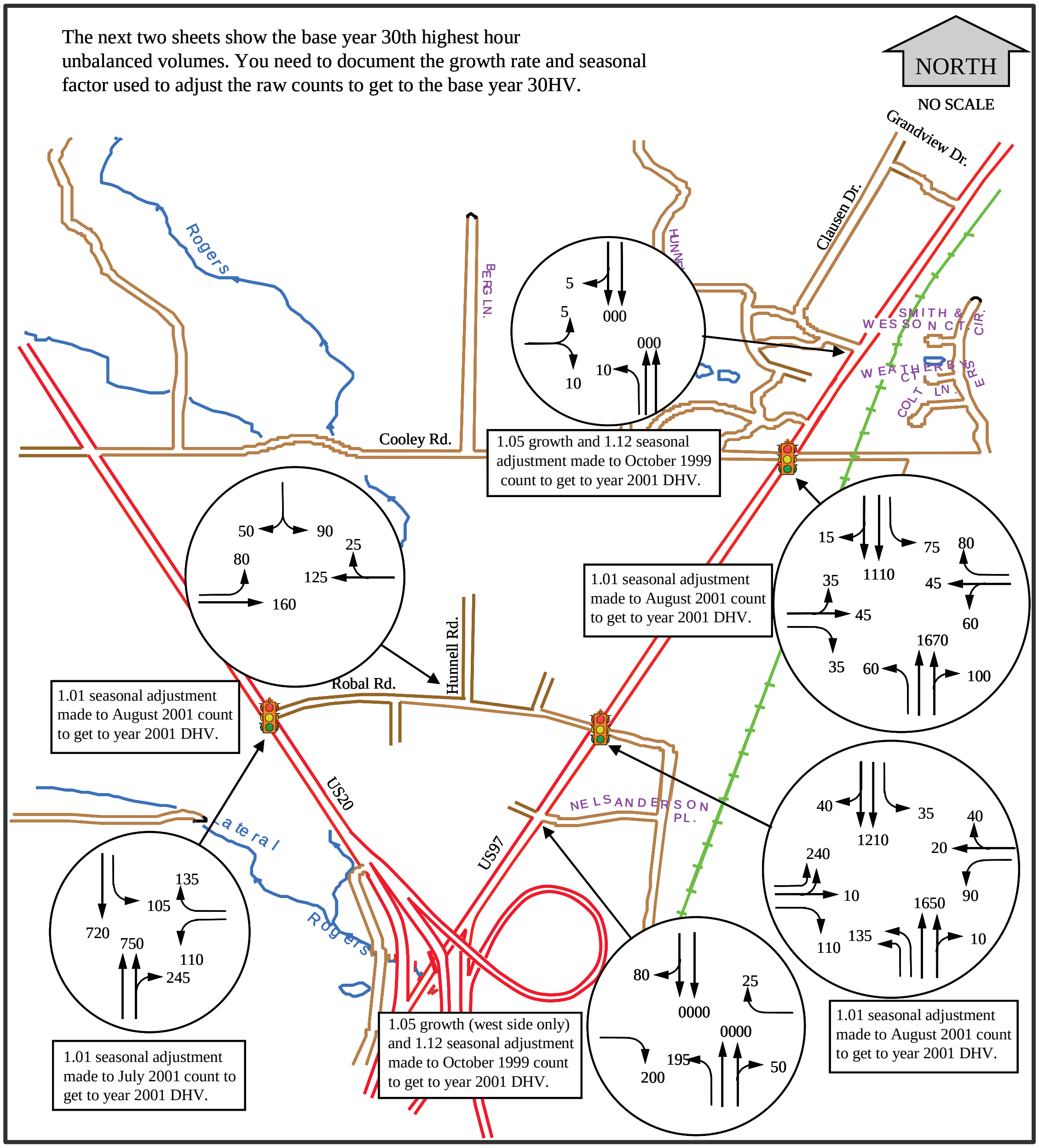
Exhibit 4-4 Raw Traffic Volumes During System Peak Hour (Sheet 2)



OREGON DEPARTMENT OF TRANSPORTATION T P A U **TRANSPORTATION PLANNING ANALYSIS UNIT**

Project Title **Raw Count Data with System Peak Hour 3:45 - 4:45 PM**	**File : .ppt**	**Prepared By:**	FIGURE B2
	Date :	**Reviewed By:**	

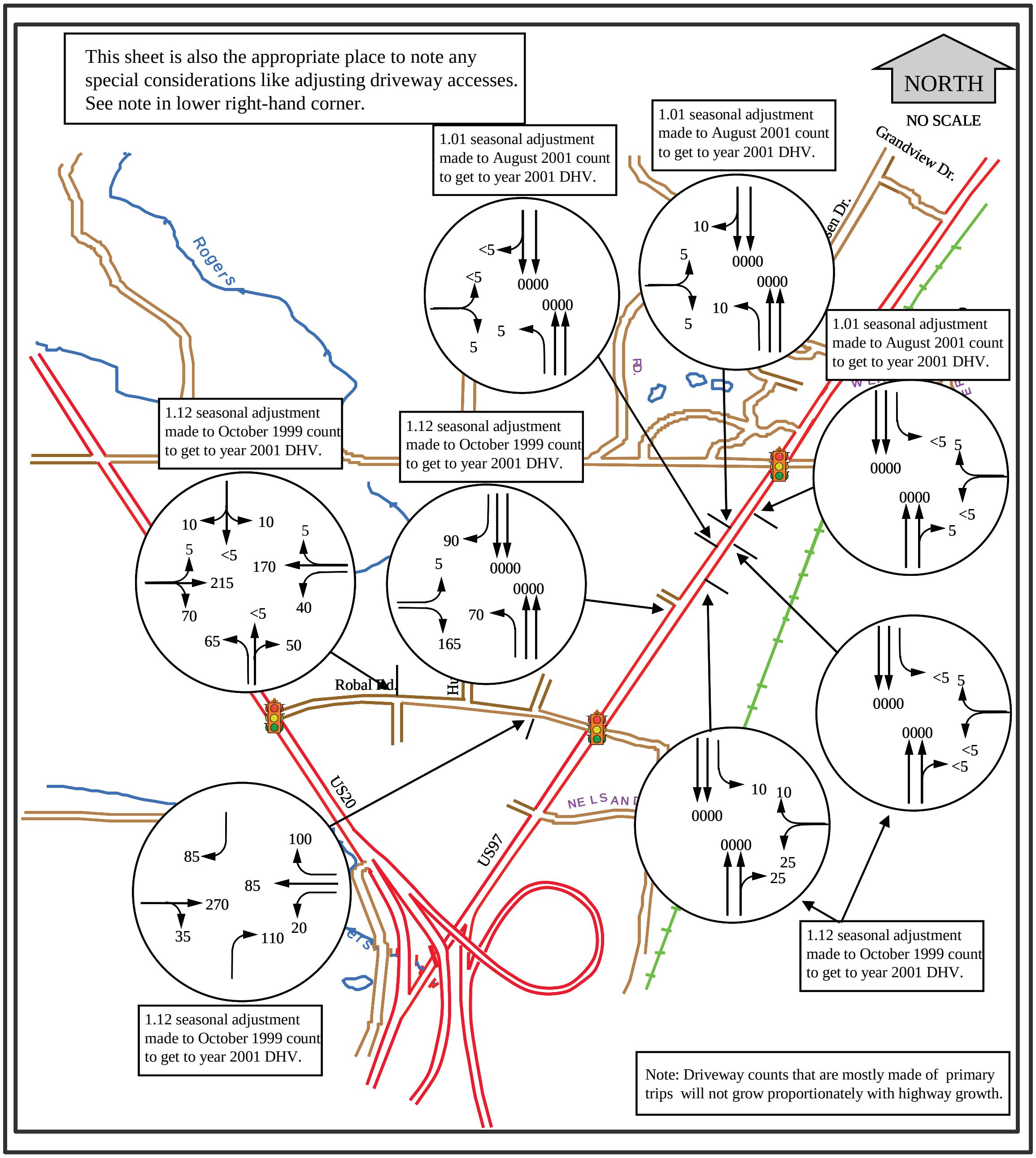
Exhibit 4-5 Base Year 30th Highest Hour Volumes (Unbalanced)



OREGON DEPARTMENT OF TRANSPORTATION — T P A U — TRANSPORTATION PLANNING ANALYSIS UNIT

Project Title Unbalanced Base Year 30HV	File : .ppt	Prepared By:	FIGURE C1
	Date :	Reviewed By:	

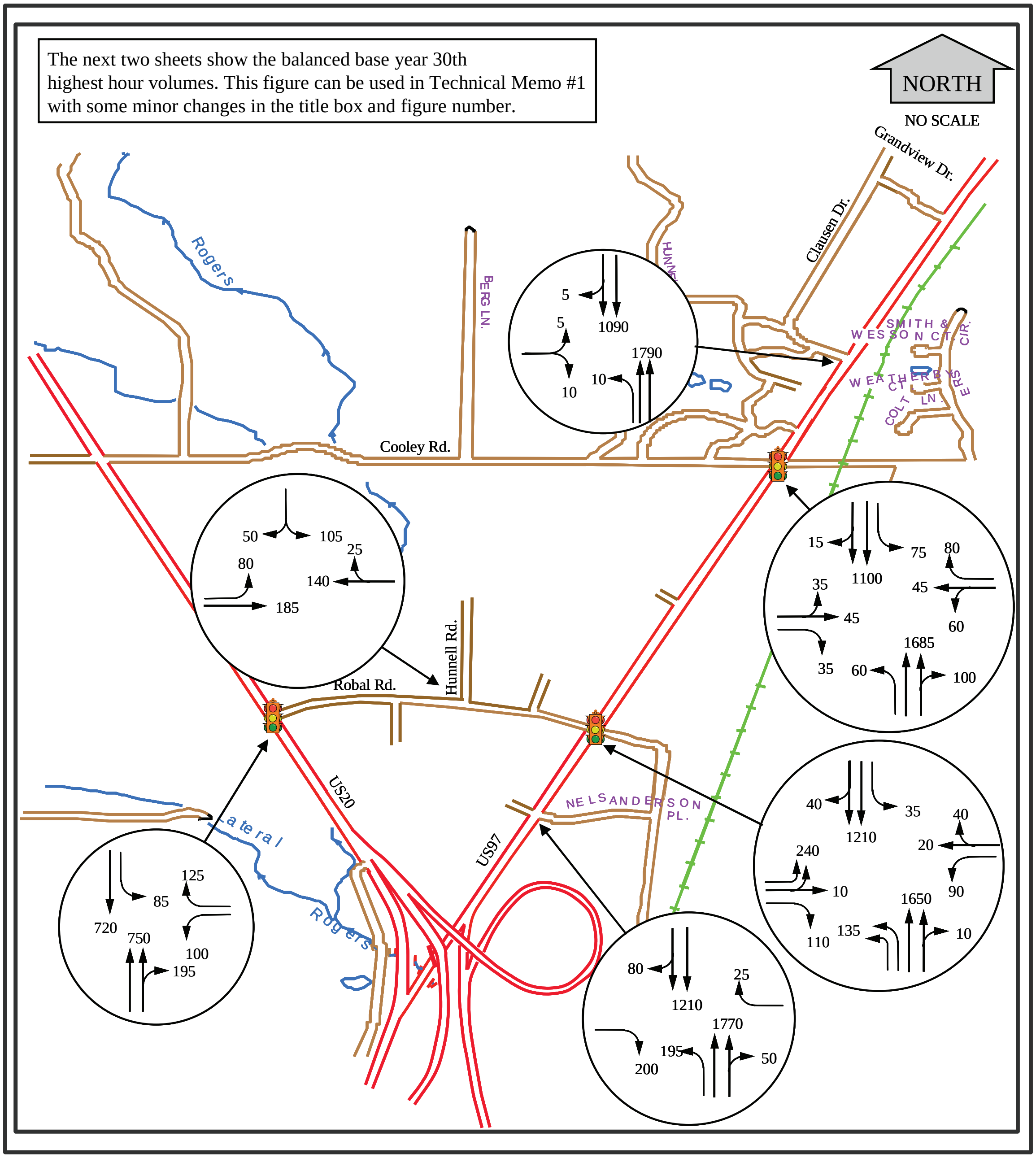
Exhibit 4-6 Base Year 30th Highest Hour Volumes (Unbalanced) (Sheet 2)



OREGON DEPARTMENT OF TRANSPORTATION T P A U **TRANSPORTATION PLANNING ANALYSIS UNIT**

Project Title **Unbalanced Base Year 30HV**	**File : .ppt**	**Prepared By:**	FIGURE C2
	Date :	**Reviewed By:**	

Exhibit 4-7 Balanced Base Year 30th Highest Hour Volumes

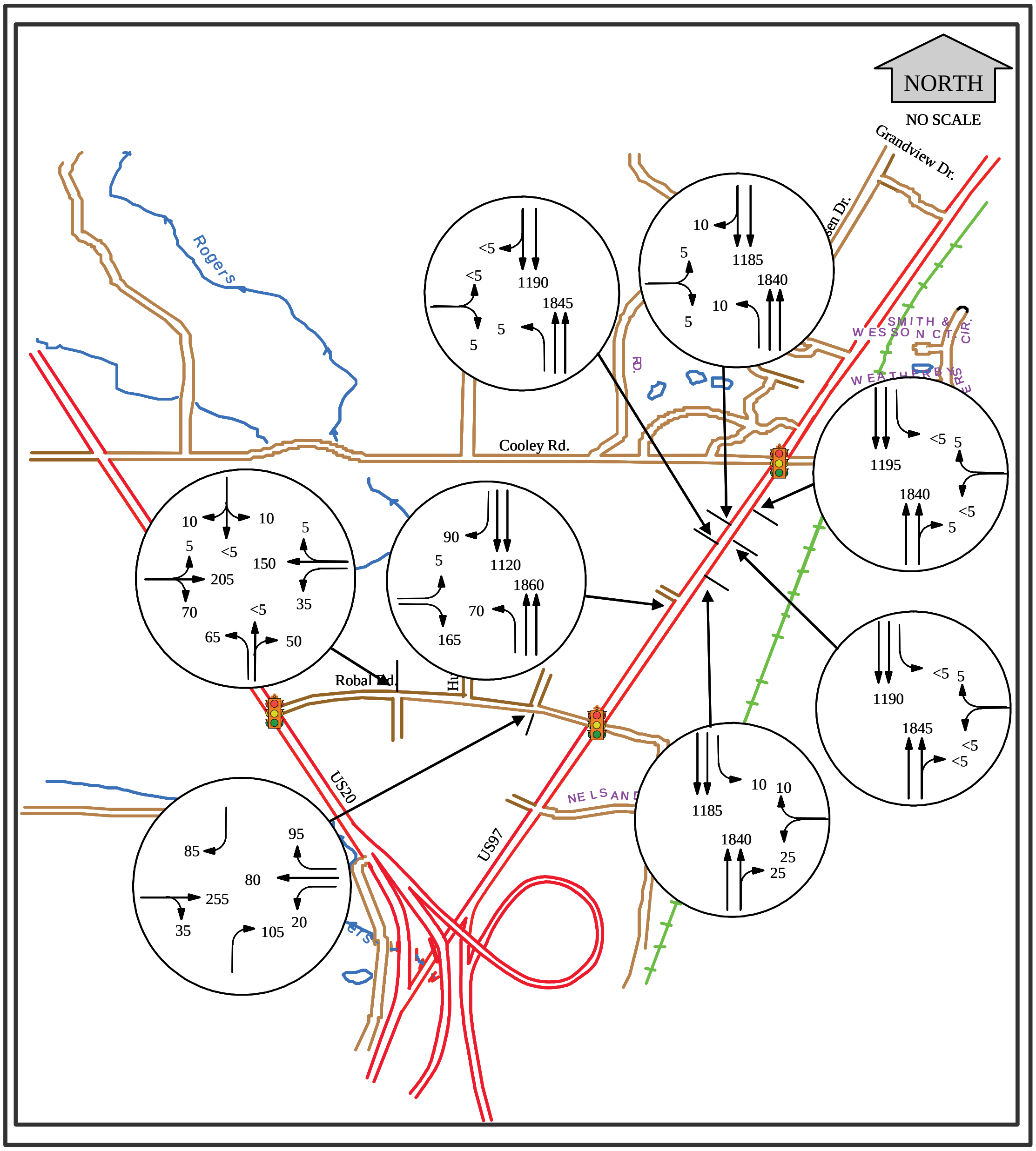


OREGON DEPARTMENT OF TRANSPORTATION

T P A U TRANSPORTATION PLANNING ANALYSIS UNIT

Project Title **Balanced Base Year 30HV**	**File : .ppt** **Date :**	**Prepared By:** **Reviewed By:**	FIGURE D1

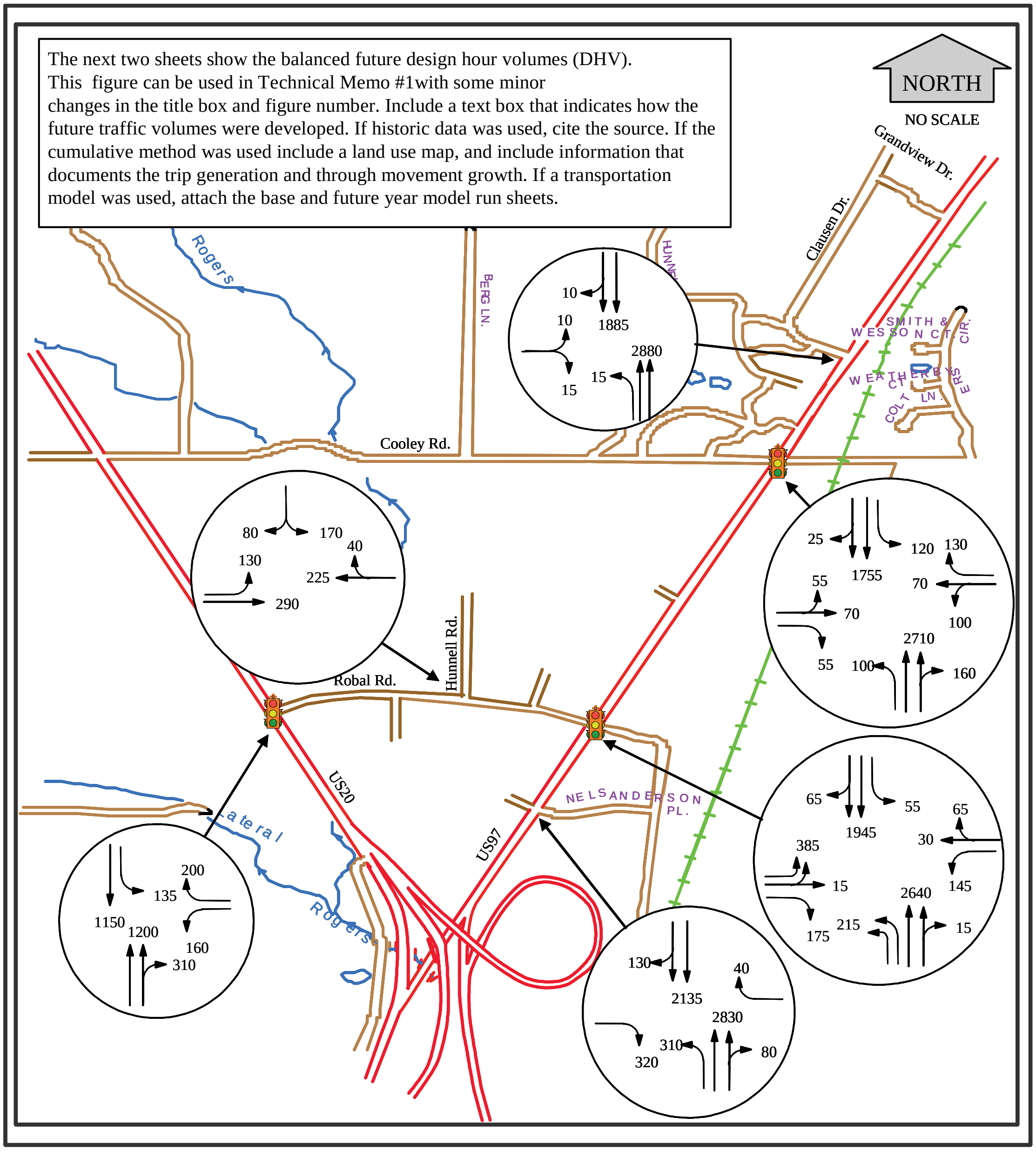
Exhibit 4-8 Balanced Base Year 30th Highest Hour Volumes (Sheet 2)



OREGON DEPARTMENT OF TRANSPORTATION T P A U **TRANSPORTATION PLANNING ANALYSIS UNIT**

Project Title **Balanced Base Year 30HV**	**File : .ppt**	**Prepared By:**	FIGURE D2
	Date :	**Reviewed By:**	

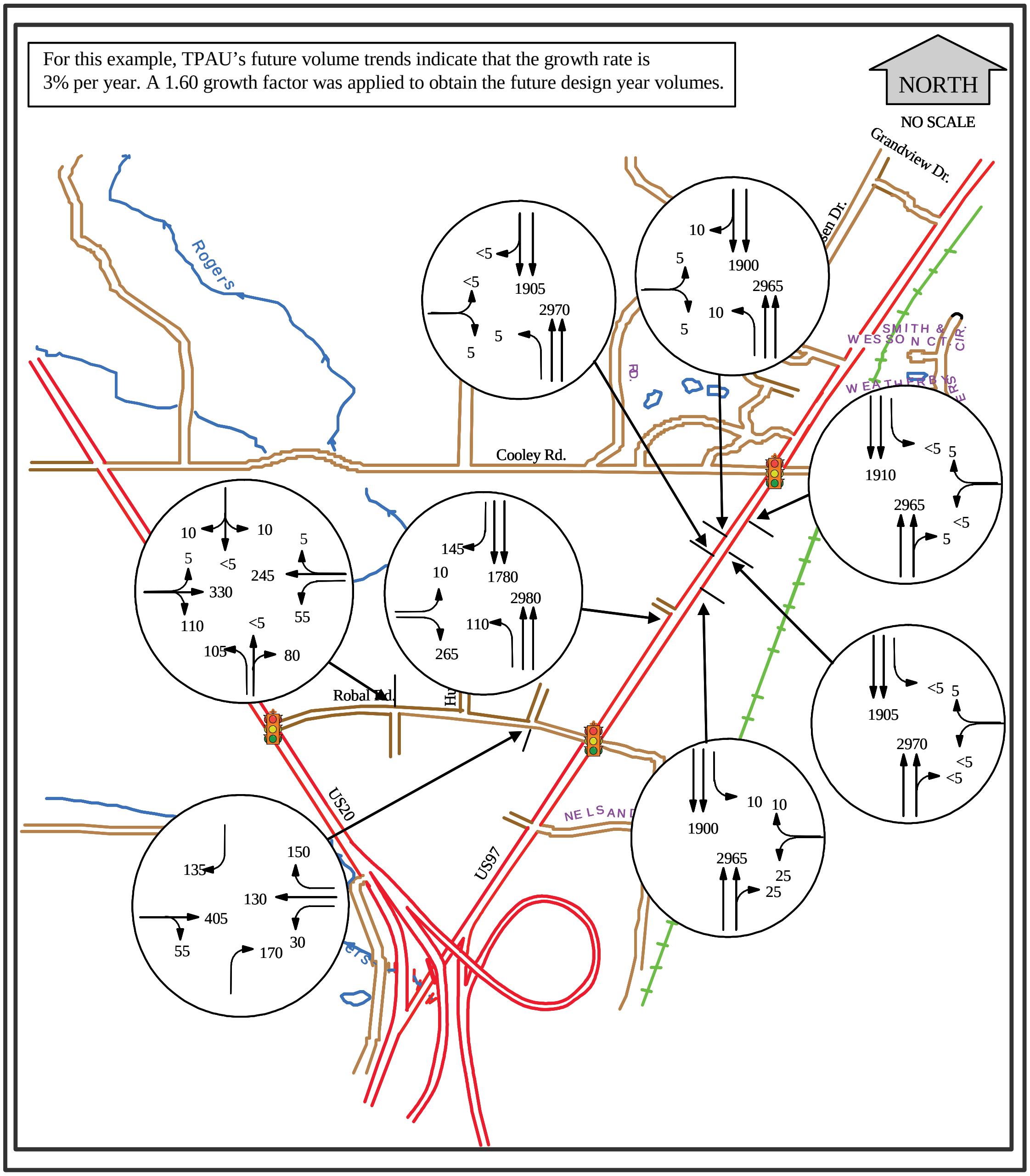
Exhibit 4-9 Balanced Future Design Hour Volumes (DHV)



OREGON DEPARTMENT OF TRANSPORTATION T P A U **TRANSPORTATION PLANNING ANALYSIS UNIT**

Project Title **Future Year DHV**	**File : .ppt**	**Prepared By:**	FIGURE E1
	Date :	**Reviewed By:**	

Exhibit 4-10 Balanced Future Design Hour Volumes (DHV) (Sheet 2)



OREGON DEPARTMENT OF TRANSPORTATION T P A U **TRANSPORTATION PLANNING ANALYSIS UNIT**

Project Title **Future Year DHV**	**File : .ppt**	**Prepared By:**	FIGURE E2
	Date :	**Reviewed By:**	

4.3 Peak Hour Selection

Daily traffic volumes, while useful for planning purposes, cannot alone be used for design or operational analysis purposes. Once all of the traffic counts have been obtained, the intersection counts should be adjusted to a single system peak hour. The peak hour is the single hour of the day that has the highest hourly volume. Use of the 15-minute breakdowns in the traffic counts is necessary in order to determine the true peak hour, resulting in a time period such as 4:00 PM to 5:00 PM or, just as easily, 4:45 PM to 5:45 PM. The final selection of a peak hour may be based on a simple majority of counts that have the same peak hour, using a controlling intersection, or the count(s) that the analyst believes are the most accurate. Counts that have longer durations or that are taken close to the 30 HV are generally more accurate. A procedure using TruckSum to determine the system peak hour volumes and other factors when the count peak hour is different from the system peak hour is provided in Chapter 11.

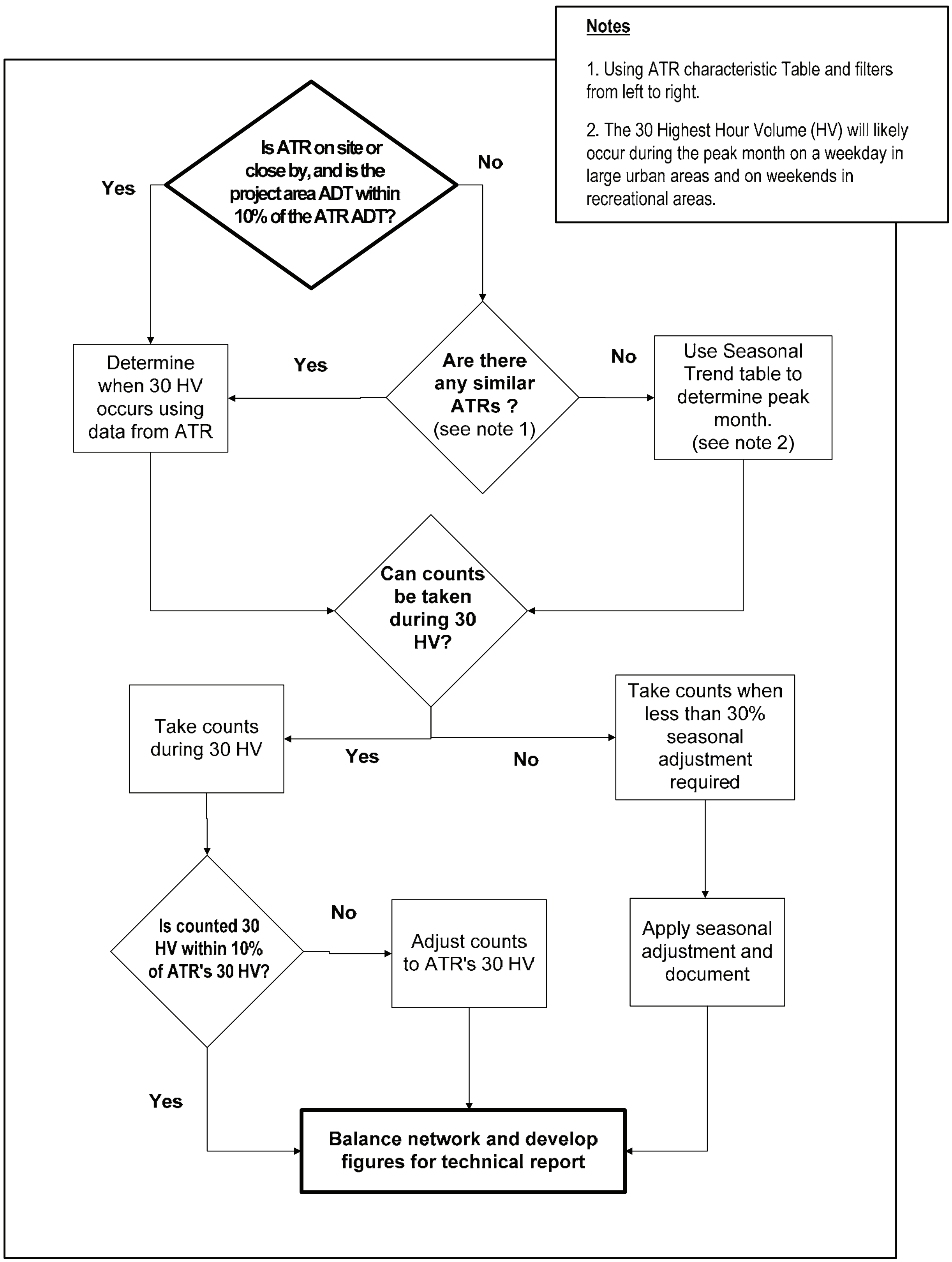
Generally PM peak hour volumes are higher than AM peak hour volumes. In areas where there are large industries with shift changes, the hour during the shift change may be as high as or higher than the PM peak hour for the remainder of the transportation network. If this is true, another set of volumes should be developed. Volumes for the non-standard peak hour should be developed along with the PM peak hour volumes so that all of the volumes may be analyzed at a later date. Multiple sets of volumes may be necessary in these circumstances, which may include areas of heavy industrial, retail or recreational uses; coastal routes; or on routes with highly directional commuter flows.

The peak hour from a manual count is converted to the 30 HV by applying a seasonal factor. The 30 HV is then used for design and analysis purposes. Experience has shown that the 30 HV in large urban areas usually occurs on a weekday during the peak month of the year. The 30 HV for an urban area typically ranges from 9- to 12-percent of the Average Annual Daily Traffic (AADT). For a recreational route, the 30 HV usually occurs on a summer weekend and ranges from 11- to 25-percent of the AADT.

It is recommended a top 200- to 500-hour count listing of the ATR(s) is obtained from the Transportation Systems Monitoring Unit. The 30 HV at the ATR(s) will be included in the list so that it will be possible to determine when the 30 HV occurs during the day and in the week. Manual counts can then be timed for the period when the 30 HV will likely occur, minimizing seasonal adjustments.

Exhibit 4-11 is a simplified flow chart of the process for developing 30th highest hour volumes.

Exhibit 4-11 Process for Development of 30th Highest Hour Volumes



4.4 <u>Seasonal Factors</u>

Since manual counts are taken throughout the year, data derived from a count taken in a particular month may need to be converted to the peak month by applying a seasonal factor. This can be accomplished using data collected from the ODOT ATR stations.

There are 141 ATR stations throughout the State Highway System. Most of these locations have loops in the roadway that count traffic flows for 24 hours a day/365 days a year, and have been in operation for many years. ATR information is available from the ODOT Transportation (Traffic) Volume Tables (TVT) located on the TDD Transportation Data Traffic Counting Program web site, as well as the ATR Characteristic Table and the Seasonal Trend Table located on the Transportation Analysis webpage of the TDD Planning Section website.

ATRs provide the percentage of AADT that occurs in the count month and in the peak month. This information can then be used to develop a seasonal adjustment that may be applied to the manual count using one of the following three methods.

- On-Site ATR Method
- ATR Characteristic Table Method
- ATR Seasonal Trend Table Method

The On-Site ATR Method is the best and most accurate method to use, followed by the ATR Characteristic Table Method and then the ATR Trend Table Method. All of the seasonal adjustment tables and ATR information are updated annually.

Seasonal factors greater than 30% should be avoided. Factors such as these indicate that a count was NOT taken at or close to the time that the 30 HV occurs. Using a winter count with a high seasonal factor to represent the peak summer period will likely not represent traffic turning movements accurately, as driving patterns change in the winter compared to the summer. As an example, suppose a count was taken at a rural intersection in the winter months with one of the minor legs of the intersection serving a campground beyond the intersection. The turning volume in the direction of the campground may be small or non-existent; say 5 vph. Even with a seasonal factor of 50%, this would result in an adjusted volume of only 8 vph, compared to an actual summer 30 HV that may be 20 vph. Simply factoring for the season would still leave the turning movements too low.

4.4.1 On-Site ATR Method

The On-Site ATR Method is used when there is an ATR within or near the project area. If located outside of the project area, there should be no major intersections between the ATR and the project area, and it should be within a minimal distance so that the traffic characteristics such as road class, number of lanes, rural/urban area, etc., are comparable. It is also important to check that the project area’s AADT in the Transportation Volume Table is within +/- 10% of the ATRs AADT.

Example 4-1 Seasonal Factor – On-Site ATR

On-Site ATR in Project Area

A traffic count was taken June 15th–18th along Kings Valley Highway No. 191 (OR 223) at MP 28.00.

- **Step 1: Transportation Volume Table** - ATR 02-005, located on Kings Valley Highway at MP 26.40, can be used.
- **Step 2: ATR Trend Summary** - The ATR number corresponds to a table in the last half of the TVT that contains yearly summaries for each ATR. From the column titled "Average Weekday Traffic/Percent of ADT," the count month and peak month percentage of ADT should be recorded. This information should be obtained from TVT's for the past five years. The peak month is the month with the highest percentage. The highest and lowest percentages should be eliminated to account for construction activity that may have occurred in the vicinity of the ATRs during the five year period. An average percent of ADT is then calculated for the remaining three years. The percentages shown in the TVT represent the 15th day of the month, so interpolation is needed if the count was taken near the beginning or end of a month.

Seasonal Adjustment Using ATR #02-005

	2003	2002	2001	2000	1999
Peak Month (July)	112%	113%	121%	111%	110%
Count Month (June)	108%	108%	108%	114%	115%

Note: Shaded values dropped from average calculation.

As shown above, the percentage of ADT values listed during June and July for the past five years are reviewed to calculate the average. The highest and lowest values, shown as shaded, are dropped from this calculation. The average monthly factors are determined as follows:

- The average peak month (July) is: (112%+113%+111%) / 3 = 112%.
- The average count month (June) is: (108%+108%+114%) / 3 = 110%.
- The seasonal adjustment is July/June = 112%/110% = 1.02.

Therefore, traffic volumes in the month of July are 1.02 times greater than in June. To convert the June traffic data to the 30 HV:

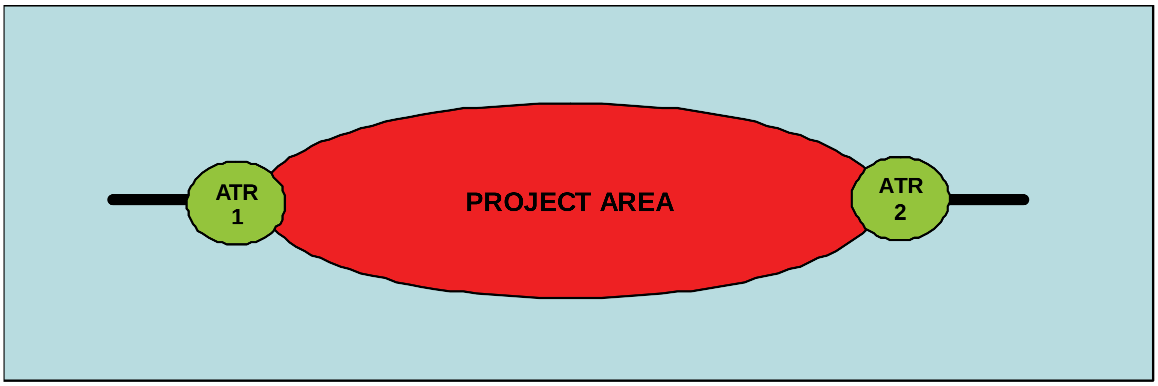
30 HV = (June PHV) x (Peak Month Percent of ADT/Count Month Percent of ADT).

If one of the peak hour turning movement volumes was 75 vph in June, then the 30 HV for July would be 1.02 x 75 vph = 77 vph.

Procedure for seasonal adjustment when 2 ATR's are within the project area:

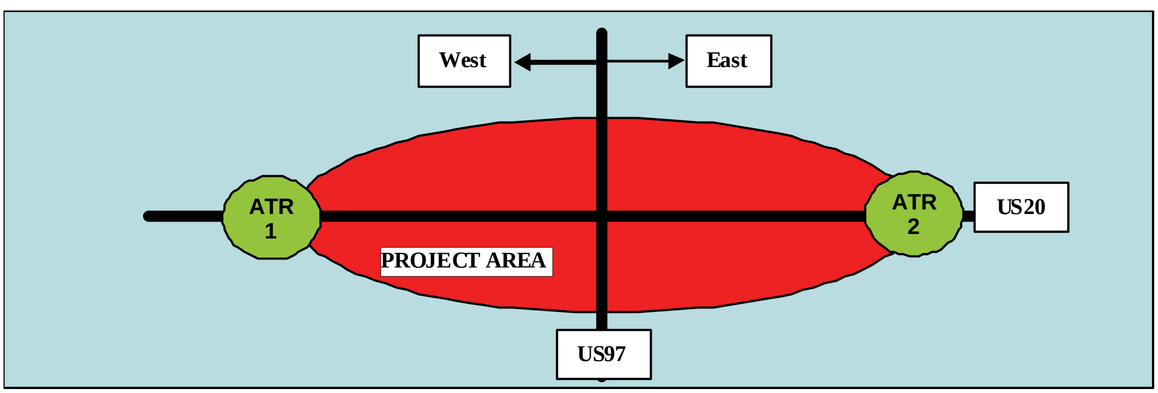
Scenario # 1: See Exhibit 4-12. In this scenario, the project area has two ATRs at each end. The project area ADT, roadway characteristics and roadway functional class are the same as at both ATRs. In order to seasonally factor the peak hour volumes within the project area, an average of the two ATR seasonal factors recommended.

Exhibit 4-12 Two ATRs in Project Area - Scenario #1



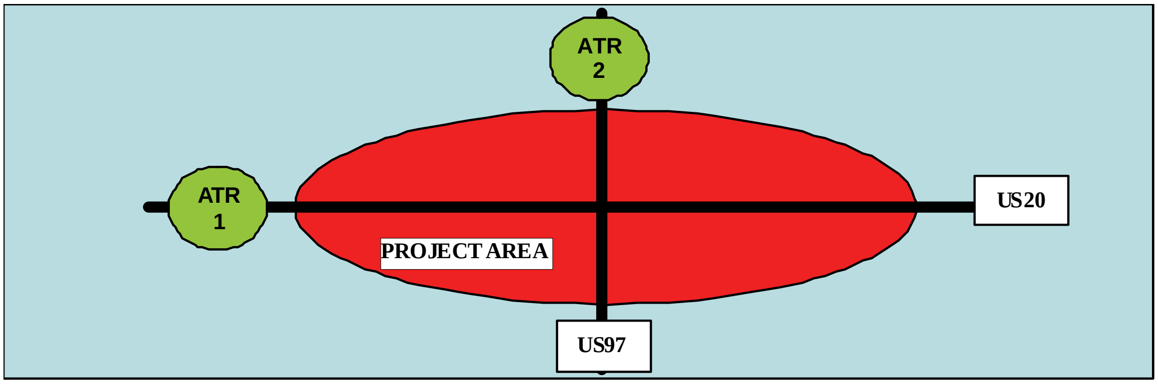
Senario # 2: Scenario 2 has two ATRs on US20 (See Exhibit 4-13) within the project area at each end. The roadways east of US97 have the same ADT and characteristics as ATR 2 while the west side has the same ADT and characteristics as ATR 1. With this scenario, each side of US97 should be seasonally factored using the ATR on that side.

Exhibit 4-13 Two ATRs in Project Area - Scenario # 2



Scenario # 3: In this scenario one ATR is located on US20 and another on US97. If US20 within the project area has the same roadway characteristics as at ATR 1, the seasonal adjustment factor at ATR 1 should be used for US20. The same process should be applied for US97 if US97 has the same roadway characteristics as at ATR 2. Otherwise, an average of the seasonal factors from both ATRs should be applied for the project area. (See Exhibit 4-14).

Exhibit 4-14 Two ATRs in Project Area - Scenario # 3



ATR Characteristic Table Method

The ATR Characteristic Table provides general characteristics for each ATR in Oregon, and should be used when there is not an ATR on-site. The Characteristic Table is a filterable Excel table that will often provide more than one ATR with similar characteristics. See example in Exhibit 4-15.

Averaging multiple ATRs with similar characteristics will yield a more appropriate factor than if only one ATR is used. Follow the steps described in the on-site ATR Method for averaging count and peak months over 5 years for each ATR with similar characteristics. The factor used to convert the traffic data to 30 HVs will be an average of these similar characteristic ATR factors. Seasonal Traffic Trend groupings for the table were constructed by plotting the monthly percent of AADT for each ATR. The plots were then grouped into trends with the greatest influence in traffic patterns.

Exhibit 4-15 ATR Characteristic Table Example

2005 ATR Characteristic Table										
Seasonal Traffic Trend	**Area Type**	**# of Lanes**	**Weekly Traffic Trend**	**2005 AADT**	**OHP Classification**	**ATR**	**County**	**Highway Route, Name, Location**	**MP**	**State Highway Number**
Summer < 2500	Rural	2	Weekday	760	District Highway	01-001	Baker	US 30, La Grand	33.20	66
Summer < 2500	Rural	2	Weekday	220	District Highway	01-007	Baker	OR 203, Medical	36.86	340
Summer < 2500	Rural	2	Steady	640	District Highway	01-010	Baker	OR 86, Baker	37.55	12

It is important to note that the trends provided in the table are not the only trends attributed to each ATR, but are the dominant trends. After the seasonal traffic trend characteristic is selected, other trend groupings, including area type (e.g., urban, rural), number of lanes and weekly traffic trends are broken down to provide more comparable sub-groupings.

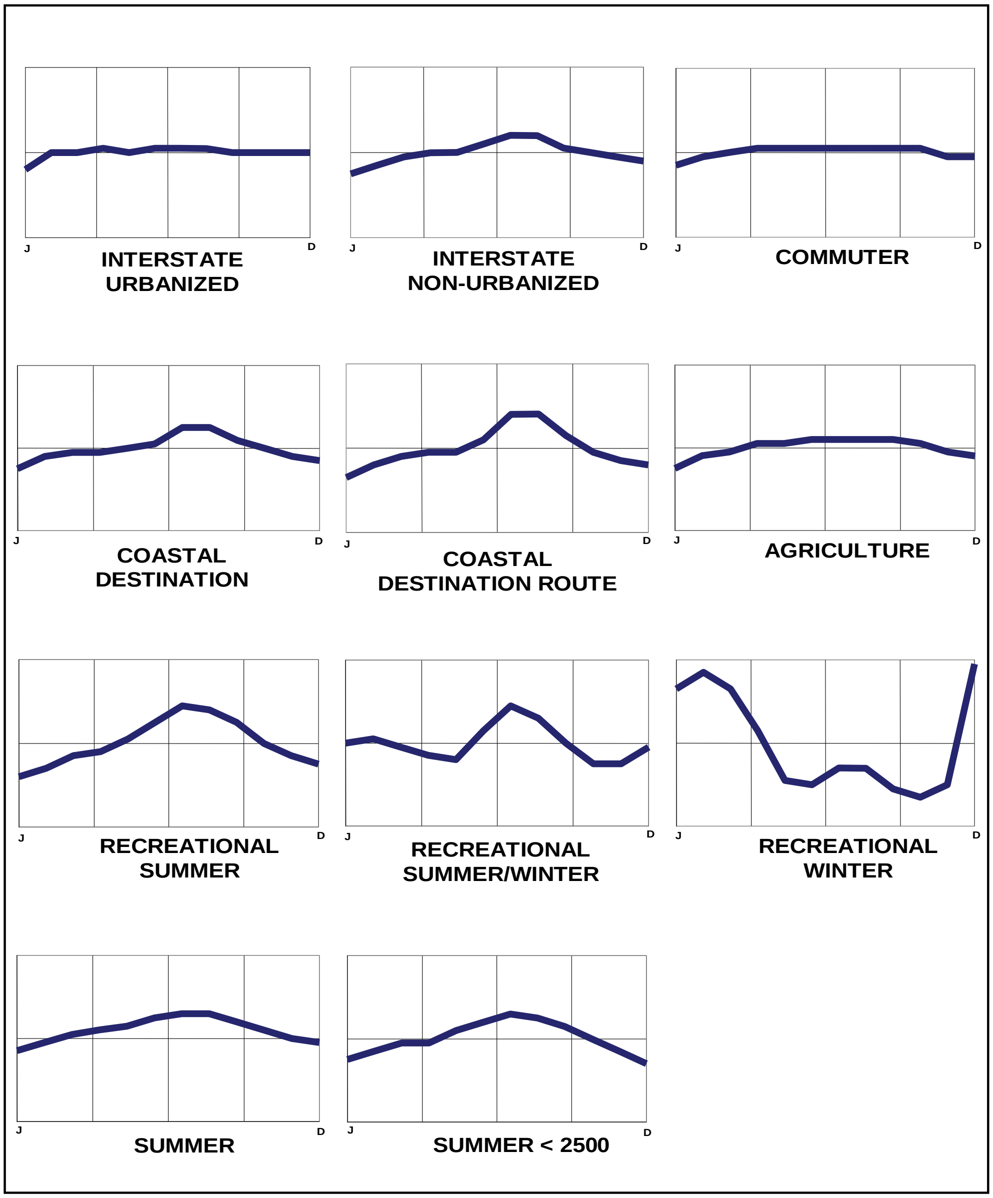
ATRs are characterized by only one of eleven seasonal trends, described below and illustrated in Exhibit 4-16. Project areas should be characterized by these trends in the order listed below.

1. **Interstate Urbanized**: ATRs located on any section of urbanized (areas of population > 50,000).interstate. (Example: I-5, Iowa Street - ATR #26-016.)
2. **Interstate Non-Urbanized**: ATRs located on any non-urbanized interstate section. (Example: I-84, west of Troutdale - ATR #26-001.)
3. **Commuter**: ATRs characterized by small seasonal changes in traffic patterns and commuting between city pairs. (Example: OR 22, West Salem Bridges - ATR #24-014.) Note: Also for non-state streets in urbanized cities.
4. **Coastal Destination**: ATRs characterized by summer peaks to/or within larger coastal city destinations as well as favorable routes from the valley. Favorable routes for Coastal Destinations include: Salmon River Highway (OR 18), Corvallis-Newport Highway (US 20/OR 34), Alsea Highway (OR 34), and Florence-Eugene Highway (OR 126). (Example: OR 18, east of Valley Junction - ATR #27-001.) Note: This grouping does not include the Sunset Highway.
5. **Coastal Destination Route**: ATRs characterized by high summer peaks on predominantly rural routes to/or between large coastal cities and coastal destinations. Rural routes include the Sunset Highway (US 26) from the Wilson River Hwy. junction, Umpqua Highway (OR 38), and Redwood Highway (OR 199). (Example: US 101, south of Rockaway - ATR #29-001.)
6. **Agriculture**: ATRs characterized by peaking in the late summer and fall harvest months. (Example: Kings Valley Highway - ATR #02-005.)
7. **Recreational Summer**: ATRs characterized by high summer peaks in recreational areas. (Example: Crater Lake Highway, south of Fort Klamath - ATR #18-021.)
8. **Recreational Summer/Winter**: ATRs characterized by both summer and winter peaks in recreational areas. (Example: Timberline Highway - ATR #03-008.)
9. **Recreational Winter**: ATRs characterized by high winter peaks in recreational areas. (Example: Century Drive Highway, Mt. Bachelor - ATR #09-011.)

If the project area trend does not fall into Trends 1 through 9, either Trend 10 or 11 should be used.

10. **Summer**: ATRs characterized by a smaller summer increase in traffic patterns when compared to Recreational Summer. (Example: US 26, south of Warm Springs - ATR #16-006.) Note: Also for non-state streets in small cities.
11. **Summer < 2,500 ADT**: ATRs with less than 2,500 ADT characterized by a smaller summer increase in traffic patterns when compared to Recreational Summer. Could be used, for example, for many rural off-system county roads. (Example: OR 31, east of Silver Lake - ATR #19-010.)

Exhibit 4-16 Seasonal Trends



ATRs are also characterized by weekly traffic trends and ADT.

- **Weekday:** Traffic volume trends greatest on weekdays; typical for commuter trend and urban areas.
- **Weekend:** Traffic volume trends greatest on weekends; typical for recreational trend and coastal destination trend.
- **Steady:** Traffic volume trends that are steady throughout the week without significant peaks on the weekend or weekdays.

ATRS are also characterized by area type and number of lanes.

- **Urbanized:** ATRs within areas of population > 50,000. (Examples: Portland and Salem) Urban Fringe: ATRs influenced by an urban area, such as an MPO area. (Example: Wilsonville)
- **Small Urban:** ATRs within areas of population between 5,000 and 49,999. (Examples: Albany and Pendleton)
- **Small Urban Fringe:** ATRs influenced by a small urban area. (Examples: US 101 south of Coos Bay and I-5 north of Albany)
- **Rural:** ATRs on routes outside of areas with population <5,000.
- **Rural Populated:** ATRs in cities with a population of less than 5,000. This also includes unincorporated communities. (Examples: Sisters and Tillamook)

To use the table, filter through the column characteristics from left to right to create a list of ATRs with similar characteristics. Starting with the "Seasonal Traffic Trend" column, filter out the traffic trend that best describes the project area. Next, filter the area type, number of lanes, and weekly traffic trend. Make sure that the section of highway where the ATR(s) is located and the project area for which the seasonal adjustments are being made have similar traffic characteristics. To be considered comparable, the AADT of the characteristic ATR should be within +/- 10% of the Transportation Volume Table AADT for the project area.

Example 4-2 Seasonal Factor – ATR Characteristics Table

ATR Characteristic Table Method for a Project Area

A count was taken June 15th–18th along Corvallis-Lebanon Highway No. 210 (OR 34), west of I-5 at MP 5.35. The Transportation Volume Table AADT is 28,100.

- **Step 1: Transportation Volume Table**: There are no ATRs on this section of the highway.
- **Step 2: ATR Characteristic Table**: This section of highway can be categorized as Commuter/Urban Fringe/Five-Lanes. Filtering through the ATR Characteristic Table from left to right, two ATRs have similar characteristics to the project area. However, ATR 26-003 has an AADT of 39,100 and is an expressway. As previously noted, characteristic AADT counts should be within +/- 10% of the Transportation Volume Table AADT in order to be considered comparable to the project area. Alternatively, ATR 27-006 is not an expressway and has an AADT of 26,900, which is within 10% of

the TVT AADT. The characteristics of these two representative locations are summarized below.

Example ATR Characteristic Table (Year 2003)

Characteristics	ATR Location 1	ATR Location 2
Seasonal Traffic Trend	Commuter	Commuter
Area Type	Urban Fringe	Urban Fringe
Number of Lanes	5	5
Weekly Traffic Trend	Weekday	Weekday
2003 ADT	39,100	26,900
OHP Classification	Statewide Hwy (Expressway)	Statewide Hwy
ATR	26-003	27-006
County	Multnomah	Polk
Highway Route, Name and Location	OR 26, Mt. Hood Hwy, E of Gresham	OR 22, Willamina-Salem Hwy Oak Knoll
ATR Milepoint	14.36	19.4
State Hwy Number	26	30

- **Step 3: ATR Trend Summary**: Data from ATR #27-006 is located in the ATR summary in the back of the TVT and under the “ATR Trend Summaries” on ODOT’s Traffic Counter Program website TDD Transportation Data Traffic Counting Program. The count was taken on June 15th, which is in the middle of the month, so the ATR percentages from the TVT can be used directly without interpolation. The peak month was found to be August for two of the three years. Because ATR #27-006 is relatively new, the percentages were averaged over the existing three years (not the normal five year historical data), as shown below.

Seasonal Adjustment Using ATR #27-006

	2003	2002	2001
Peak Month (August)	110%	110%	110%
Count Month (June)	107%	106%	106%

- The average peak month (August) is: (110%+110%+110%) /3 = 110%.
- The average count month (June) is: (107%+106%+106%) /3 = 106%.
- The seasonal adjustment is August/June = 1 10%/106% = 1.04.

Therefore, traffic volumes in the month of August are 1.04 times greater than in June. To convert the June traffic data to the 30 11V: 30 11V = (June P11V) x (Peak Month Percent of ADT/Count Month Percent of ADT).
If one of the peak hour turning movement volumes were 100 vph in June, then the 30 11V for August would be 1.04 x 100 vph = 104 vph.

4.4.2 Seasonal Trend Method

The seasonal trend table is used when there is not an ATR nearby or in a representative area. The Seasonal Trend Table was constructed by averaging seasonal trend groupings from the ATR Characteristic Table. Essentially, by using a factor from the table, the average for the entire trend grouping is applied to the project area as shown in Exhibit 4-17.

Exhibit 4-17 Example ATR Seasonal Trend Table (Year 2003)

	Jan 1	Jan 15	Feb 1	Feb 15	Dec 15	Peak Period Seasonal Factor
Recreation Summer/Winter	1.2349	1.2922	1.4023	1.5123	1.1776	0.8582
Recreation Winter	0.9119	1.0561	1.0292	1.0023	0.7676	0.7676

To determine the appropriate seasonal trend, select from the list the trend that best describes the project area. Trends should be characterized in the same order as previously described in the ATR Characteristic Table Method. The Seasonal Factor Table is updated yearly. It is not necessary to average 5 years worth of seasonal factors for this method, or compare AADTs because, as previously stated, this method uses an average of all ATRs in the characteristic trend. In certain areas, averaging seasonal trends may yield a more appropriate factor than just a single trend. These areas include:

- **Coastal Destination and Coastal Destination Route Trends:** It may be necessary to average trends in areas such as Warrenton, Depoe Bay and Yachats. While these cities are destinations along the Oregon Coast, they do not have the summer influx of traffic associated with larger coastal destinations such as Lincoln City and Seaside. A Coastal Destination Trend Factor for these areas may be too high, while a Coastal Destination Route Trend Factor may be to low. When analyzing coastal cities such as these, it is appropriate to average the trends to yield a more reasonable factor.
- **Summer and Commuter Trends**: It may be necessary to average trends when analyzing mid-sized cities such as Philomath, Dallas and Sutherlin. For urbanized areas the commuter trend is appropriate, while for smaller areas the summer trend is appropriate. However, for mid-sized areas such as these, the summer or commuter trends may alone be too high or too low. A more reasonable factor would be obtained by averaging the summer and commuter trends.
- **Interstate and Interstate Urbanized Trends**: It may be necessary to average trends when analyzing interstates in small urban and fringe areas (urban and small urban) such as Albany, Wilsonville and north of Roseburg. For rural areas the interstate trend is appropriate, while for urbanized areas the interstate urbanized trend is appropriate. For small urban and fringe areas such as these, however, these trends may alone be too high or too low. A more reasonable factor would be obtained by averaging the interstate and

interstate urbanized trends.

It is important to note that these are the only trend grouping pairs that would be appropriate to average, with the exception of interchange ramps, which should use an average of the mainline and cross road seasonal adjustments. Interstate should only be averaged with interstate urbanized, and should never be averaged with Coastal or Recreational. The same is true for the other trends not listed in the above examples. The Seasonal Trend Table is located on the Transportation Analysis webpage of the TDD Planning website.

Factoring count data to the peak month requires dividing the seasonal factor for the count period by the seasonal factor for the peak period. The peak period seasonal factor for a traffic trend is the lowest value in the row, and is highlighted in the last column in the table.

Seasonal factors are given for the 1st and the 15th of each month so if the count date is not at the beginning/end or in the middle of a month interpolation is needed.

Example 4-3 Seasonal Factor – Seasonal Trend Table

This example demonstrates the Seasonal Trend Method for a Project Area.

A count of 11,000 was taken July 1st – 5th along Oregon Coast Highway No. 9 (US 101) at MP 63.19 (north of Tillamook).

- **Step 1: Transportation Volume Table**: There are no ATRs on this section of the highway.
- **Step 2: ATR Characteristic Table**: This section of highway can be categorized as Coastal Destination/Populated Rural/Two-Lanes. Filtering through the ATR Characteristic Table, from left to right, two ATRs have similar characteristics to the project area. However, none of the characteristic ADT values are within +/- 10% of the Transportation Volume Table ADT for the project area. Refer to the table below for details regarding these two candidate locations.

Example ATR Characteristic Table (Year 2003)

Characteristics	ATR Location 1	ATR Location 2
Seasonal Traffic Trend	Coastal Destination	Coastal Destination
Area Type	Pop Rural	Pop Rural
Number of Lanes	2	2
Weekly Traffic Trend	Weekday	Weekend
2003 ADT	6600	19500
OHP Classification	Statewide Hwy-Scenic Byway	Statewide Hwy (Expressway)
ATR	26-003	27-006
County	Multnomah	Polk
Highway Route, Name and Location	US 101, Oregon Coast Hwy, S of Bandon	OR 18, Salmon River Hwy, E of Valley Junction
ATR MP	275.87	23.76
State Hwy Number	9	39

- **Step 3: Seasonal Trend Table**: Since there are no ATRs with similar characteristics, the Seasonal Trend Table must be used. The correct values are obtained by following the “Coastal Destination” row to the “Jul_1” count month column, and to the “Peak Period Seasonal Factor” column at the end of the table, as summarized below.

Seasonal Trend Table (Year 2003)

	Jun 15	Jul 1	Jul 15	Aug 1	Peak Period Seasonal
Coastal Destination	0.9948	0.9546	0.8940	0.8334	0.8334

- The peak period seasonal factor is 0.83 34.
- The count date seasonal factor (July 1st) is 0.9546.
- The seasonal adjustment is: Count Date Seasonal Factor/Peak Period Seasonal Factor = .9546 / .8334 = 1.15.

Therefore, the peak period volumes for a Coastal Destination are 1.15 times greater than volumes for the 1st – 5th of July.

To convert the July traffic data to the 30 HV:

30 HV = (July PHV) x (Count Date Seasonal Factor / Peak Period Seasonal Factor).

If one of the peak hour turning movement volumes were 100 vph in July, then the 30 HV would be 1.15 x 100 vph = 115 vph.

4.5 Volume Development for Sketch Planning Analysis

For certain planning studies, such as a county TSP analysis, sketch planning level analysis of highway segments may be appropriate, for all or portions of the study. To develop planning-level design hour volumes, TVT AADT volumes can be used. The TVT AADT volumes are used to derive the 30 HV by multiplying by the K-30 factor. A K factor is the ratio between a peak hour and the ADT. The K-30 factor is the ratio of the 30th highest hour to the AADT and should be used for this purpose. Short term count K factors should not be used for this purpose. A background report on this topic is available on the TPAU website[3].

K-30 factors are derived from ATRs and can be found in the TVT section on Summary of 4 Use of Short-Term Interval Counts to Determine K Factors, Don R. Crownover, P.E., ODOT Transportation Systems Monitoring (TSM) Unit, August, 2006. Trends at ATRs. They are listed under Historical Traffic Data, Percent of ADT for the 30th Hour. A representative ATR needs to be identified following the procedures described in Section 4.3.

Example 4-4 Converting ADT to DHV (K_{30}) factor

This example illustrates how to calculate and apply the K-30 Factor.

Find the 30 HV for sketch planning analysis for a segment of Kings Valley Highway No. 191 (OR 223) at MP 28.00.

- **Step 1: Transportation Volume Table –** The TVT is used to find the AADT for this segment.

The 2005 AADT from the Transportation Volume Table is found to be 1200.

- **Step 2: ATR Trend Summary –** ATR 02-005, located on Kings Valley Highway at MP 26.40, can be used for this location. The ATR number corresponds to a table in the last half of the TVT that contains yearly summaries for each ATR. From the column titled "Percent of ADT" the 30th Hour percent of ADT should be recorded for the past five years. The highest and lowest percentages should be eliminated to account for construction activity that may have occurred in the vicinity of the ATR during the five-year period. An average percent of ADT is then calculated for the remaining three years.

K-30 Factors Using ATR #02-005

	2003	2002	2001	2000	1999
K-30	****	13.9%	11.3%	11.0%	11.3%

Note: Shaded values dropped from average calculation.

As shown in Example 4-1, the percentage of ADT values listed during June and July for the past

[3] Use of Short-Term Interval Counts to Determin K Factors, Don R. Crownover, P.E., ODOT Transportation Systems Monitoring (TSM) Unit, August, 2006.

five years are reviewed to calculate the average. The highest and lowest values, shown as shaded, are dropped from this calculation. The average K-30 factor is determined as follows:

- The average K-30 factor is: (11 .3%+1 1 .0%+1 1.3%) / 3 = 11.2%.

Calculate the two-way 30 HV:

30 HV = (AADT) x (Average K-30 Factor) = 1200 x 0.112 = 134 vph.

Obtain D-30 factor from critical hour listing.

From 2003 critical hour listing for ATR 02-005
D-30 NB = 0.56 and D-30 SB = 0.44

Calculate the directional 30 HV:

30 HV NB = (30 HV) x (D-30 factor) = 134 x 0.56 = 75 vph.
30 HV = (30 HV) x (1-D-30 factor) = 134 x 0.44 = 59 vph, round to 60 vph.

4.6 Forecasting

The analyst should work with the PT to determine the future year before beginning any forecasting. The design hour that is used for many projects is 20 years after the year of project opening. In planning, a 20-year horizon is typically used when evaluating transportation needs and solutions. TIS horizon year procedures are provided in the Development Review Guidelines. For refinement and other similar plans, the horizon year should be 25 to 30 years out, which would increase the life of the plan, especially if the project development process does not directly follow. There are three main methods for estimating DHVs:

- Historical Trends
- Cumulative Analysis
- Urban Travel Demand Models

4.6.1 Historical Trends

The historical trends method uses traffic volumes from previous years to project future volumes. This method assumes that the future growth trend will be similar to the historical trend. It is used mainly in rural or small urban areas where significant growth is not anticipated. Current and future year traffic volumes are available in the Future Volumes Table webpage.

The Future Volumes Table is updated annually. Within the table, the milepoint location for the project highway that most closely resembles the traffic flows for the section being analyzed is selected. Sometimes, another milepoint may be closer to the section being analyzed, but there may be a cross street that affects traffic volumes on the highway so that the growth rate is different.

There are three columns in the table for the most recent three years of traffic count data. Only one of the columns is filled; corresponding to the last year that the location was counted. The far right column contains the R-squared value for the regression equation that was used to estimate the historical trend. The R-squared value measures the degree of correlation between the dependent variable (historical traffic volumes) and the independent variable (time). A value of 1.0 indicates an exact linear relationship between the historical counts and time. Ideally, the R-squared value should exceed 0.75. However, values higher than 0.50 are still acceptable, if there is nothing else available. Conversely, a low R-squared value indicates a weak relationship. In this case, the table should be investigated for a nearby location having similar traffic characteristics and a more acceptable R-squared value.

Example 4-5 Future Volumes Using Historic Trend

In this example, the forecast 20 year traffic volumes are developed based on historical counts.

For the Lava Butte ATR (#09-003) located on US 97 at MP 142.41, The following table shows the 1999 traffic volume, Year 2019 traffic volume and the R-squared value.

Example Future Volumes Table

Hwy#	DIR	MP	Description	1999	2019	RSQ
4	1	141.01	.01 miles S of Badger Rd	28400	47200	0.9212
4	1	141.52	.22 miles S of Murphy Rd	24000	41400	0.656
4	1	142.21	ATR 09-003 - Lava Butte	19600	32000	0.9338
4	1	143.47	.01 miles S of Galen Baker Rd	14200	23600	0.7328
4	1	153.09	.01 miles S South Century Dr	9600	11100	0.5788

RSQ = Root Mean Square

Based on the data above, the 20-year growth factor would be 1.63 (32,000/19,600). Assuming linear growth in the future, the annual growth factor would be (1.63 –1.0) /20 = 0.032, or 3.2%. The R-squared value of 0.9338 is acceptable, indicating a strong relationship. To convert the 1997 30 HV from Example 1 to a 2019 DHV, the 1997 30 HV is multiplied by the 20-year growth factor, with an additional two years of growth added to this.

2019 DHV = 1997 DHV x (20-Year Growth Rate + 2 x Annual Growth Rate) = 112 vph x (1.63 + (2 x 0.032))
= 112 vph x 1.694
= 190 vph

When dividing the estimated future year volume by the most recent count volume it is important to note the numeric difference between the two years. In the example above, a 20-year growth rate was used between 1999 and 2019. Other highways may have been last counted in 1997 or 1998. This would mean that a 21- or 22-year growth rate should be applied. Dividing the total growth by 20 years would, in these cases, overestimate the growth rate.

For areas with multiple growth factors, discard any with an R-squared value less than 0.75. Remaining growth factors that are within 1 or 2 percent can be averaged.

4.6.2 Cumulative Analysis

The cumulative analysis method is generally used to forecast volumes for small urban areas that are growing at a fairly uniform rate or for areas where only minor changes are expected to take place. This method is also used to project land use changes and effects on the transportation system. Due to increased traffic volumes practical use is limited to small urban areas or in sub-areas of larger regions because of the complexity in tracking changes associated with larger areas, such as an increase in parallel routes and/or the number of alternatives in the analysis.

The cumulative method uses information on existing and planned land uses in addition to historical trends to predict total future traffic volumes. Total future volumes are estimated as the sum of the existing volumes plus future background traffic plus future development traffic (optional).

Total Traffic Volumes = Existing Volumes + Future Background Traffic
+ Future Development Traffic (optional)

Future development traffic only applies in case of a TIS. As described in Section 4.2, the existing 30 HV traffic is derived using counts and seasonal adjustments. The future background traffic includes additional traffic from the growth in through trips (may be estimated based on historical growth trends), traffic generated by approved and pending developments and build-out development of vacant land (as determined from current zoning and land use densities). In some cases, the vacant land growth rate may not reach build out by the forecast horizon year. If historical development rates indicate full development is not expected within the horizon year, then a lesser growth level should be assumed. Traffic generated by future development is estimated using the Institute of Transportation Engineers (ITE) Trip Generation Manual (if manual trip calculations are used) or a travel demand model for larger studies. In case of a TIS, the future development traffic is the specific development analyzed in the TIS. The analyst should be cautioned not to double count the impact of the subject property with the future build-out of vacant developable land covered in the background traffic portion. The generated traffic is then distributed throughout the study area according to O-D study data, if available, or local knowledge.

Example 4-6 Cumulative Analysis for TIA

This example demonstrates the addition of local development trips to the DHV forecast. The example continues the use of the DHV obtained in Example 4-4, which was projected to increase from 112 vph in 1997 to 190 vph in 2019 based on the historical growth trend. Assume that the land use in the area is currently exclusive forest use.

Increases in through trips are assumed in the 78 vph increase in the historical trend method. Future traffic growth will be related only to the increase in forest area use, i.e., no new development is reflected in the historical growth trend. In 2001, however, a new housing development was approved for construction, with completion by 2003. This is a 50-acre development with single-family homes on 2-acre lots resulting in 25 new homes.

From the ITE Trip Generation Manual, it is estimated that 25 single-family detached houses generate 25 peak hour trips total. Of the 25 trips in the afternoon peak hour, 16 are returning home. From O-D study data and existing traffic patterns, it is estimated that 15 of the 16 homeward trips are a part of the same turning movement containing the projected 190 vph from Example 4. The other 10 trips are a part of different turning movements, therefore:

2019 Future Volumes = 2019 DHV + 2019 Development Volumes
= 190 vph + 15 vph
= 205 vph

It is important to not double count traffic when using the cumulative method. If the turning movement described in this example accessed only a local street, the historical growth increase would not be applied since this is used only for through traffic on streets that extend beyond the study boundary. Thus, the only increase in turning volumes would be due to the new 25 single-family homes, and the turning volume would be 112 vph + 15 vph = 127 vph instead of 205 vph.

Example 4-7 Zonal Cumulative Analysis

The following procedure may be used for applying the cumulative method for projects. (Generally, this level of effort is too complex for a Transportation Impact Study.)

- Identify the study area. The study area should be defined such that all relevant facilities are included, since there may be other roadways that could directly influence the traffic patterns on the facilities being analyzed.

Divide the study area into zones as shown below. In areas where development is not expected to occur uniformly, but vary widely between zones, it is recommended that select link and zone tree data be reviewed to identify potential changes in travel patterns and traffic growth along specific facilities.

Study Area Zones

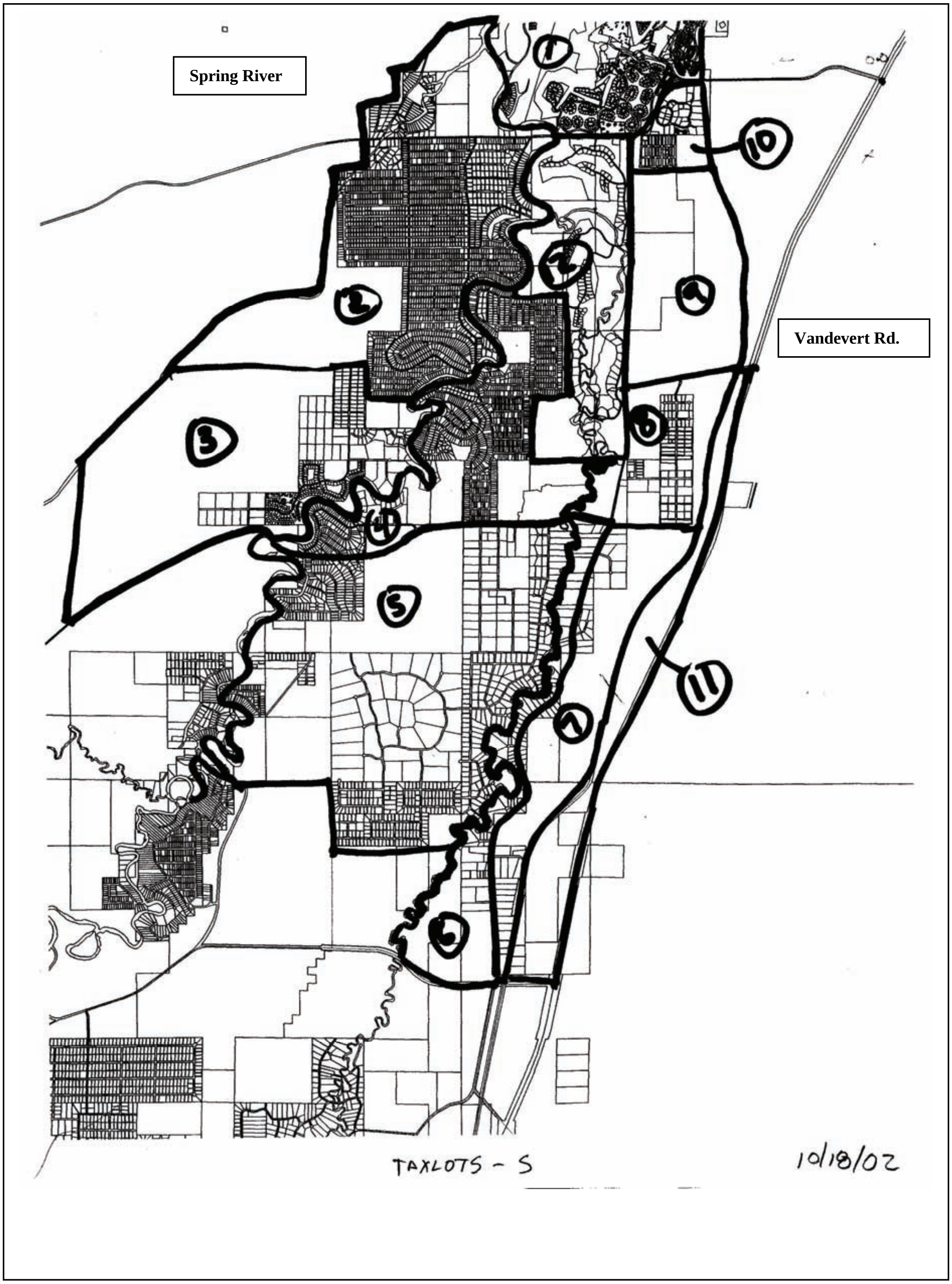
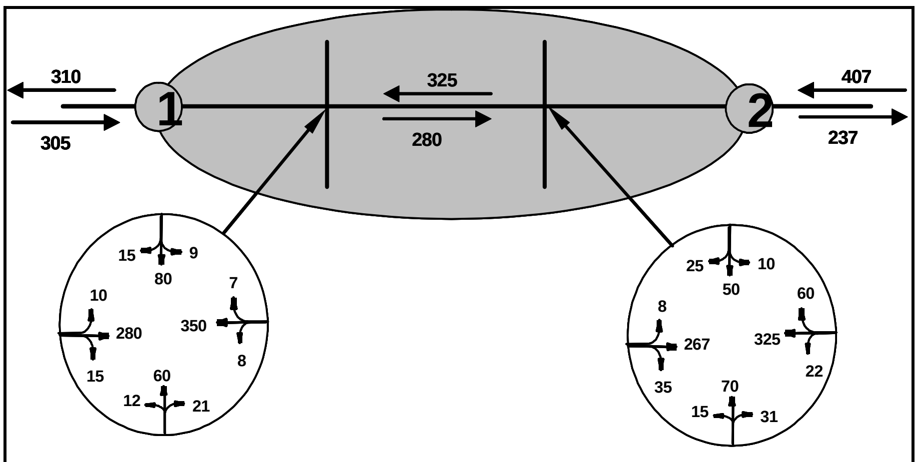


Exhibit 4-18 Vacant Lots and Projected Trip Generation

(1) Zone	(2) Vacant Lots (2002)	(3) Building Rate	(4) 2027 Built Lots	(5) 2027 Total Trip Gen. (Enter/Exit)
1	537	80	537 (Buildout = 7 yrs.)	542 (349/193)
2	984	30	750 (Built by 2027)	758 (488/270)
3	167	17	167 (Buildout = 10 yrs.)	169 (109/60)
4	640	35	640 (Buildout = 19 yrs.)	646 (416/230)

Using the base year 30 HV, remove the percentage of external-external trips from the total external trips. If the origin-destination survey is available, use it. Otherwise, based on the 30 HVs, directionally hold a volume at one end (external station) and proceed to the other end (external station) by subtracting all turn volumes at each intersection downstream, as shown in Exhibit 4-19.

Exhibit 4-19 Determining Percent of External-External Trips at External Stations



Assume no other accesses between external-stations 1 and 2:
The eastbound external-external trip is 237 vehicles.
The westbound external-external trip is 310 vehicles.

External trips are those trips that have at least one end located outside of the study area, as defined by the cordon line. The cordon line is an imaginary line that denotes the boundary of the study area. See

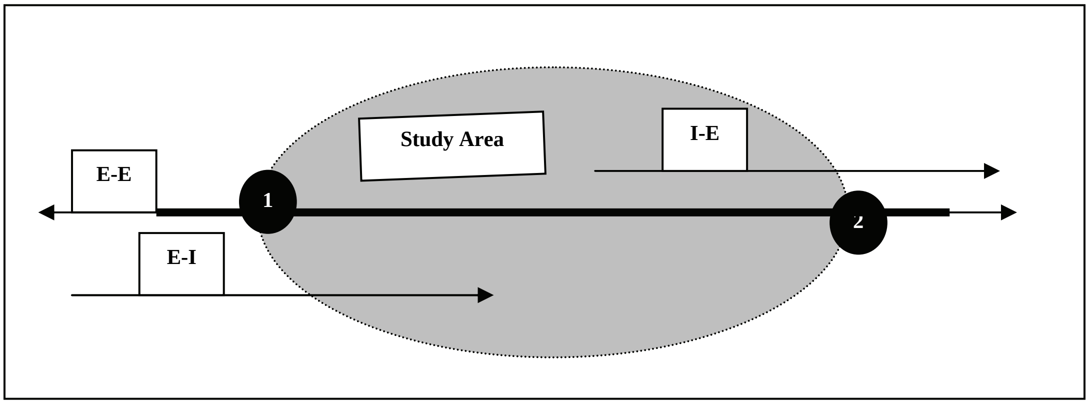
Exhibit 4-20.

External-external (through) trips are trips with both ends (origin and destination) outside of the cordon line.

External-internal and internal-external trips are trips with one end outside of the cordon line.

The trips that do not cross the cordon line are internal-internal trips.

Exhibit 4-20 External-External, External-Internal and Internal-External Trips



Determine the percentage of external-external trips then calculate the increased external-internal and internal-external trips for the design year at the external stations as follows:

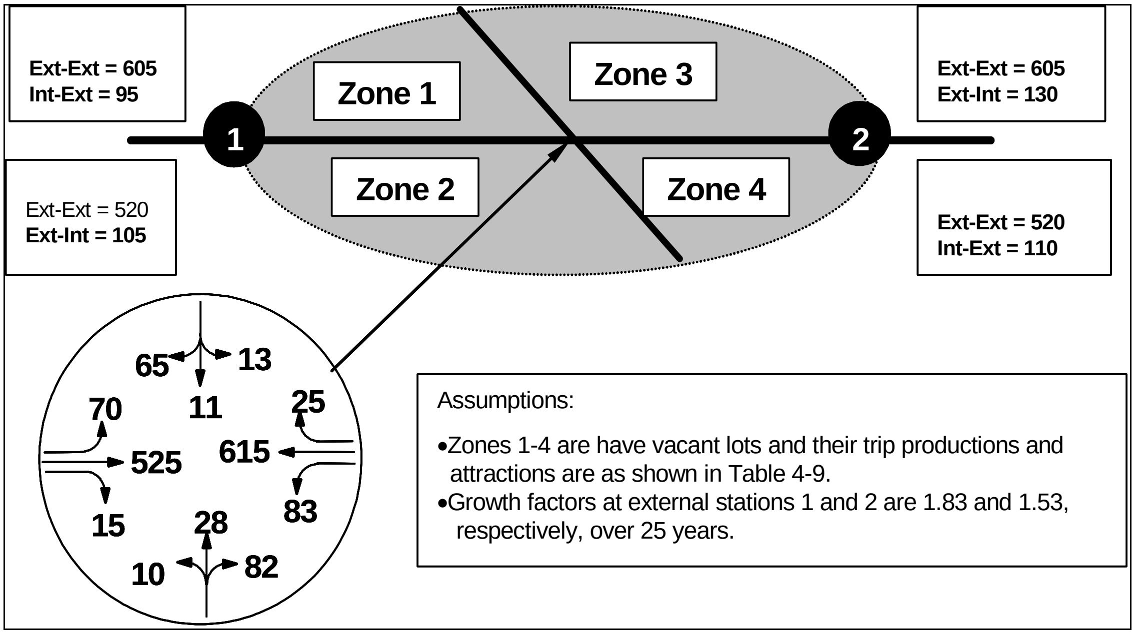
- Apply the growth factor to the base year 30 HVs to estimate the design year 30 HVs. See Exhibit 4-18, Column (4).
- Based on the percentage of external-external trips, calculate the base year external-external trips. See Exhibit 4-18, Column (3).
- Calculate the increase in external-external trips for the design year as the product of the percentage of external-external trips and the difference between the design year 30 HVs and the base year 30 HVs. See Exhibit 4-18, column (6).
- Calculate the increase in external-internal and internal-external trips for the design year as the difference between the design year 30 HVs and the sum of the base year 30 HVs and the increase in external-external trips for the design year. See Exhibit 4-18, column (7).

Example 4-8 Zonal Cumulative Analysis – E-I and I-E Trip Forecast Generation

External-Internal and Internal-External trip forecasts are demonstrated in this example.

Using the information in Exhibit 4-21, find the increase in external-internal and internal-external trips over 25 years.

Exhibit 4-21 Determining Percent of External-External Trips At External Stations



Solution:

As given in Exhibit 4-21.

- Traffic volumes at the external-station 1 are 625 entering and 700 exiting.
- The probability of an external-external trip entering at the external-station 1 is 520/625 = 0.83 and the probability of an external-external trip exiting at Node 1 is 605/700 = 0.86.
- Traffic volumes at the external-station 2 are 735 entering and 630 exiting.
- The probability of an external-external trip entering at the external-station 2 is 605/735 = 0.82 and the probability of an external-external trip exiting at Node 2 is 520/630 = 0.82.
- The estimated increase in internal-external and external-internal trips over 25 years is shown in Exhibit 4-22, column (7).

Exhibit 4-22 25-Year Internal-External, External-Internal Trip Increase

Ext. Trip Table	Direction	(1) 2002 DHV	(2) Growth Factor	(3) 2002 E-E =(1)*(5)	(4) 2027 DHV =(1)*(2)	(5) E-E Trip Prob.	(6) 2027 E-E Trip Growth =(5)*((4)-(1))	(7) 2027 E-I, I-E Trip Growth = (4)-(1)-(6)
External-station 1	Enter (attr.)	625	1.83	519	1144	0.83	431	88
	Exit (prod.)	700	1.83	602	1281	0.86	500	81
External-station 2	Enter (attr.)	735	1.53	603	1125	0.82	320	70
	Exit (prod.)	630	1.53	517	964	0.82	274	64

After the external-external trip growth has been removed from the total external trip growth, the remaining trips are distributed to the internal zones according to the following procedure.

- Distribution of growth in external-internal trips:
    - Calculate the attraction probability of each zone's new trip attractions by dividing its new trip attractions by the study area's total new trip attractions.
    - Distribute the growth in external-internal trips for each external station by multiplying these trips by each zone's attraction probability.
- Distribution of growth in internal-external trips:
    - Calculate the production probability of each zone's new trip productions by dividing its new trip productions by the study area's total new trip productions.
    - Distribute the growth in internal-external trips for each external station by multiplying these trips by each zone's production probability.

Example 4-9 Zonal Cumulative Analysis – E-I and I-E Trip Distribution

In this example, I-E and E-I trips are distributed to the identified zones.

Distribute the new external-internal and internal-external trips in Exhibit 4-22 to the four zones shown in Exhibit 4-21. The new trip productions and attractions for these zones are shown in Exhibit 4-22.

Solution:

The calculation of the attraction and production probabilities is shown in Exhibit 4-23. For example, Zone 1's attraction probability is 349/1362 = 0.256 and its production probability is 193/753 = 0.256.

Exhibit 4-23 Example External Trip Attractions and Productions Probabilities

Zone	1	2	3	4	Total
Total New Trips	542	758	169	646	2115
Trip Attractions	349	488	109	416	1362
Attraction Probability	0.256	0.358	0.080	0.305	1.0
Trip Productions	193	270	60	230	753
Production Probability	0.256	0.359	0.080	0.305	1.0

The distribution of new external-internal trips is shown in Exhibit 4-24. For example, Zone 1's new external-internal trips at external-station 1 are 88*0.256 = 23.

Exhibit 4-24 Example External-Internal Trip Distribution

External-station	New E-I Trips	Zone 1	Zone 2	Zone 3	Zone 4
1	88	23	32	7	27
2	70	18	25	6	21

The distribution of new internal-external trips is shown in Exhibit 4-25. For example, Zone 1's new internal-external trips at external-station 1 are 81*0.256 = 21.

Exhibit 4-25 Example Internal-External Trip Distribution

External-station	New I-E Trips	Zone 1	Zone 2	Zone 3	Zone 4
1	81	21	29	6	25
2	64	16	23	5	20

After the new external-internal and internal-external trips have been distributed for each zone, the remaining new attractions and productions are internal-internal trips. For Example 4-8, the total internal-internal trips for Zone 1 are the difference between Zone 1's total new attraction/production trips and Zone 1's total new internal-external/external-internal trips (542 - 23 - 18 - 21 - 16 = 464).

To distribute the internal-internal trips for each zone, use the same distribution process as described in the new external-internal and internal-external trips distribution.

Example 4-10 Zonal Cumulative Analysis – I-I Trip Generation and Distribution

This example illustrates the trip distribution of Internal to Internal trips

Find and distribute the internal-internal trips for each zone as shown in Exhibit 4-21.

Solution:

The internal-internal trips for each zone and the associated attraction and production probabilities are shown in

Exhibit 4-26.

Exhibit 4-26 Example Internal Trip Attractions and Productions Probabilities

	Zone 1	Zone 2	Zone 3	Zone 4	Total
Total Internal-Internal Trips	464	649	145	553	1811
Internal Attractions	308	431	96	368	1203
Attraction Probability	0.256	0.358	0.080	0.306	1
Internal Productions	156	218	49	185	608
Production Probability	0.257	0.358	0.081	0.304	1

The distribution of new internal-internal attractions are shown in Exhibit 4-27. For example, the attraction trips to Zone 1 from Zone 2 are 308*0.358 = 110.

Exhibit 4-27 Example Internal Trip Attribution Distribution

Zone	I-I Attraction	Zone 1	Zone 2	Zone 3	Zone 4
1	308	79	110	25	94
2	431	110	154	34	132
3	96	25	34	8	29
4	368	94	132	30	112

The distribution of new internal-internal productions is shown in Exhibit 4-28. For example, the production trips from Zone 1 from Zone 2 are 156*0.358 = 56.

Exhibit 4-28 Example Internal Trip Production Distribution

Zone	I-I Attraction	Zone 1	Zone 2	Zone 3	Zone 4
1	156	40	56	13	47
2	218	56	78	18	66
3	49	13	18	4	15
4	185	48	66	15	56

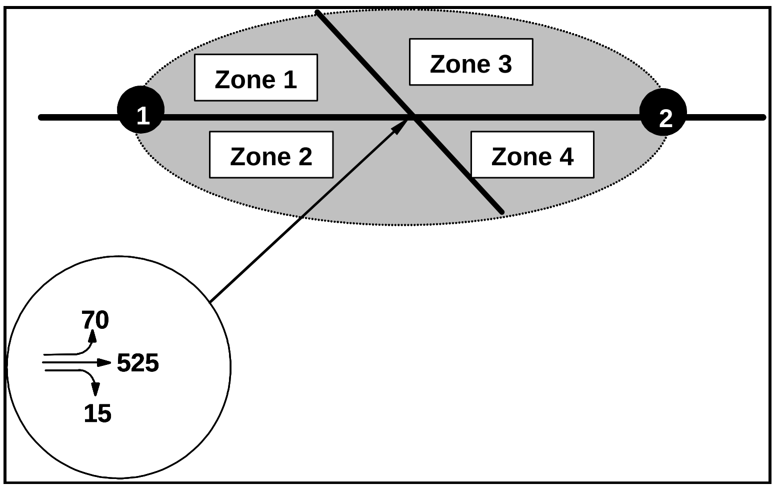
Following trip distribution, the next step in the procedure is traffic assignment, which involves assigning traffic to the road network. Trip assignment is the process used to estimate paths the trip will take, which ultimately results in traffic flow on the network. It assigns the trips to specific routes and establishes volumes on links, taking into consideration network characteristics to find the shortest path between origins and destinations. Identify the specific roadways that will be selected for each trip based on network link travel times. This step is a manual process that requires the use of engineering judgment.

Example 4-11 Zonal Cumulative Analysis – I-I Assignment

Referring to the example intersection in Exhibit 4-29, a possible set of assignment paths associated with the future eastbound through movement may be:

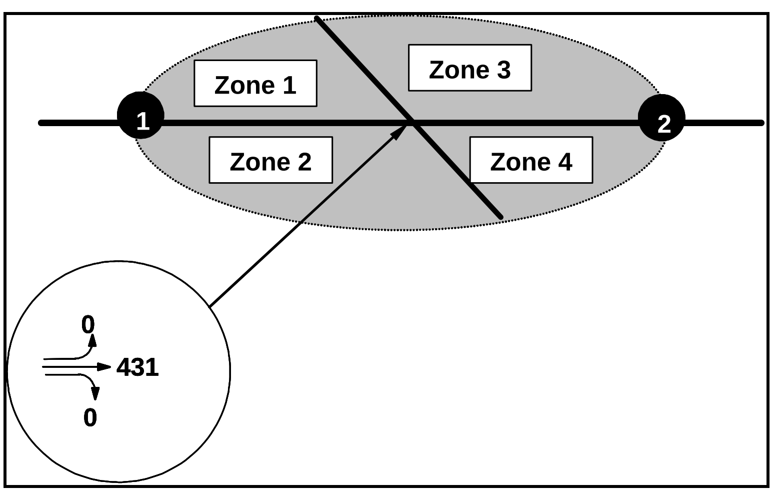
- Base-year 525 eastbound through trips.

Exhibit 4-29 Eastbound Assignment, Base Year



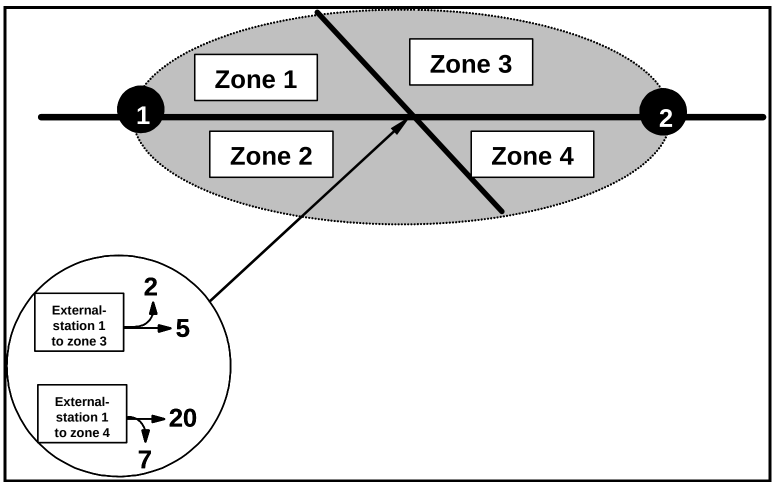
- 431 growth external-external trips. See Exhibit 4-22, column (6).

Exhibit 4-30 Eastbound Assignment, External-External



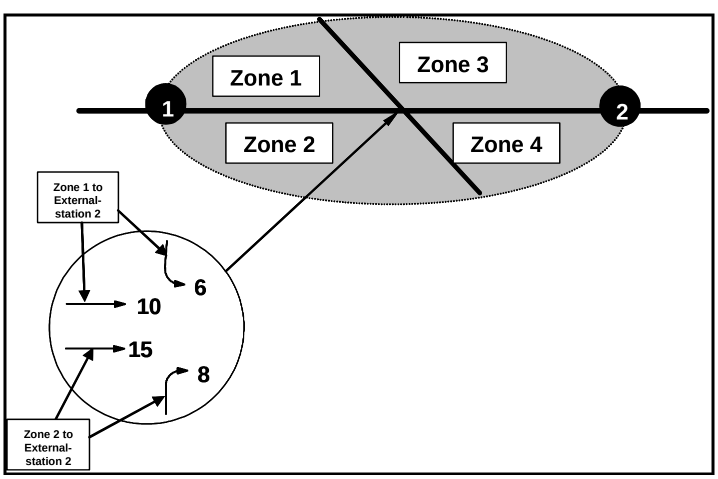
- From Exhibit 4-24, the total external-internal trips from external-station 1 to Zone 3 is 7. These seven trips can access to Zone 3 by turning left at the intersection or through downstream access via the mainline. From Exhibit 4-31, two of the seven trips will travel to Zone 3 by turning left at the intersection and the remaining five trips will travel to Zone 3 through downstream access via the mainline. The same process is followed for trips from the external-station 1 to Zone 4. And the total external-internal trips from external-station1 to Zone 4 are 27. Same engineering judgment from Zone 3 applies to Zone 4 depending on the accessibility to Zone 4. Here, assuming 20 out of 27 will access to Zone 4 by downstream accesses.

Exhibit 4-31 Eastbound Assignment, External-Internal



From Exhibit 4-25, the total internal-external trips from Zone 1 to external-station 2 are 16. The assignment of these trips depends on the accessibility from Zone 1 to the external station 2 is 16. From Exhibit 4-32, 10 of the 16 trips (depending on the accessibility from Zone 1 to the road network) will travel to external station 2 by using upstream access points. The remaining six trips will travel to external station 2 by turning left at the intersection.

Exhibit 4-32 Eastbound Assignment, Internal-External

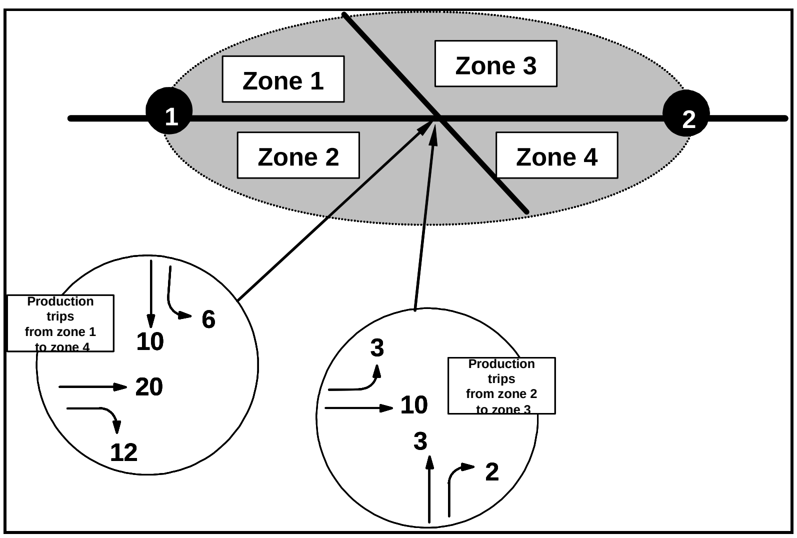


From Exhibit 4-28, there are internal-internal production trips from Zone 1 to Zones 2, 3 and 4, and from Zone 2 to Zones 1, 3 and 4. Assuming internal-internal production trips from Zone 1 to Zones 2 and 3 and from Zone 2 to Zones 1 and 4 are not gone through the intersection, there are only internal-internal production trips between Zone 1 and 4 and Zones 2 and 3. The total internal-internal production trips from Zone 1 to Zone 4 is 48 trips and from Zone 2 to Zone 3 is 18 trips.

Exhibit 4-33

- shows the internal-internal trip assignment from Zone 1 to Zone 4 and from Zone 2 to Zone 3 based on its network accessibility assumption.

Exhibit 4-33 Eastbound Assignment, Internal-Internal



So the future eastbound through movement at the example intersection is a sum of 525, 431, 5, 20, 10, 15, 20 and 10 that equals 1036 vehicles.

4.6.3 Urban Travel Demand Models

The output from urban travel demand models may be used to estimate future traffic growth. A map of the transportation models used by ODOT is available on the Planning Section website on the Transportation Modeling References webpage. Transportation models use current and projected land use and transportation network data to estimate current and future travel demand. The data is obtained from many different sources, including census data, state employment data, O-D surveys, household travel surveys, traffic counts and field surveys.

Model output can be produced for either the daily period or specific time periods throughout the day, such as the AM and PM peak hours, AM and PM peak periods and mid-day period. Seasonal adjustments should not be applied to model output. Model output cannot be directly input into traffic analysis software, because it must first be adjusted (post processed) to reflect project specific actual (counted) current traffic volumes. However, the relative difference between the model output for two scenarios (e.g., current and future conditions) can be used directly such as for the screening of preliminary alternatives.

Common Modeling Terms

- **Links** - Represent road segments and are identified by nodes at each end.
- **Nodes** - Indicate the intersections of links.
- **TAZ (Transportation Analysis Zone)** - The model or study area is broken into sections. Each of these sections is called an analysis zone.
- **Centroids (special nodes)** - They represent the center of an activity zone called a TAZ. This is not necessarily the geometric center of the zone.
- **Centroid Connectors -** Links that connect centroid nodes with the model network. These can represent local streets not included in the model network. Centroid Connectors provide the linkage between the trips associated with the TAZ land uses and the roadway segments (or links).
- **Screenlines** - Imaginary lines that are strategically drawn across network links. The volumes on the links crossed by the screenlines are summed. One use of a screenline might be to compare the volume of traffic entering and leaving the study area for each alternative.
- **Cordon** - An imaginary boundary (non-linear) strategically drawn across an area. The volumes on the links crossing the cordon are typically summed to understand the amount of trips entering and exiting an area.
- **Model Volumes and 30 HV Volumes** - These two volumes cannot be compared directly. Models are mathematical representations of the population and employment data that is arranged by TAZ. What matters is the relative (proportional) change between two sets of model data. This change is applied to the field data. This is what is meant by post-processing. Post-processing is basically applying proportions and percents.

<u>Growth Pattern Types</u>

Growth pattern types can all be present in a model in different areas. The analyst must have knowledge of the project area and what the model assumes for the study area in order to properly pick the future growth curve. The growth curve can be a combination of two conditions on the overall timeline. For instance, for the first 10 years the growth is linear, but the last 10 years the growth is geometric. Growth curves can also be estimated by using a combination of differently sloped lines (piecewise, such as linear with different growth rates).

- **Exponential (compound)** - Compound growth is typically associated with brand new growth in an area that has plenty of land and road capacity. This is typically limited to five years or less. Use of an exponential curve over a prolonged period can seriously overestimate future growth.
- **Straight-Line** - Steady growth over time.
- **Logarithmic (Decelerating)** - Growth tapers off as land approaches built-out status and

capacity of roadways. Future growth is mainly contributed by growth in background (through) traffic.

Centroid connectors can be useful in determining detailed growth estimate data. These connectors provide loading points from the TAZ's to the model network. These also represent the local streets that are not in the model. The growth rates determined from the centroid connectors can be applied to obtain future volumes on streets/driveways not in the model. This technique should be used with caution, however, as centroid connectors may represent multiple streets and are not calibrated.

Note: The following is applied on a directional link basis. Post-processing travel demand model data is applied on each directional link within the study area, so the use of spreadsheets is **highly** recommended.

The base year of model used should be adjusted to conform to base year of project by interpolating between the model base and future scenarios. This can be generally done with straight-line growth as the adjustments are short. The base and future are compared to produce a (typically) 20-year growth factor, which is reduced to a yearly growth factor. The ideal situation is to have the base year of the project and the model base year the same.

Example 4-12 Model Year Adjustment – Straightline Method

Yearly model growth = ((2020 model/ 2000 model) -1) / 20

2004 model = ((yearly model growth * 4) + 1) * 2000 model

Once the base model year is adjusted, then the future model year can be calculated. Ideally, the future model year is the same as the project future year. If future model year and project future year are not the same, use the appropriate growth pattern and resulting curve equation to extrapolate out to the future model year. Models are generally updated every five years. This is about the limit for extrapolating beyond the future year, otherwise the chance that zones will be "overbuilt" quickly increases.

However, in undeveloped territory and/or built-out areas another curve growth method may be more appropriate.

Example 4-13 Model Year Adjustment – Geometric Method

Use geometric growth to calculate 2025 DHV from 2004 adjusted model data and 2020 future model data.

Geometric growth equation: F = P(1+i)n

F is the future year total volume; P is the existing year volume; i is the growth rate; and n is the number of years between the existing year and the projected future year. The n can be also be adjusted to any intermediate future year to provide the future year volume.

$2025 \text{ model} = 2004 \text{ model } (1+ i_{16})^{21}$
$i_{16} = ((2020 \text{ model}/2004 \text{ model})^{1/16}) - 1$

4.6.4 Model Post Processing

Post-processing is a method for developing future traffic volumes based on traffic counts and the relative differences between model scenarios. The entire post-processing procedure and guidelines are explained in detail in the NCHRP Report 255, which is available on the Transportation Modeling References webpage on the TDD Planning Section website. The basis of post-processing is to multiply current counted traffic volumes by the relative difference between current and future modeled volumes in order to obtain adjusted future volume estimates.

There are multiple methods available to convert model volumes into design hour volumes, but the two most common are the growth method and the difference method. See NCHRP Report 255.

Growth Method
The Growth Method uses growth equations to calculate future design hour volumes. Caution should be used with the method as it may severely overestimate growth on links that have little volume in the base year and significant volume in the future year. The basic form of the growth method (per NCHRP Report 255) is:

Future model year / Base model year = Future DHV / Base 30 HV

Example 4-14 Post-Processing – Growth Method

- Example: 2025 DHV = (2025 Model / 2004 Model)* 2004 30 HV
- If the 2025 Model had 800 vph, the 2004 Model had 50 vph and the 2004 30 HV had 550 vph, the growth method using a simple linear method would be:

2025 DHV = (800/50)* 550 = 8,800 vph

- Using a geometric form the equation would be:

2025 DHV = 2004 30 HV * (i16 + 1) ^ 21

Difference (Incremental) Method
The Difference (Incremental) Method should be used in areas where large differences in base and future model link volumes exist. This method is preferred in NCHRP Report 255. The basic form of the difference method is:

Future DHV = Base 30 HV + (Future model year – Base model year)

Example 4-15 Post-Processing – Difference Method

2025 DHV = 2004 30 HV + (2025 Model -2004 model)

Using the same numbers from the growth method above, using the difference method would be:
2025 DHV = 550 + (800 – 50) = 1,300 vph.

Both methods should be compared in a spreadsheet on a directional link basis using percent and absolute difference. Areas with large percent and absolute differences should use the difference method. Areas that compare favorably can use one or the other or an average of the two. The analyst will need to analyze the data to find the natural breakpoints of when and where to use each method.

The two previous examples show the large difference between the growth and difference methods where the growth method overestimates the future 2025 volume because the base year model volume is much smaller than the future model year volume. In this case the difference method would be used.

If the 2004 Model had 600 instead of 50, the growth method would result in 733 vph and the difference method would result in 750 vph. When the base and future models are closer, the results from the two methods are much closer. In this case, the growth method or an average of the growth and difference method would be used.

Link growth rates can be averaged together to reduce the number of calculations/adjustments necessary if they are close together (less than 10%).

Screenlines

Screenlines can be used for calculating overall growth rates or used for calculating volumes on new links (links that only exist in one of the scenarios that you are comparing).

Screenlines are useful where there are significant differences in growth within the study area. Screenlines should be strategically placed to cross the major links of the different growth areas. The same screenlines are drawn for the base model year as the future model year on the model volume plots. The link volumes crossed by each screenline are summed. The future summation of each screenline is divided by the corresponding base screenline summation. This provides the growth rate for the different areas cut by the screenlines.

The main use of screenlines is to determine the future design hour volume of links that exist only in one scenario. This comes up when a new route is added to a scenario. This can occur with both no-build and build alternative scenarios.

Latent Demand

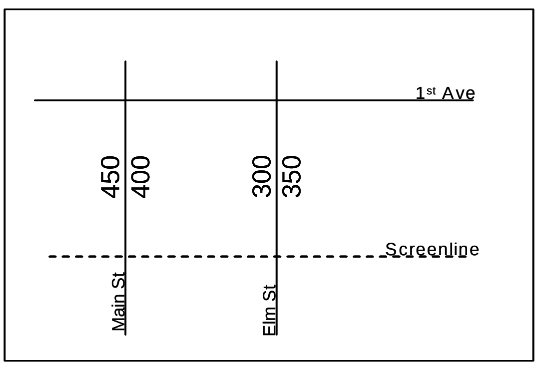
An effect that may be observed from model results is where the Future Build volume is significantly greater than the future No Build volume, for the same year. This can occur where the Future No Build demand has exceeded capacity, and a portion has shifted to avoid congestion. The shifted demand is called latent demand. Demand can shift in a variety of ways, for example to other routes, modes, destinations, or time periods. Once the facility is at capacity, peak hour volumes no longer increase over time, while latent demand may continue to increase. When a Build alternative alleviates the congestion, a portion of the shifted demand may return, which is reflected by an increase in the Future Build volume.

Example 4-16 Post-Processing – Use of Screenlines

For example, assume a roadway network in the future no-build year had two north-south links at Main Street and Elm Street, but a build alternative added a new north-south link at Oak Street for a total of three north-south links. The analyst has the model outputs for the scenarios with and without the new connection and the future no-build DHV diagram (without the connection).

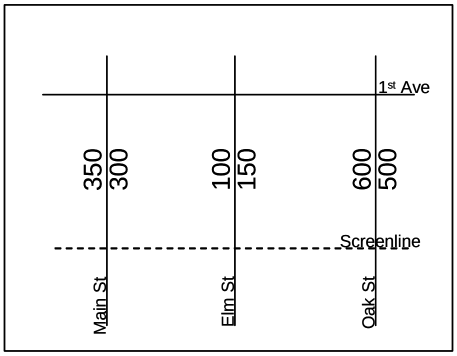
1. Draw a screenline across Main and Elm Streets in the future no-build model scenario, and sum up the future no-build model volumes for each street as well as the total north-south future model volume.

Future No-Build Model Scenario



2. Draw a screenline across Main, Elm and Oak Streets in the build alternative scenario and sum up the build alternative model volumes for each street as well as the total north-south build alternative model volume. Also calculate the directional (northbound and southbound) splits for all three streets.

Build Alternative Model Scenario



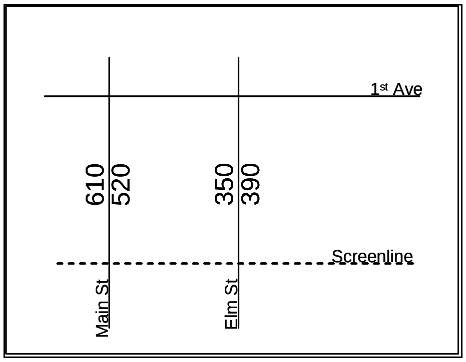
Main Street Total = 350 + 300 = 650 vph
Elm Street Total = 100 + 150 = 250 vph
Oak Street Total = 600 + 500 = 1100 vph
Total North-South Volume = 650 + 250 = 2000 vph

Note: in this example, the build alternative scenario pulls in 25% more traffic (2000 vph vs. 1500 vph) than the future no-build model scenario. This is a result of previously diverting traffic returning to the route that it wants to use. In this case the analyst must use the build alternative model scenario to create the future design hour volumes. If the difference was less than 10%, then the analyst could use the future no-build volume distributed on the build network.

Directional Splits Calculation:
Northbound Main Street = 300/650 = 0.46
Southbound Main Street = 1 – 0.46 = 0.54
Northbound Elm Street = 150/250 = 0.60
Southbound Elm Street = 1 – 0.60 = 0.40
Northbound Oak Street = 500/1100 = 0.45
Southbound Oak Street = 1 – 0.45 = 0.55

3. Draw a screenline across Main and Elm Streets in the future DHV diagram, and sum up the total future DHV volumes for each street as well as the total future north-south DHV volume.

Future No-Build DHV



Main Street Total = 610 + 520 = 1130 vph
Elm Street Total = 350 + 390 = 740 vph
Total North-South Volume = 1870 vph

4. Calculate the total north-south build alternative DHV by creating a ratio by dividing the build alternative grand total (Main/Elm/Oak Streets) by the future grand total (Main/Elm Streets) and then multiplying this ratio with the future DHV (Main/Elm Streets).

   General equation form:
   Build Alternative Model/Future No-Build Model = Build Alternative DHV/Future No-Build DHV

   Modified equation to solve for the total Build Alternative DHV:
   Build Alternative DHV = (Build Alternative Model/Future No-Build Model) x Future No-Build DHV

   Build Alternative DHV = (2000/1500) x 1870 = 2493 vph

5. Calculate the percentage splits for each of the build alternative north-south links.

   Main Street split = 650/2000 = 0.325 (32.5%)
   Elm Street split – 250/2000 = 0.125 (12.5%)
   Oak Street split – 1100/2000 = 0.55 (55%)

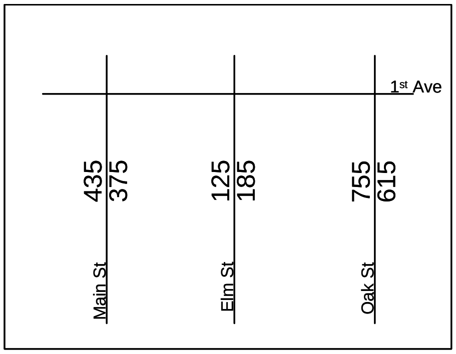
6. Apply the percentage split for the new link (Oak Street) to the future build alternative grand total calculated in Step 4. This will determine the total build alternative DHV on the new link (Oak Street). Repeat for the other north-south screenline links (Main and Elm Street)
   Build Alternative DHV (Oak) = Oak Street split (from Step 5) x Build Alternative DHV total (from Step 4)

   Build Alternative DHV (Oak) = 0.55 x 2493 = 1371 vph
   Build Alternative DHV (Main) = 0.325 x 2493 = 810 vph

Build Alternative DHV (Elm) = 0.125 x 2493 = 312 vph

7. Apply the directional splits from Step 2 to the DHV totals for each street calculated in Step 6. This will compute the directional DHV for each street.

Future Build DHV



Main Street DHV (NB) = 0.46 x 810 = 373 vph
Main Street DHV (SB) = 0.54 x 810 = 437 vph

Elm Street DHV (NB) = 0.60 x 312 = 187 vph
Elm Street DHV (SB = 0.40 x 312 = 125 vph

Oak Street DHV (NB) = 0.45 x 1371 = 617 vph
Oak Street DHV (SB) = 0.55 x 1371 = 754 vph

Turn Movements

<u>Future No-Build</u>

Once the growth rates have been determined and applied to the project roadway network the future turning movements need to be calculated. The volume of traffic entering each intersection must balance with the traffic leaving the intersection. Typically the no-build turning movement volumes are the same as the existing year movements with a growth factor applied. Impacts from other projects should show up in the future no-build model run, and would impact the growth rate or methodology used to calculate the future no-build design volumes.

There are times when another project is expected to impact the travel patterns in the study area, and the turning movements would have to be manually adjusted to reflect this. The turning movements provided by the model can help point out some of these impacts. Note: A capacity constrained demand model should indicate the shift in travel patterns and the directional link volumes from the model should be used as a starting point to arrive at a future DHV, arriving at a post-processed set of volumes requires a method such as described above or in NCHRP 255. The origin and destination matrix can also be a helpful tool to obtain the distribution of trips between zones. Model runs with and without committed/STIP projects can be run to determine the impacts, if any, from nearby financially constrained future projects.

Future Build

At this point the directional design hour volumes have been calculated for each link so the intersection approach volumes are known. The volume of traffic entering each intersection should balance with the traffic leaving the intersection.

Build turning movements are a combination of considering the known travel pattern changes and the existing turning movements. The model-assigned turning movements can be used as a starting place if the movements are converted to percentages. In difficult cases the origin and destination matrix (or select link plots, discussed in the next section) can be requested from the modeler. Developing the build turning movements in some cases is not a straightforward process. It may involve looking at all the possible movements and using your best judgment and knowledge of the area. Changes are also made as the intersection is balanced to hold the link volumes. The link volumes need to be held as much as possible to preserve the future growth ratios.

There are matrix-based programs that assign turning volumes based on the link volume. These automate the NCHRP Report 255 Chapter 9 turning movement process, which can save considerable time over a hand calculation. The intersection approach volumes and the exiting volumes are entered. The programs typically assume that more traffic is attracted to the higher volume links and will weight the movements accordingly. The program goes through an iterative process to closely match the data that was entered. All matrix programs use the NCHRP 255 method, which is the Fratar Method of balancing volumes.

Select-Link/Select-Zone Plot

Models are much more useful than just obtaining future growth rates. A select-link or select-zone plot can be requested. Select-link/select-zone plots show the distribution of volume from/to a given link or zone across the model network. This is useful in determining, for example, volume distribution for a future development not included in the model. Another example is using select-links to determine the percentage of through trips through the project area. They also can be used to help show the analyst if trip assignments are reasonable after network changes are made.

5 ASSESSING PERFORMANCE

5.1 Purpose

The analysis of the existing transportation system uses the data collected and developed as described in previous chapters to evaluate the transportation system prior to making any changes to land use or infrastructure. This chapter presents analysis procedures not specific to traffic flow type and identifies specific methodologies and input parameters to be used on ODOT projects. Topics covered include:

- Crash Analysis
- Peak Hour Factors
- Access Management
- Conflict Points
- Sight Distance
- Multi-Modal Analysis
- Other Analysis Issues/Procedures

5.2 Crash Analysis

Crash analysis typically involves the identification of the problem areas on facilities experiencing an above-average frequency of crashes or reoccurring crash patterns and an investigation of conditions that may contribute to the problem identified. If an analyst generally understands crash trends within a study, the analyst can use the information in the analysis and recommendations. This analysis should not be confused with the operational analysis that would be done separately by Region Traffic staff.

5.2.1 Calculating Crash Rates

Crash rates are commonly used to determine if the frequency of crashes experienced at a given intersection or segment of roadway is above average. Because the total number of crashes experienced on a segment or intersection of roadway(s) is typically proportional to the number of vehicles using that facility, rates are often calculated to allow for comparisons of different facilities. The most common basis for comparison is to calculate the number of crashes experienced per million users. Specifically, for roadway segments, the number of crashes per million vehicle miles of travel (MVM) is calculated and for intersections, the number of crashes per million entering vehicles (MEV). These rates can also be calculated using only specific crash types such as fatalities, fatal, injury or property damage only crashes. The corresponding formulas for these calculations are shown in Exhibit 5-1.

Exhibit 5-1 Equations for Crash Rate Calculations

Description	Expression	Formula
Segment Crash Rate (crashes per million vehicle miles of travel, MVM)	Annual number of crashes times one million, divided by the annual vehicle-miles of travel.	$\frac{\text{Annual number of crashes x } 10^6}{\text{(AADT) x (365 days/year) x (segment length in miles)}}$
Intersection Crash Rate (crashes per million entering vehicles, MEV)	Annual number of crashes times one million, divided by the annual volume of entering traffic.	$\frac{\text{Annual number of crashes x } 10^6}{\text{(AADT) x (365 days/year)}}$

Note that care should be taken when calculating crash rates for segments that are less than one mile. The resulting rate for a short section can appear to be much higher than is actually the case. Evaluate crash rates for short sections by trying small changes in the number of crashes. If the changes in crash rate are dramatic then either this type of crash rate should not be used or its use should be accompanied with a warning. Whenever possible crash rates for short sections should be normalized (or lengthen the section) to a full one-mile section without including features that will significantly influence the outcome, e.g., a major intersection hosting a high concentration of crashes.

For reporting crash rates on state highways, ODOT uses the segment crash rate. ODOT does not have an established standard intersection crash rate to compare with as a baseline. The ODOT CAR Unit publishes an annual document called the Oregon State Highway Crash Rate Tables,

available at: http://www.oregon.gov/ODOT/TD/TDATA/car/CAR_Publications.shtml.
In this document crash rates for given segments of all state highways are calculated and listed for each of the last five years. In addition to this a variety of summaries of crash rates for state highways considering fatalities and different highway types, as well as information about the data used in the crash rate calculations, is provided.

Of particular interest is Table II, which shows statewide average crash rates for each of the last five years, for freeways and non-freeways on the state highway system, by urban and rural area and by primary and secondary designation. This table is often used in crash analysis to compare the segment crash rate calculated for a study highway to the statewide average rate shown in the table for a comparable highway type. In the selection of the appropriate highway type for comparison, the analyst must determine whether the study highway segment is classified as a freeway or non-freeway, is located within an urban or rural area and is on the primary or secondary highway system (a listing of primary and secondary highways is included after Table IV). Note that the category "State Highway System" provided alongside the primary and secondary system categories in Table II is merely a combination of the primary and secondary highway systems and should not be used for most crash rate comparisons.

Example 5-1 Crash Rate Calculation and Comparison

A principal highway segment in a rural area has experienced 22 reported crashes over the last 3 years. The segment ADT is 23,000 and length is 1.6 miles.

$$\text{Rate} = \frac{\text{Number of Crashes X 1,000,000}}{\text{Length (in miles) X ADT X (Yrs X 365)}}$$

$$= \frac{\text{22 X 1,000,000}}{\text{1.6 X 23,000 X 3Yrs X 365}}$$

$$= \text{0.55 Crashes per Million Vehicle Miles (MVM)}$$

As shown in in the table below, the statewide average crash rates are as follows:

2005	0.67
2006	0.69
2007	0.68
Average	0.68

The segment crash rate of 0.55 is less than the average statewide rate of 0.68.

Statewide Crash Rate Table

TABLE II: FIVE-YEAR COMPARISON OF STATE HIGHWAY CRASH RATES

Table II presents a comparison of state highway crash rates for the past five years, for urban and rural areas, by functional classification. Mileage is shown for the current data year only.

See Table IV for information on official highway mileage and VMT data.

JURISDICTION AND FUNCTIONAL CLASSIFICATION	MILES*	2007 Rate	2006 Rate	2005 Rate	2004 Rate	2003 Rate
TOTAL STATE HWY SYSTEM	**7,455.23**	**0.85**	**0.85**	**0.87**	**0.79**	**0.98**
Interstate Freeways	729.57	0.38	0.39	0.41	0.38	0.42
Other Fwys/Expressways	52.26	0.73	0.78	0.80	0.78	0.87
Non-Freeways (combined)	6,673.40	1.27	1.26	1.25	1.13	1.45
Other Principal Arterials	3,281.04	1.28	1.29	1.28	1.16	1.52
Minor Arterials	1,964.61	1.20	1.14	1.14	1.02	1.20
Urban Collectors	8.69	1.10	0.68	1.19	1.23	2.08
Rural Major Collectors	1,382.20	1.25	1.11	1.14	0.93	1.25
Rural Minor Collectors	36.86	0.64	0.66	1.30	0.32	1.30
Rural Local	0.00	0.00	16.52	4.23	2.68	8.06
Rural Areas	**6,414.01**	**0.58**	**0.58**	**0.59**	**0.51**	**0.59**
Interstate Freeways	539.37	0.28	0.29	0.31	0.25	0.26
Non-Freeways (combined)	5,874.64	0.79	0.77	0.77	0.68	0.80
Other Principal Arterials	2,658.39	0.68	0.69	0.67	0.61	0.71
Minor Arterials	1,839.04	0.99	0.93	0.98	0.83	0.97
Rural Major Collectors	1,340.60	1.24	1.08	1.10	0.92	1.20
Rural Minor Collectors	36.61	0.69	0.36	1.40	0.35	1.40
Rural Local	0.00	0.00	16.52	4.23	2.68	8.06

* Couplet and Roadway 3 data are included. Frontage road and connection data are excluded.

When comparing a statewide average rate to a segment crash rate for a study highway, simply exceeding the statewide average rate should not be interpreted as proof that a section is hazardous. A segment crash rate that exceeds the statewide average crash rate should merely be considered as an indication that further investigation is necessary. It should also be stated that cost effective improvements to increase safety could still be identified even with a segment crash rate lower than the statewide average.

When an intersection crash rate may be appropriate to report, a rule of thumb is that intersections with a crash rate of 1.0 or greater is generally considered to be an indication that further investigation is warranted. . This is not to say whether a location is "bad" if over or "okay" if under 1.0. It should also be stated that cost effective improvements to increase safety could still be identified even with an intersection crash rate lower than the statewide average.

Another analysis tool that ODOT uses is the Safety Priority Index System (SPIS) which provides an alternative method of ranking for intersections and segments of roadways on State Highways. SPIS incorporates crash rate, frequency and severity components to provide a single index to compare a roadway or intersection.

The top 5% SPIS ranking requires the Region Traffic offices to conduct a safety investigation each year to determine if there is an appropriate safety improvement fix to the problem. The SPIS ranking can be determined by contacting the appropriate Region Traffic Office for assistance or at the following intranet website http://intranet.odot.state.or.us/tstrafmgt/PSMS/SPIS/spis.htm. The Traffic-Roadway Section is contracting with Oregon State University and Portland State University to develop a Safety Investigations Manual which is planned to be available in Fall of 2009.

5.2.2 Identifying Crash Patterns

The commonly used procedure to identify crash patterns is categorizing crashes by characteristics such as types, time of day, weather conditions and locations. In this form it may be easier for the analyst to identify crash trends such as a high number of a certain type of crash or a location or movement that experiences a disproportionate amount of the total crashes.

Caution should be exercised when identifying actual crash locations from reported data. Crashes may be reported at the nearest integer milepoint even if they occurred hundreds of feet away, i.e., a crash reported at milepoint 12 even though it actually occurred at milepoint 12.34. This can be evidenced by clusters of crashes reported at even milepoints. Crash data should be checked for discrepancies such as where a crash occurred on a curve, but the reported milepoint is located on a straightaway section.

5.2.3 What Data to Report

An analysis report should contain summarized information about crashes within the study area. The summary should contain trends, crash rates and a general discussion of the crashes. If on the state highway the report should contain the segment crash rate and reference to the most recent SPIS data.

5.2.4 Countermeasure Selection

While crash patterns found should be addressed during the alternatives analysis, countermeasure selection is generally not addressed in most transportation analysis projects, but is conducted as a separate effort that may be initiated by the identification of crash patterns in the project report. However, the following discussion will help provide an understanding of the potential uses of crash analysis. Countermeasures can generally be grouped into three categories: education, enforcement and engineering.

- Education related countermeasures include a variety of public information campaigns using a broad range of media to reach a target audience. These types of countermeasures can be effective in reducing driver error by making motorists aware of the risks and consequences of certain driving behaviors and environments encountered.
- Enforcement countermeasures typically involve increased policing activity to encourage compliance with existing traffic controls and regulations. Increased enforcement is often implemented when engineering countermeasures are already present, but have become ineffective due to frequent violations by motorists. The application of enforcement

countermeasures is typically carried out by the Region Traffic Section.

- Engineering countermeasures include a broad range of improvements or modifications to the transportation system in an attempt to improve roadway safety. Such countermeasures may include geometric improvements, ITS applications, changes to traffic controls (signing, striping, signals, etc.), changes to roadway surfacing, or operational enhancements.

There are many resources that provide potential countermeasures to select from for a given crash pattern. When selecting countermeasures, the analyst should coordinate with the ODOT TRS.

5.3 Peak Hour Factors

Peak hour factors (PHF) are used to account for the non-uniformity of traffic flow within the peak hour by converting hourly volumes to peak flow rates associated with a selected interval of time within the peak hour. The most common interval of time selected for traffic analysis is the peak 15 minutes. In areas near capacity the peak 15-minute flow can cause up to several hours of congestion. This typically happens when the demand exceeds the available capacity of the transportation system resulting in "peak hour spreading", which is the extension of the peak period caused by a system breakdown. Therefore, it is often essential that the transportation system be designed to accommodate the peak 15 minutes of the peak hour.

Peak hour factors should be applied in most capacity analyses in accordance with the *HCM*, which selected 15-minute flow rates as the basis for most of its procedures. It is especially critical to examine the peak 15-minute period when potential queue lengths may become an issue, and at locations with sharp peaking characteristics such as employment sites and locations with low peak hour factors (less than 0.90).

5.3.1 Calculation

The PHF is typically calculated using data from traffic counts. It is the traffic volume during the peak 60-minute period divided by four times the volume during the peak 15-minute period.

$$PHF = \frac{\text{Volume During Peak 60-Minute Period}}{4 \times (\text{Volume During Peak 15-Minute Period})}$$

5.3.2 Existing Conditions

PHFs calculated from actual traffic count data should always be used for analysis of existing conditions. For all applications other than sketch planning-level analysis, traffic count data should be obtained in 15-minute intervals, and one of the three methods for calculation and application of PHFs described below should be followed. Each of the methods should be reviewed and the method that best represents the conditions should be used. In the following methods the system peak hour is first selected, and PHFs are calculated within that hour. For sketch planning-level analysis where traffic counts are not provided in 15-minute intervals, the *HCM* suggests the following defaults:

- 0.95 for congested conditions
- 0.92 for urban areas
- 0.88 for rural areas

- **Existing PHF – Method 1**: This analysis method uses an intersection PHF to estimate peak 15-minute period equivalent hourly flow rates from the peak 60-minute period volumes. The peak 15-minute period with the highest intersection total entering volume (TEV) should be used to determine the PHF for each intersection. The application of global PHF's is generally not appropriate when count data is available. The intersection PHF is calculated as follows.

- **Step 1**: Determine the peak 15-minute period that has the highest intersection total entering volume (TEV).
  - **Step 2**: Calculate the intersection PHF based on the time period determined in Step 1, by dividing the TEV peak 60-minute volume by four times the TEV occurring during the peak 15 minutes.
  - **Step 3**: In the analysis, apply the intersection PHF from Step 2 to each movement peak 60-minute volume.

- **Existing PHF - Method 2**: As an option, in cases where unusual peaking occurs on individual approaches, approach PHFs can be determined from the traffic count volumes. The peak 15-minute period with the highest intersection TEV should be used to determine the PHFs. PHFs are calculated for each approach or movement as follows. If an approach or movement PHF is calculated to exceed 1, entering a value of 1.00 will ensure a slightly conservative analysis.

  - **Step 1**: Determine the peak 15-minute period that has the highest intersection total entering volume (TEV).
  - **Step 2**: Calculate the PHF for each approach or movement based on the time period determined in Step 1 by dividing the approach peak 60-minute volume by four times the approach or movement peak 15-minute volume.
  - **Step 3**: In the analysis, apply the approach or movement PHFs from Step 2 to the approach or movement peak 60-minute volumes (usually calculated by the analysis software).

- **Existing PHF - Method 3**: As an additional option in cases where unusual peaking occurs on individual approaches, the traffic count volumes for all movements that occur during the single peak 15-minute period can be used directly in software that multiplies the peak 15-minute period volumes by a factor of four. If this method is used both the actual 60-minute period hourly volumes and the equivalent peak 15-minute hourly flow rates should be shown on the Existing Traffic flow diagrams, and clearly labeled to avoid confusion.

  - **Step 1**: Determine the peak 15-minute period that has the highest intersection total entering volume (TEV).
  - **Step 2**: For the time period determined in Step 1, enter the peak 15-minute volumes directly in the software.
  - **Step 3**: Select software analysis procedure based on the peak 15-minute period.
  - **Step 4**: On the flow diagrams show and clearly label both the actual 60-minute period hourly volumes and the equivalent peak 15-minute hourly flow rates to avoid confusion.

5.3.3 Future Conditions

Because traffic flow patterns may change over time and future conditions can not be directly measured, analysis of future years should incorporate the following default values by approach for the PHF unless better information is available:

- 0.85 for minor street inflows and outflows
- 0.90 for minor arterials
- 0.95 for major streets

Engineering judgment must be used in the selection of PHFs for future years. In cases where the existing PHF is higher than the default value for the future PHF, it may be appropriate to retain the existing value for the future year, as PHFs do not typically decrease as traffic volumes and congestion increase. Likewise for areas that have low existing peak hour factors, using the future PHF default values could produce results that would underestimate the future traffic conditions. For areas with aggressive traffic demand management strategies contained in an adopted plan, a different PHF (to reflect spreading of the demand) may be used for future year analysis if agreed to by ODOT during the scoping process. For areas with pronounced peaking characteristics, such as industrial sites and schools, PHFs lower than the default values listed above should be used.

5.4 Access Management

Access management includes the spacing, design, operation and control of all public and private approaches (driveways, streets, ramps, etc.) to a roadway in a manner that balances the competing needs of property access with safe and efficient travel on the transportation system in accordance with pre-established management objectives. These management objectives typically reflect a functional hierarchy where facilities such as freeways and major arterials give priority to through travel, while minor collectors and local streets provide for more direct property access.

5.4.1 Impacts of Access Management Implementation

Because every approach to a roadway creates new conflict and decision points, the reduction and orderly provision of access can have direct benefits to safety and the efficient flow of traffic. Drivers are more likely to have a collision when required to react to multiple conflicts. Numerous studies from around the nation have shown that reducing access density reduces crash rates.[4] Therefore, the implementation of cost-effective access management techniques, such as median construction, left turn prohibitions, and approach consolidation, can extend the life of existing transportation facilities delaying the construction of more expensive improvement projects and preserving the investment in the infrastructure.

Some of the benefits of access management include:

- Reduction in vehicular crash rates.
- Improved travel efficiency with fewer delays.
- Enhanced bicycle and pedestrian environment with fewer vehicle conflicts.
- Improved pedestrian crossings where medians can be used as refuges.
- Improved movement of goods and services.
- Reduction in vehicular emissions.
- Improved fuel efficiency.
- Avoiding impacts to private properties from roadway widening.
- Efficient customer access to adjacent businesses.

5.4.2 ODOT Access Management Policies: 1999 Oregon Highway Plan

ODOT’s policies regarding access management are published in Goal 3 of the 1999 Oregon Highway Plan (OHP), which includes the establishment of a highway classification system and corresponding management objectives and spacing standards, direction for the use of medians, special consideration for interchange areas, and an allowance for deviations and appeals. These policies were used as the basis for the rules and procedures adopted for the governing of highway approaches in OAR 734-051.

[4] *Access Management Manual*, Transportation Research Board, Washington, D.C., 2003, p. 15.

5.4.3 OAR 734-051: Highway Approaches, Access Control, Spacing Standards and Medians

The administrative rules contained within OAR 734-051 govern all approaches in existence and approach applications filed after March 1, 2004. While these rules are based on the policies from the 1999 OHP, they have the force and effect of law and shall overrule the policies should any conflict arise. The rules contain detailed procedures and criteria that shall be followed for all decisions affecting highway access including approach applications, site plan reviews and project development.

5.4.4 Evaluation of Existing Access Conditions

The extent to which existing access conditions are evaluated may depend on the type of project. An evaluation of access conditions as part of a planning study, such as a transportation system plan or refinement plan, may be conducted at a lower level of detail if recommendations for improvements are not intended to be carried out immediately. However, for development review and transportation facility projects, where action will occur within a time frame of less than 5 years, a greater amount of information will be needed.

Long-Range Implementation: Planning Level Evaluation
For projects such as transportation system plans, corridor plans and refinement plans, recommendations regarding access management are somewhat conceptual and generally focus on long-range implementation over a 20-year planning period. Detailed information about individual properties is not necessary since things like property ownership and site circulation may change before any action is taken. For these types of projects existing approach spacing and local street connectivity are the primary elements of interest.

For a given corridor, the number of existing approaches and the distances between them should be recorded and compared to the applicable access management spacing standards for that facility. It may be easier to display this information by presenting it as an access density, where the number of existing access points per mile is compared to the maximum access points per mile that would be allowed under the access management spacing standards. In addition, any physical restrictions such as topography, waterways and historic features that may limit the ability to meet access management spacing standards should be recognized.

The condition of the existing local street system should also be examined, because improved connectivity can divert trips away from the highway and provide alternate means of property access. Identify the number and location of public street approaches to the highway and compare it to what would be allowed by the applicable access management spacing standards. If some public approaches are too close together, look for opportunities to consolidate them or increase the distance between them. Opportunities to construct parallel roadways (e.g., frontage roads) and new public approaches that would have adequate spacing and could provide shared access to multiple properties should also be explored. In addition, existing public approaches maintaining poor alignment or inadequate sight distance should be identified for improvement by future projects.

Short-Range Implementation: Land Development and Transportation Projects

For projects where some or all recommended actions on individual approaches will be carried out within 5 years or less, a significant amount of information will be needed to assure each property is addressed appropriately. These types of projects typically include land development proposals, transportation facility projects, access management plans (AMPs) and interchange area management plans (IAMPs). The types of information needed can generally be categorized as either physical characteristics or access rights. Work with the District permits personnel to get detailed data.

As with the long-range planning projects, when assessing existing conditions, the existing access spacing should be compared to the applicable access management spacing standards, and potential improvements to the local street system should be considered. In addition, opportunities to make immediate improvements to property access must be explored, such as the elimination of approaches, relocation of approaches to non-state facilities and the establishment of shared approaches between multiple properties.

For more detailed recommendations and requirements regarding the treatment of access proposals, ODOT's Access Management Manual and PD-03, an Operational Notice titled "Project Development Access Management Sub-Teams," which has been included in the Access Management Manual, should be consulted. Adopted AMPs and IAMPs should also be consulted, if they exist.

5.5 Conflict Points

Introduction

Every roadway access creates conflict points for drivers, pedestrians, and bicyclists. Conflict points are locations where one vehicle path impacts another. Each conflict point is a possible crash location. Crashes occur at conflict points when one roadway user fails to yield to another. The crash potential associated with each conflict point varies depending on the complexity and volume of the movements. Multi-lane highways increase the number of conflict points as well as the crash potential because of the increased exposure area, exposure time, and potential for obstructed sight distance by vehicles in adjacent lanes. Reducing conflict points decreases crash potential. Conflict points are classified as diverging, merging, weaving, turning, and crossing.

Crossing paths are major conflicts. Diverging, merging, and weaving paths are minor conflict points. Diverging conflicts occur where one path separates into two. Merging conflicts occur where two paths come together. Weaving conflicts involve vehicles changing paths. Both major and minor conflict points may occur at high speeds, but minor conflicts typically involve vehicles traveling in the same direction. Vehicles crossing paths at high speeds may not have the sight distance or ability to minimize the severity of the crash.

Turning and crossing conflicts can also involve pedestrians and bicyclists. A crossing conflict point, which includes pedestrians and bicyclists, occurs where a vehicle path passes through a crosswalk or bike path; a turning conflict point occurs where the turning vehicle path passes through a crosswalk or bike path. Pedestrian crashes are not limited by locations. The pedestrian - vehicle conflicts are counted separately from vehicle - vehicle conflict points.

This section is a reference only. Every intersection is unique and must be analyzed appropriately. The analysis should consider geometry, permitted turn movements, the level of control of non-permitted movements, and the number of lanes for each movement. Reducing conflict points allows drivers to move through an area with less distraction so that traffic flows smoothly at constant speeds. With fewer conflict points, drivers can better maintain their attention on roadway conditions.

Conflict points can be reduced through three measures:

1. Limit the number and/or type of access points
2. Install medians, channelization, and other control devices (e.g., roundabouts) to restrict or control turning movements
3. Grade separate traffic flows

Limit the number and/or type of access points

Limiting driveways or access points should be considered at locations with poor sight distance, high crash rates, high volume-to-capacity ratios, or poor access to facilities.

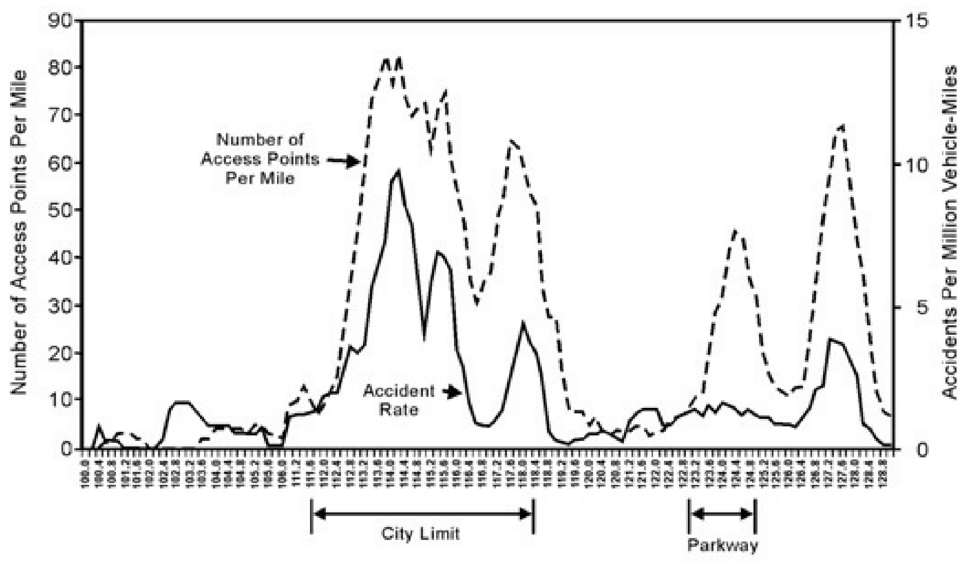
Sight distance can be improved by clearing signs and foliage or directing traffic to one access point where drivers have adequate sight distance. Improving the sight distance serves both the general traffic flow and vehicles entering the general traffic flow. The access point may funnel entering traffic to one location, so that drivers passing by the access are aware of entering traffic.

Combining driveways is another way to reduce accesses and conflict points. Closely spaced driveways can cause traffic to slow and increase the crash potential. Accommodating entering traffic at one location simplifies driver tasks. Combining multiple driveways into one joint-use driveway directs traffic more safely, clearly, and efficiently.

Exhibit 5-2 shows the relationship between access density and accident rates in Lincoln City and Lincoln Beach. Crashes increase as access points increase. The area labeled City Limit in

Exhibit 5-2 is Lincoln City on US101, which has a high density of access points. The area labeled Parkway is in Lincoln Beach where a non-traversable landscaped median limits access to driveways and side streets. Crash rates in the Parkway section are greatly reduced.

Exhibit 5-2 Access Points Per Mile vs. Crashes Per Mile



Comparison of Access Connections Per Mile with Crashes Per Mile on a Segment of US 101 in Oregon

From Analysis of Traffic Accidents Within the Functional Area of Intersections and Driveways Technical Report Trans 1-95 1995

Locating driveways on lower classification roadways or backage/frontage roads also reduces the number of conflict points along the roadway. Removing driveways from an arterial or a collector decreases delay caused by turning vehicles. Diverting traffic to local roads directs traffic to one access point and simplifies conditions.

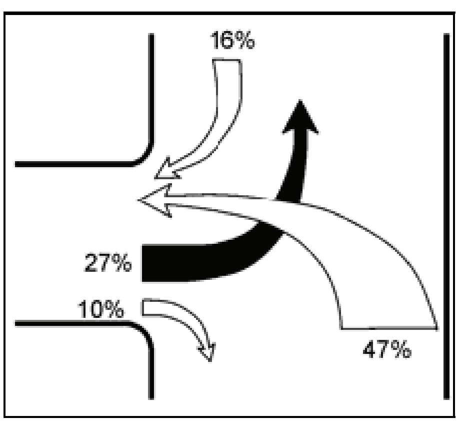
Refer to the Oregon Administrative Rule (OAR) 734, Division 51 for information concerning signal spacing, backage/frontage roads, or access rights. Sometimes access rights to individual parcels are obtained. Rules regarding obtaining property access rights to state highways, signal spacing, and backage/frontage roads are found in OAR 734, Division 51 and on the Access Management Unit's (AMU) website at www.oregon.gov/ODOT/HWY/ACCESSMGT/.

Non-traversable median or traffic control devices to restrict turning movements
Medians are a highway element sometimes provided to separate traffic traveling in opposite directions. A median can be traversable or non-traversable. A traversable median may be a painted or concrete mountable median which allows traffic over it. A non-traversable median is a physical barrier (examples include the Jersey barrier, landscaped, or grassy median) that separates opposing traffic and prohibits movement across the median. Installing a non-traversable median restricts turning and crossing movements at roadway accesses. Non-traversable medians reduce conflict points by eliminating turn movements that impact the general traffic flow. More information about median types can be found in Chapter 5.5 of the Highway Design Manual (HDM).

A median that impacts the State Highway Freight System must comply with the ORS 366.215 which states that the Oregon Transportation Commission (OTC) may not permanently reduce the vehicle-carrying capacity of an identified freight route when altering, relocating, changing, or realigning a state highway unless safety or access considerations require the reduction.

According to the Transportation Research Board, Access Management Manual 2003, 47% of crashes involve left turn ingress movements and 27% of crashes involve left turn egress movements, shown in Exhibit 5-3. Controlling turn movements can sometimes allow one turn direction and divert others elsewhere.

Exhibit 5-3 Percent of Driveway Crashes by Movement



Non-traversable medians reduce overall crash rates, improve pedestrian safety, and enhance visuals. They restrict traffic from making complex left turns and provide median openings at designated locations. They may provide a pedestrian refuge between directions of traffic and decrease pedestrian clearance intervals. A Georgia study presented at the Fourth National Conference on Access Management in Portland reports that a highway with a non-traversable median has 78% less pedestrian fatalities per 100 miles of road than a highway with a two-way left-turn lane (http://www.accessmanagement.info/pdf/AM00PAPR.pdf). Landscaping large medians improves the aesthetics of the roadway. Although medians improve the roadway, they are costly and may require the acquisition of right-of-way.

Grade-Separated Roadways

Grade-separated roadways are where one roadway crosses over the other. Grade separation should be considered for safety or where an at-grade signal cannot accommodate traffic capacity. Conflict points are decreased by removing major flow grade crossings and by re-routing turning traffic. Interchanges are grade-separated connections of two or more roads. Interchanges reduce conflict points and the severity of crashes, increasing safety. Although interchanges occupy a large amount of space and require costly structural work, they reduce delay and improve the safety and efficiency of a corridor.

It can be difficult to accommodate pedestrians at interchanges because of the separation of paths. For more information about accommodating pedestrians or bicyclists, refer to the Oregon Bicycle and Pedestrian Plan:
http://www.oregon.gov/ODOT/HWY/BIKEPED/planproc.shtml.

Weaving Sections

Weaving sections, where vehicles traveling in the same direction cross paths, create several conflict points. Exhibit 5-4 and Exhibit 5-5 show two examples of weaving vehicle paths entering and exiting the roadway. A ramp weave and a major weave (Type A weaves) have five minor conflict points. The combination of a major weave with lane balance at the exit gore (Type C weave) has seven minor conflict points. More information about weaving sections can be found in Chapter 6.

Exhibit 5-4 Conflict Points for a Type A Weave

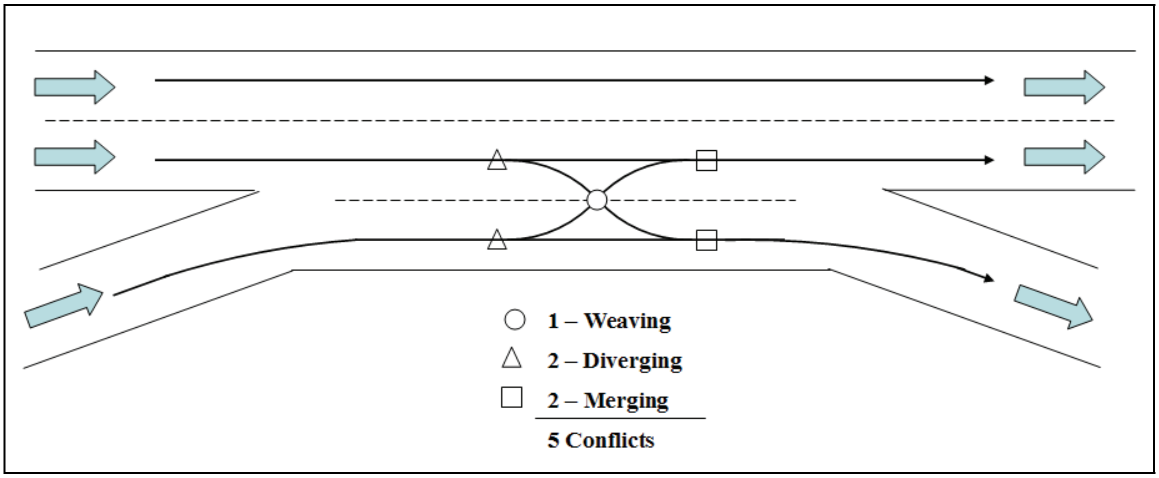
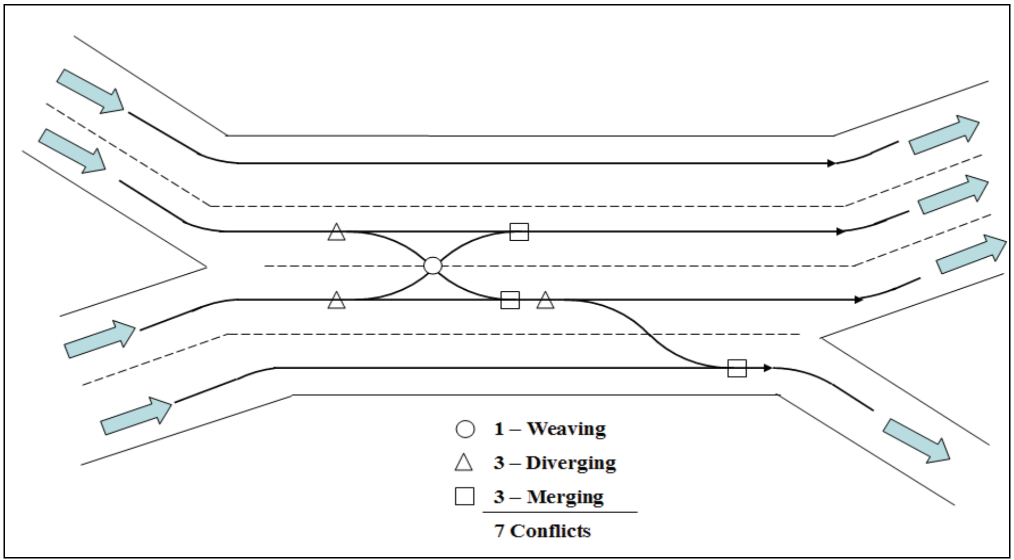


Exhibit 5-5 Conflict Points for a Type C (Three Lane Weave)



Unchannelized Intersections

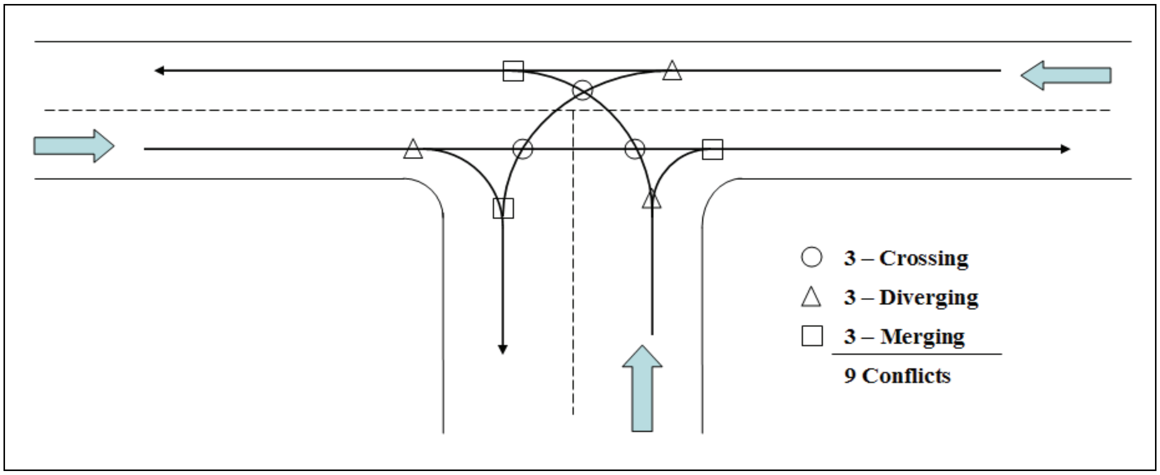
Unchannelized intersections are located where two roadways with traversable medians and no turn restrictions intersect. Unchannelized intersections allow full access to traffic. Every intersection should be analyzed as a unique situation. Unchannelized intersections may have other forms of traffic control such as a traffic signal. A traffic signal does not reduce the number of conflict points, but increases driver communication and awareness. For movements that operate with separate signal phases the exposure to conflicts is significantly reduced when compared to movements that operate with unsignalized control or permissive signal phasing.

NOTE: The intersections analyzed and illustrated in the figures typically show the number of conflict points for one lane of travel per movement. Any additional lanes must be included in the analysis. The conflict points range from 5 at the intersection of two one-way roads to 32 at a four-way intersection

T-Intersection

A T-intersection is where one roadway ends at its intersection with another roadway. The T-intersection shown in Exhibit 5-6 permits turns in all directions. The intersection in the figure has nine conflict points, three of which are major.

Exhibit 5-6 Conflict Points for the T-Intersection



Four-Leg Intersection of Two One-Way Roads

One-way roads limit the types of turns needed, reducing conflict points. Couplets are typically grids of one way streets. Couplets often have multiple lanes in each direction and move traffic more efficiently through a corridor. The four-leg intersection of one-way roads is shown in Exhibit 5-7 and an intersection with two lanes in each direction is shown in Exhibit 5-8.

Exhibit 5-7 Conflict Points for a Four-Leg Intersection of Two One-Way Roads

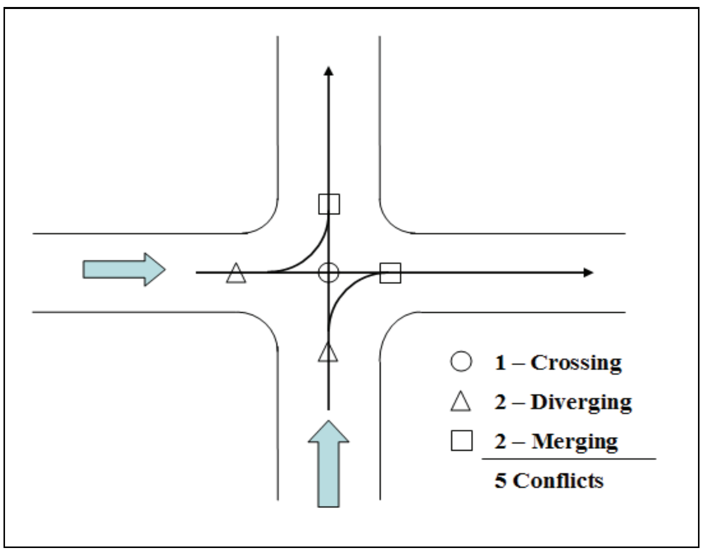
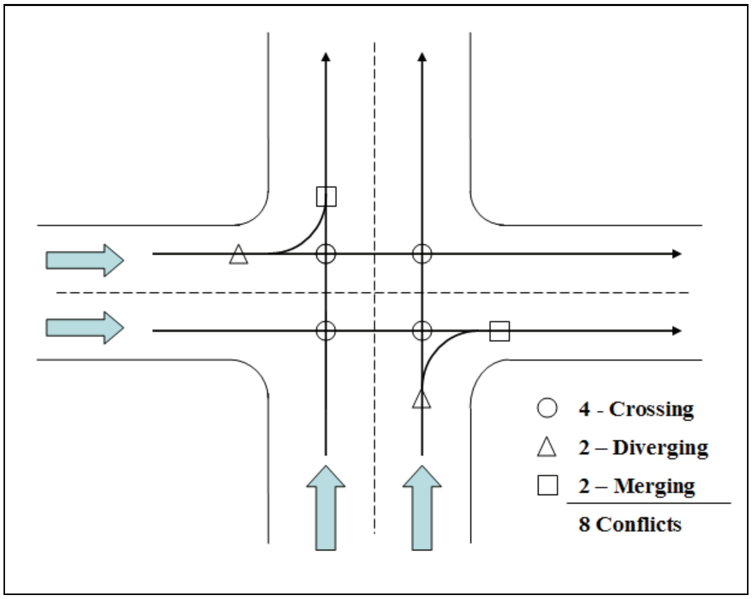


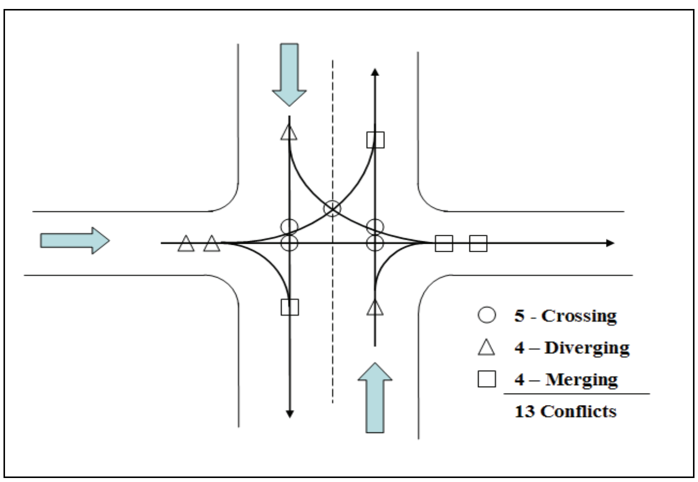
Exhibit 5-8 Conflict Points by Lane for a Four-Leg Intersection of Two One-Way Roads



Four-Leg Intersection of a Two-Way Road and a One-Way Road

A four-leg intersection of a two-way road and a one-way road is shown in Exhibit 5-9. The one-way road may be part of a couplet. The one-way road limits turns and reduces conflict points.

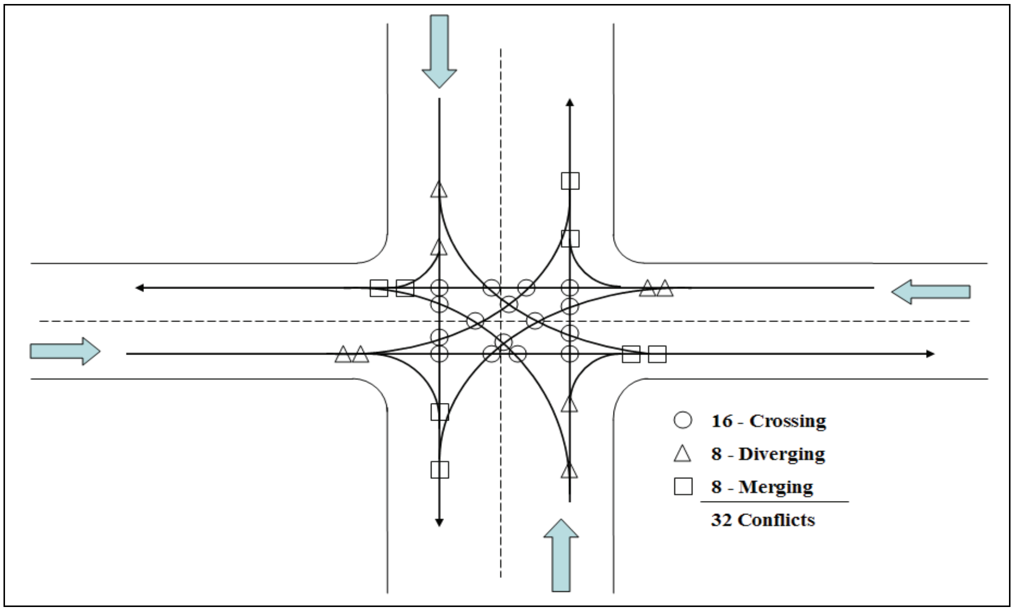
Exhibit 5-9 Conflict Points for a Four-Leg Intersection of a Two-Way Road and a One-Way Road



Four-Leg Intersection

The four-leg intersection with no medians, shown in Exhibit 5-10, has the most conflict points of all intersections. The four-leg intersection allows movement in all directions and is the most familiar to drivers.

Exhibit 5-10 Conflict Points for a Four-Leg Intersection



Channelized Intersections

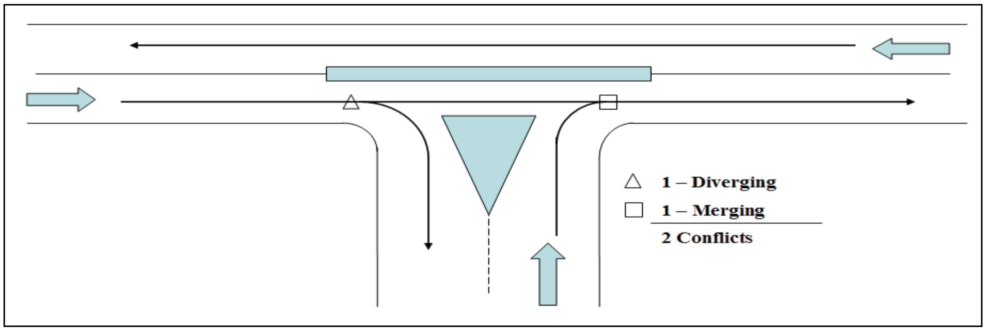
A channelized intersection has restricted turn movements by signs, pavement markings, medians or some other type of traffic control. Channelized intersections include, but are not exclusive to right-in/right-out intersection; non-traversable median separated four leg intersection, left turn ingress intersection, left turn egress intersection, and roundabout. Both ends of the non-traversable median should be analyzed for the required traffic control to meet traffic needs safely.

Intersections and driveways that are restricted to right-in/right-out have two conflict points, but complex channelized intersections may have up to eleven conflict points.

Right-In/Right-Out Intersection

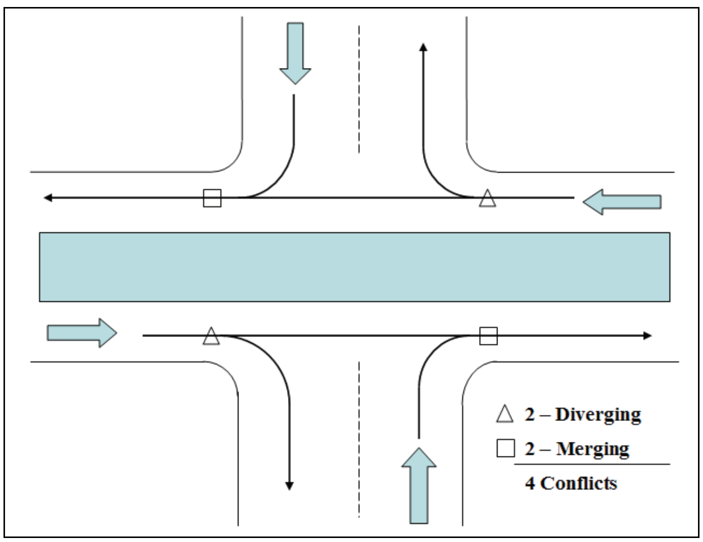
The right-in/right-out geometry shown in Exhibit 5-11 restricts traffic to right turn movements only and forces roadway users to complete a left turn at another location. This improves safety at the location of the intersection, but requires left turning vehicles to travel further to get to their destination.

Exhibit 5-11 Conflict Points for the Right-In/Right-Out Intersection



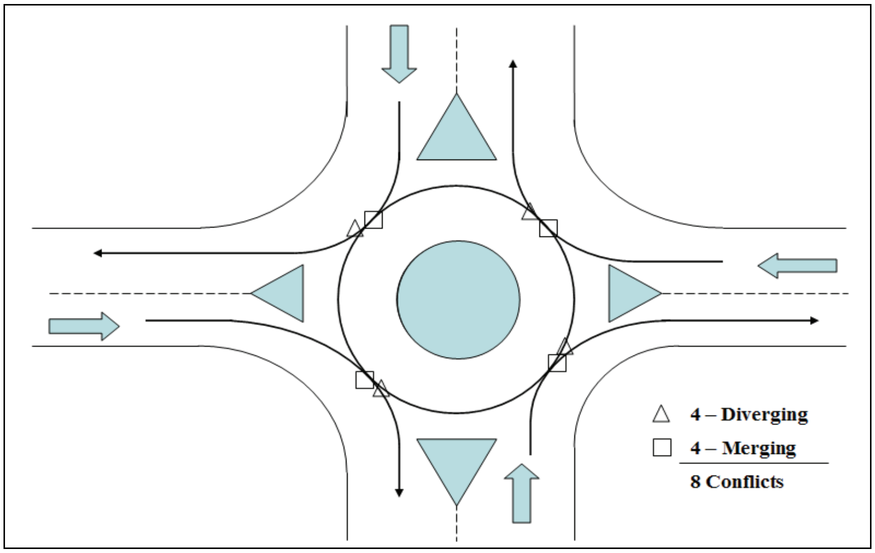
Non-Traversable Median Separated Four-Leg Intersection
Installing a non-traversable median changes the traffic flows of the typical four-leg intersection by effectively creating two right-in/right-out intersections as shown in Exhibit 5-12. Left turning or crossing vehicles must complete that maneuver at another location, eliminating the major conflict points. This reduces the conflict points from the typical 32 to just four. The intersection is restricted to right turns, improving safety and operations, but does add out of direction travel.

Exhibit 5-12 Conflict Points for a Median Separated Four-Leg Intersection



Roundabout
Roundabouts are considered at locations where speeds and volumes may not require a traffic signal for smooth operations. The roundabout, shown in Exhibit 5-13, directs all traffic to move in a counter-clockwise direction allowing movements in all directions. This limits conflict points to merging and diverging movements. Bypass lanes would produce additional conflict points at their respective merge and diverge locations. Chapter 7 contains capacity analysis procedures for roundabouts and bypass lanes.

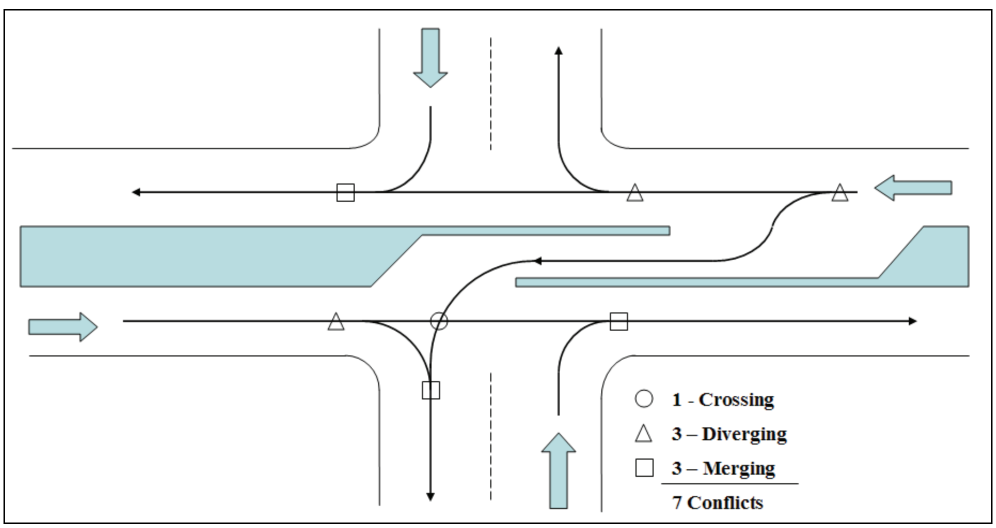
Exhibit 5-13 Conflict Points for the Single Lane Roundabout



Left Turn Ingress Intersection
The left turn ingress intersection shown in

Exhibit 5-14 permits one direction of traffic to turn left, from a turn bay, while the opposing left and crossing movements are prohibited. The permitted left turn may have significantly higher volumes or may service a critical access.

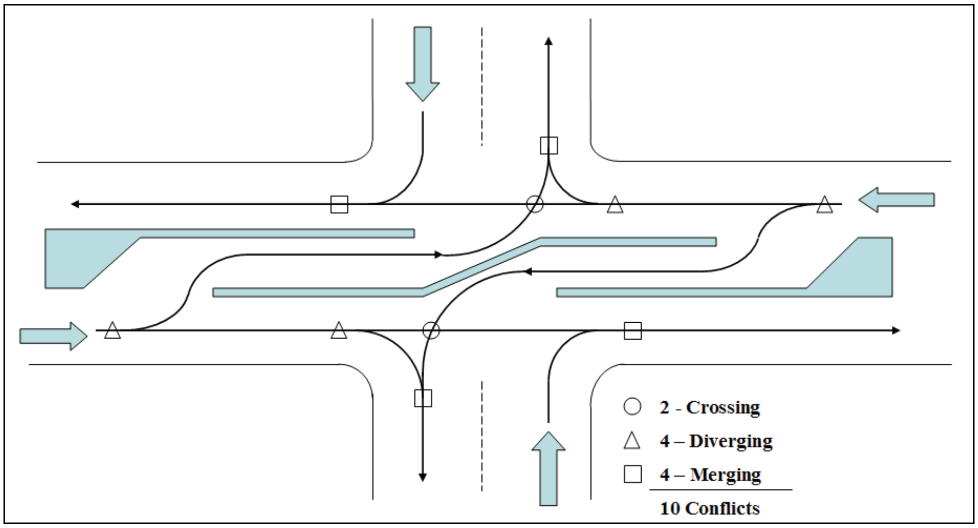
Exhibit 5-14 Conflict Points for a Median with One Left Turn Ingress Intersection



Two Left Turn Ingresses Intersection
Exhibit 5-15 shows the geometry for two left turn ingresses. Only the traffic turning left into the access street is permitted through the median. Vehicles can remain in the median until there is a sufficient gap to complete the turn. More left turn egress and left turn ingress examples are

shown in Exhibit 5-16 through Exhibit 5-18. Locations that provide a median restricting egress or ingress turns require median openings for vehicles to make U-turns. The median openings add a merge and a diverge point to the segment or intersection. Any intersection along a median that permits U-turns must be analyzed for conflict points, with the inclusion of the merge and diverge conflict points caused by the U-turn.

Exhibit 5-15 Conflict Points for a Median with Two Left Turn Ingresses Intersection



Other Ingress and Egress Examples

Exhibit 5-16 Conflict Points for a Median with a Left Turn Ingress and Egress Intersection

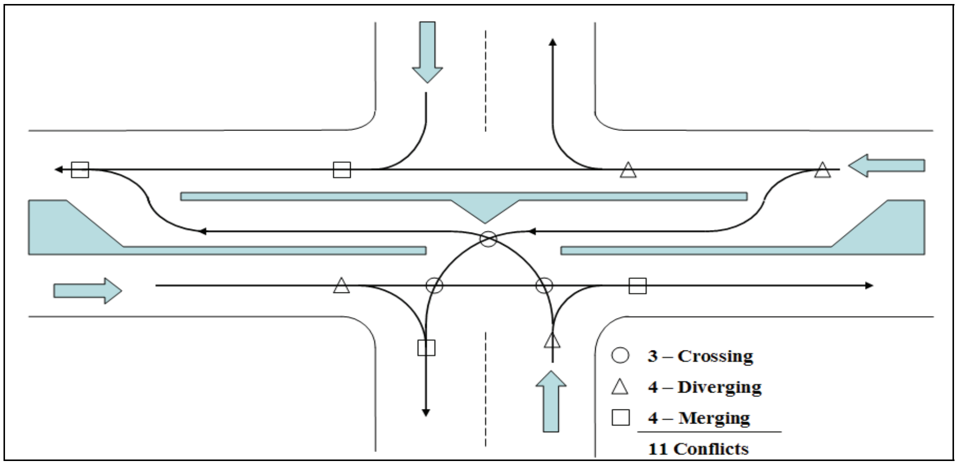


Exhibit 5-17 Conflict Points for a Median with One Left Turn Egress Intersection

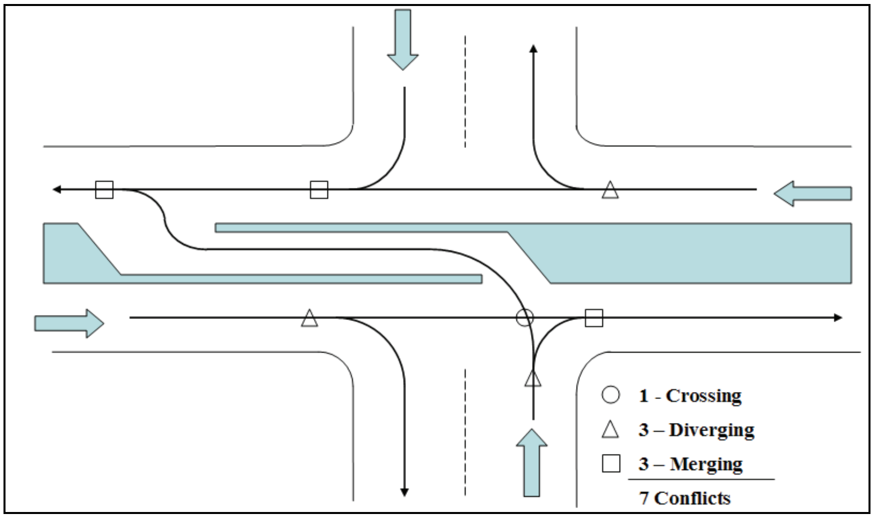
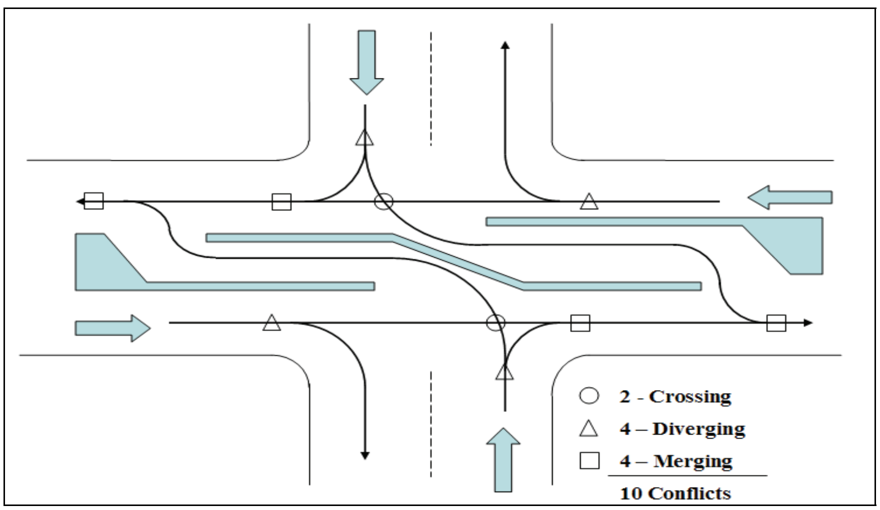


Exhibit 5-18 Conflict Points for a Median with Two Left Turn Egresses Intersection



Indirect Left Turns

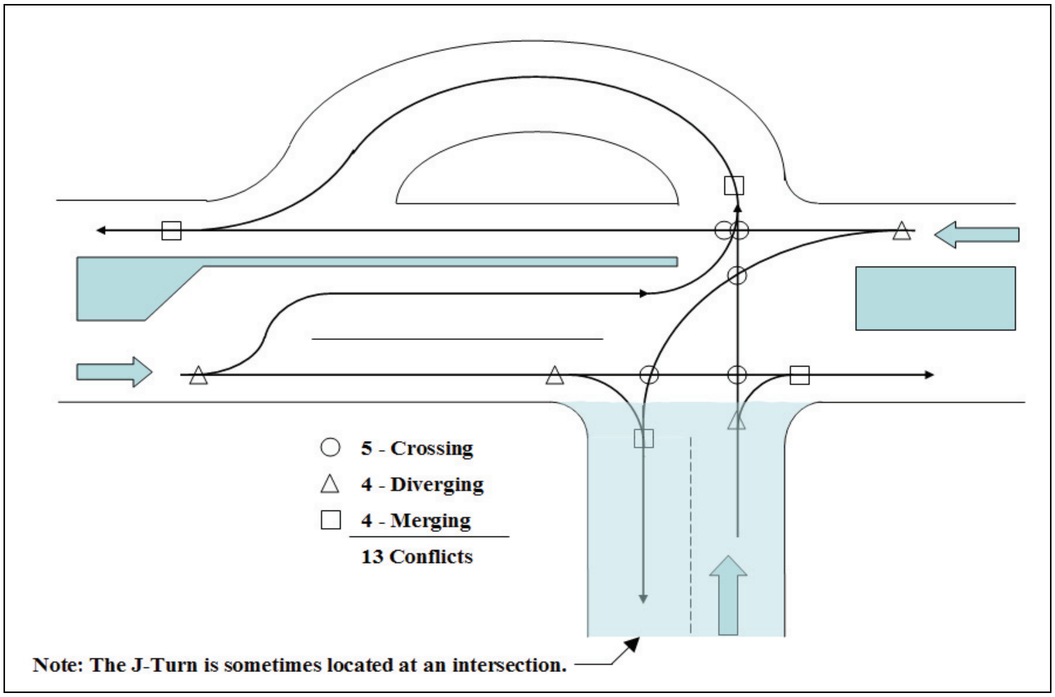
Some intersections require creative accommodations for left turning vehicles. The treatment of left turns must be considered at intersections with restricted turns, high volumes or high speeds to achieve the greatest capacity and safety for traffic.

J-Turn

The J-turn provides an opportunity for vehicles to turn left on either side of a non-traversable median. The median restricts intersections and driveways to right-in/right-out which reduces the conflict points. Vehicles can turn left at median openings which may be accompanied by a stand alone J-turn, a J-turn intersection, or a signalized J-turn intersection. The J-turn intersection shown in Exhibit 5-19 has thirteen conflict points. Locations where the J-Turn does not meet a crossing roadway or driveway have three conflict points: one diverge, one crossing, and one merge conflict point.

The J-turn intersection also provides turning vehicles an opportunity to join traffic moving in its desired direction. An add-lane at the J-turn will eliminate the merge conflict point, shown in Exhibit 5-19, and increase the capacity of the segment. The J-turn reduces the congestion and improves the safety of median openings. A J-turn intersection may or may not have pedestrian crossings.

Exhibit 5-19 Conflict Points for a J-Turn Intersection

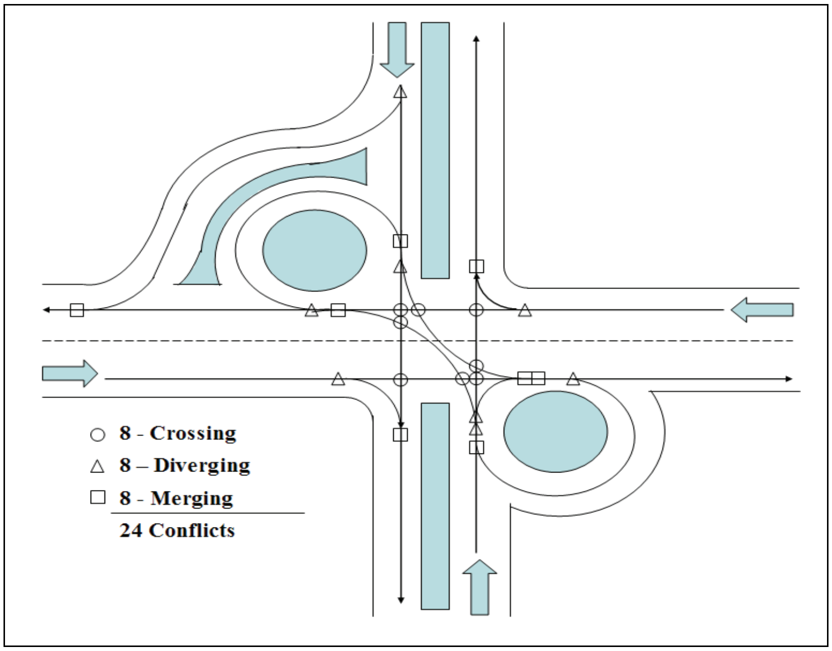


Jughandle Intersection

This jughandle intersection, shown in Exhibit 5-20, is commonly used when there are high left turn volumes which can not be accommodated by a signalized intersection. The left turning traffic passes through the intersection as a through movement both before and after diverging onto a loop ramp to complete the left turn. This reduces the congestion by removing phases from the traffic signal and improves the safety of the intersection by reducing the number of conflict points. The jughandle can be located in various quadrants of the intersection depending on the restricted turn movements.

Note: Jughandle configurations are not always installed as pairs and are not always accompanied with right turn bypass lanes.

Exhibit 5-20 Conflict Points for a Jughandle Intersection



Pedestrian Conflict Points

Pedestrian conflict points are counted separately from vehicle-vehicle conflict points. Pedestrian conflict points are located at the intersection crosswalks. Turning vehicles and crossing vehicles are counted separately. Sometimes pedestrian movements need to be furnished away from the intersection. Locations where no crosswalk is located, such as the main line of an intersection with two left turn ingresses, may be serviced by an overpass or pedestrians may cross further down the road at a designated pedestrian crossing or signal. Intersections with wide cross sections, such as a median separated four-way intersection, are more attractive to pedestrians when it has a non-traversable landscaped median. The median serves as a pedestrian refuge and can decrease the pedestrian clearance intervals. Exhibit 5-21 through Exhibit 5-23 show various examples of pedestrian conflict points from two one-way roads to a four-way intersection with restricted left turn ingresses.

Exhibit 5-21 Pedestrian Conflict Points for a Four-Leg Intersection

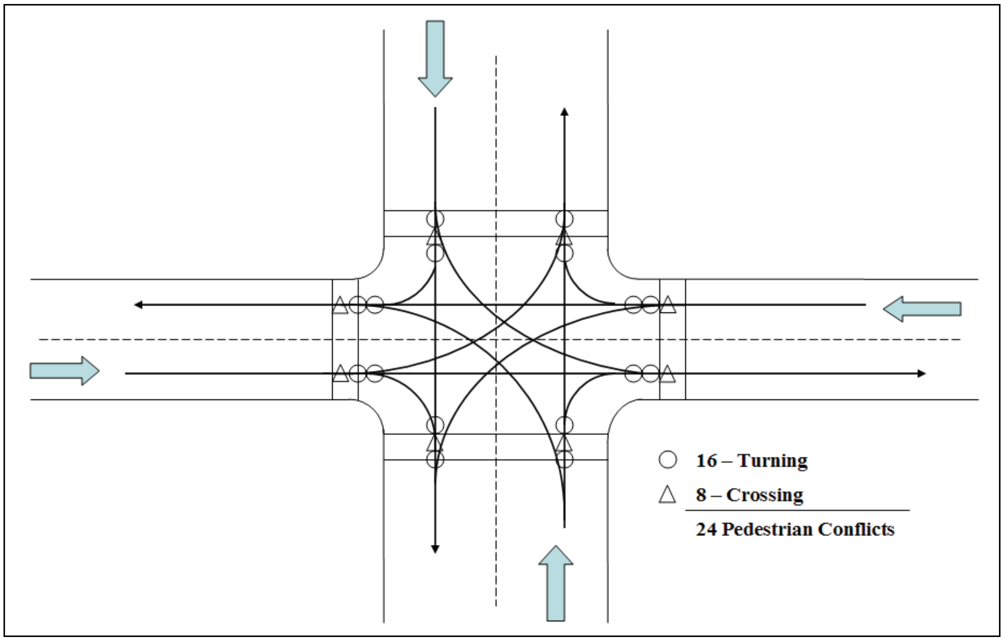


Exhibit 5-22 Pedestrian Conflict Points for a Median Separated Four-Leg Intersection

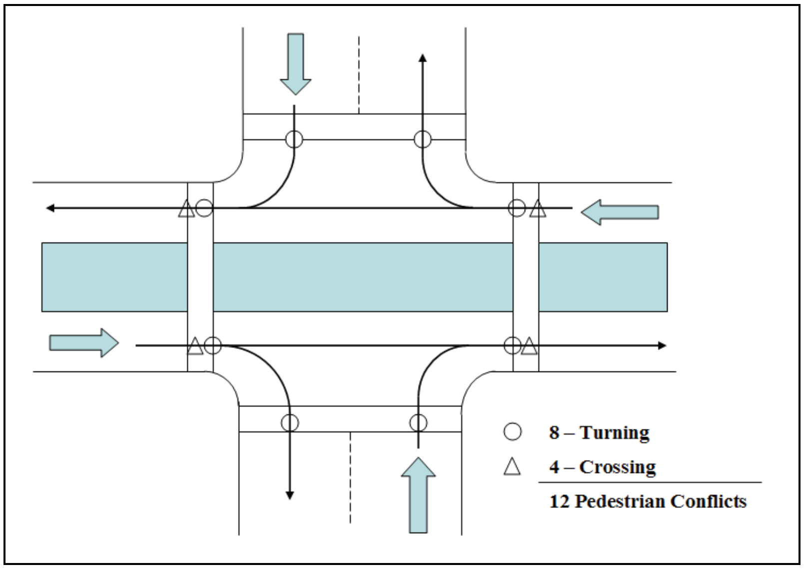
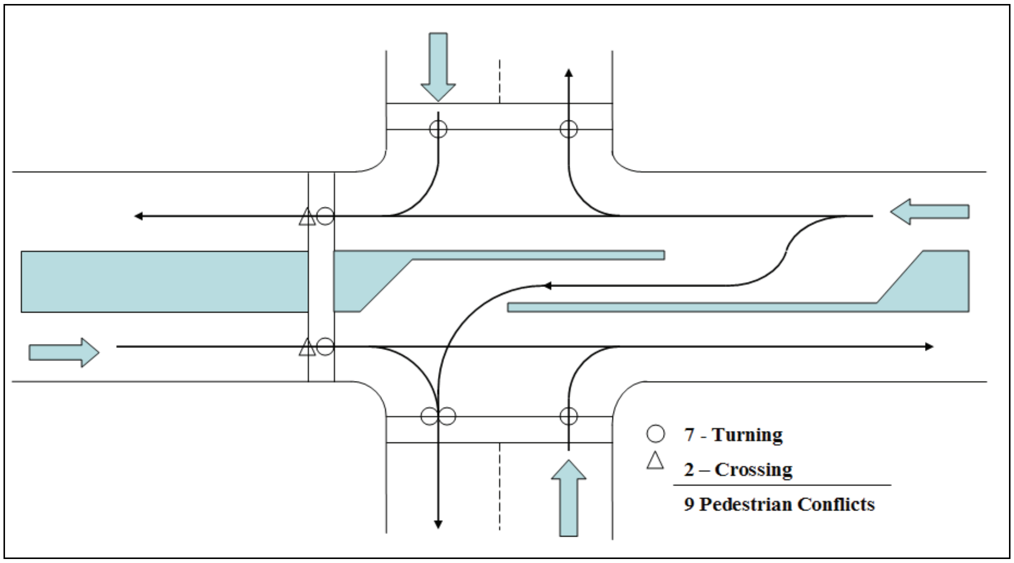


Exhibit 5-23 Pedestrian Conflict Points for a Median with One Left Turn Ingress Intersection

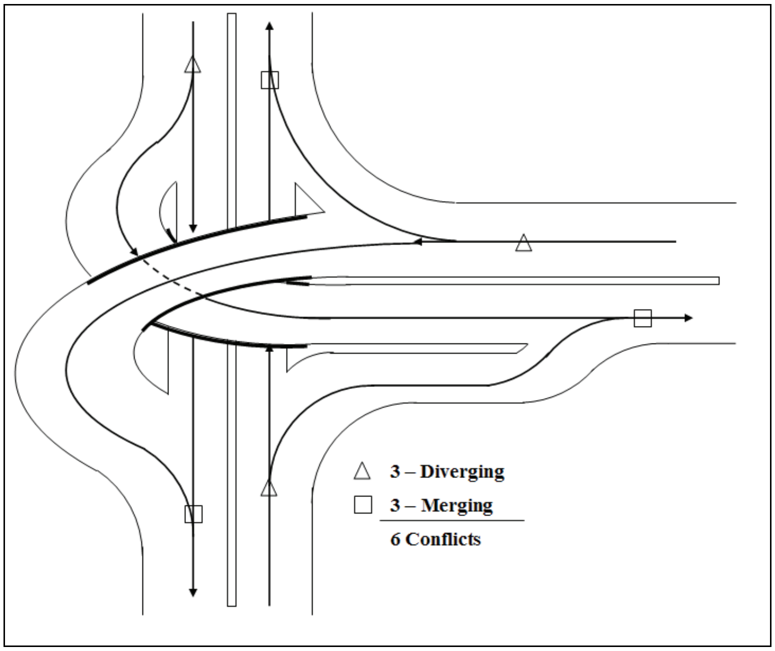


Grade-Separated Access
High speed and high volume junctions may need grade-separated access to meet traffic capacity and safety. Interchanges remove major flow grade crossings increasing capacity and reducing conflict points. Interchanges may have as few as six conflict points (directional interchange) or as many as twenty-eight conflict points (left turn flyover).

Directional Interchange
The directional interchange has all free flow ramps with only six (three diverging and three merging) minor conflict points in the system as seen in Exhibit 5-24. It is similar to a T-intersection with large volumes and high speeds. Traffic does not cross paths due to the grade separation. Each free flow connection has a merge and diverge conflict point. Due to the high volumes and speed, pedestrian crossing does not occur at the street level.

The full directional interchange, shown in Exhibit 5-24 has three levels of grade separation. A partial directional interchange is a junction where one leg has lower speeds and is accommodated by a loop ramp. A partial directional interchange has only two levels of grade separation. Since each free flow ramp has one merge and one diverge conflict point, the partial directional interchange has the same conflict points as the full directional interchange. Likewise, a four way directional interchange would have eight conflict points, four merge and four diverge.

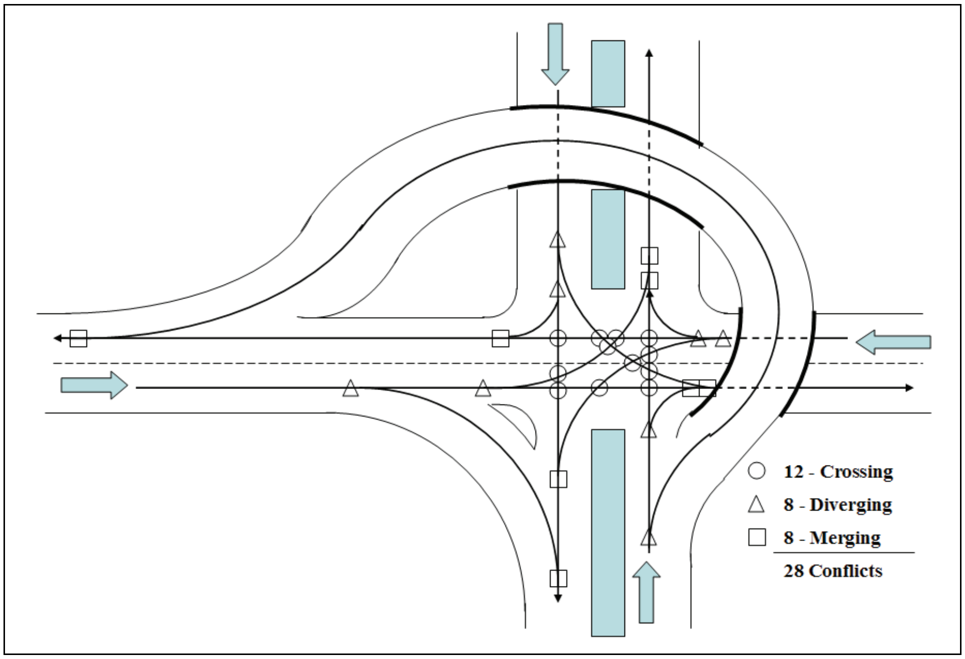
Exhibit 5-24 Conflict Points for a Directional Interchange



Left-Turn Flyover Intersection
The left-turn flyover intersection, shown in

Exhibit 5-25, is commonly used when one direction has high left turn volumes which cannot be accommodated by a signalized intersection. The left turning traffic is grade-separated as it crosses over the opposing traffic, reducing conflict points and congestion. Pedestrian crossings may or may not be modified from the standard intersection.

Exhibit 5-25 Conflict Points for a Left-Turn Flyover Intersection

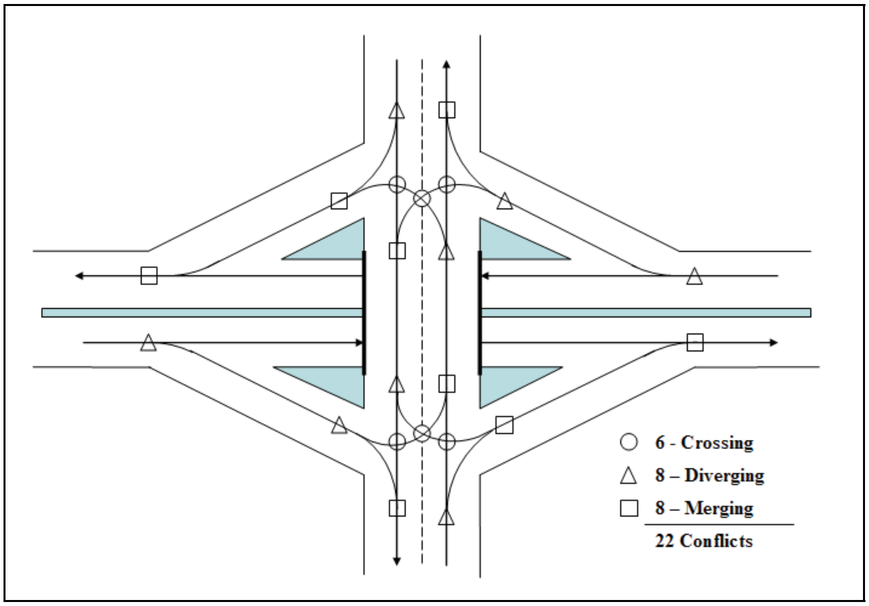


Diamond Interchange

The diamond interchange has four ramps and may have traffic control at the minor road. Traffic is directed to turn on or off each ramp creating conflict points. There is potential for weaving conflicts if this interchange has two or more lanes in each direction. Conventional, compressed, and tight diamond interchanges all travel the same paths and the conflict points are the same. Exhibit 5-26 shows the conflict point configuration for a conventional diamond interchange.

Pedestrian crossings are generally not provided at locations along the major, free-flowing movement.

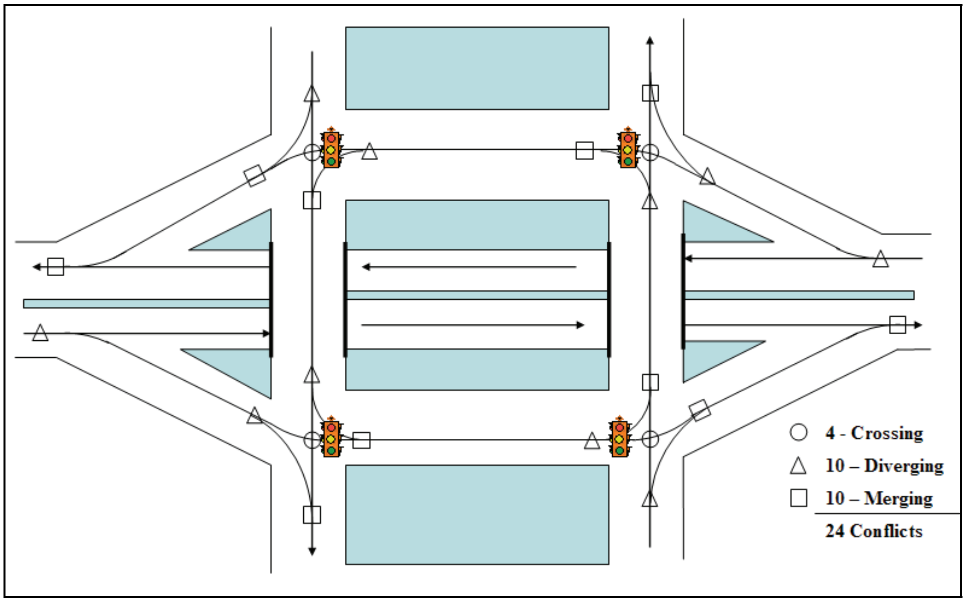
Exhibit 5-26 Conflict Points for a Diamond Interchange



Split Diamond Interchange

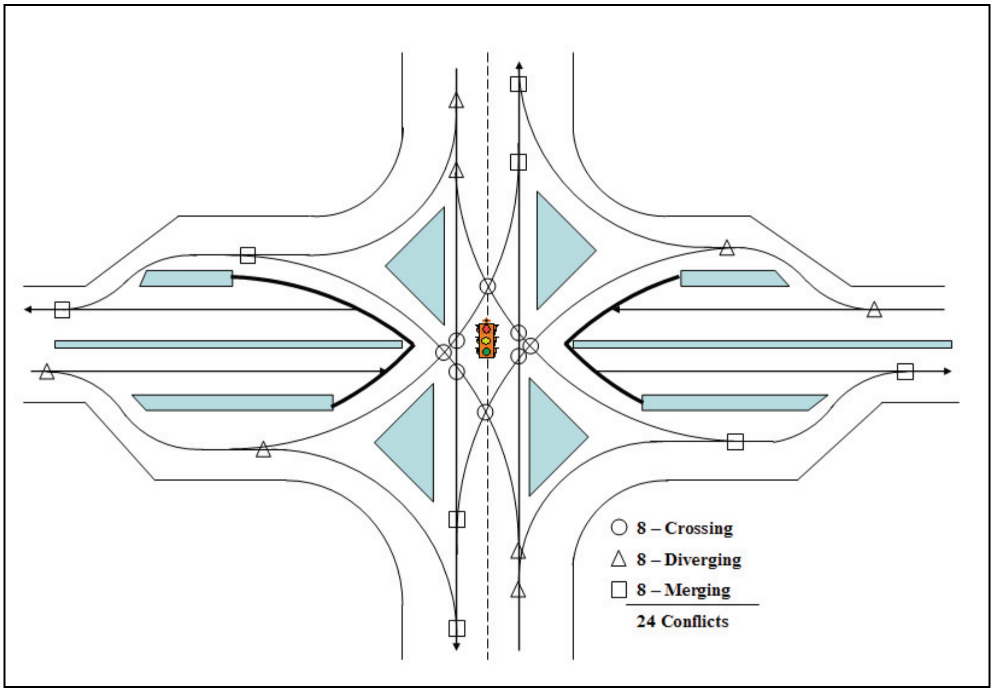
A split diamond interchange, shown in Exhibit 5-27, has only four ramps which connect to each other with segments that travel parallel to the major roadway. This type of interchange is appropriate where minor roads are one-way streets and will most likely be accompanied with traffic signals at the ramp terminals. It may be furnished on a regular grid system where the minor streets are two-way as well.

Exhibit 5-27 Conflict Points for a Split Diamond Interchange



Single Point Urban Interchange (SPUI)
The single point urban interchange has four ramps that converge to one point which is controlled by a traffic signal. All minor movement vehicles must travel through the same grade-separated intersection. This type of interchange conserves space and provides large capacity, since the signal operates with fewer phases. These conflict points are shown in Exhibit 5-28.

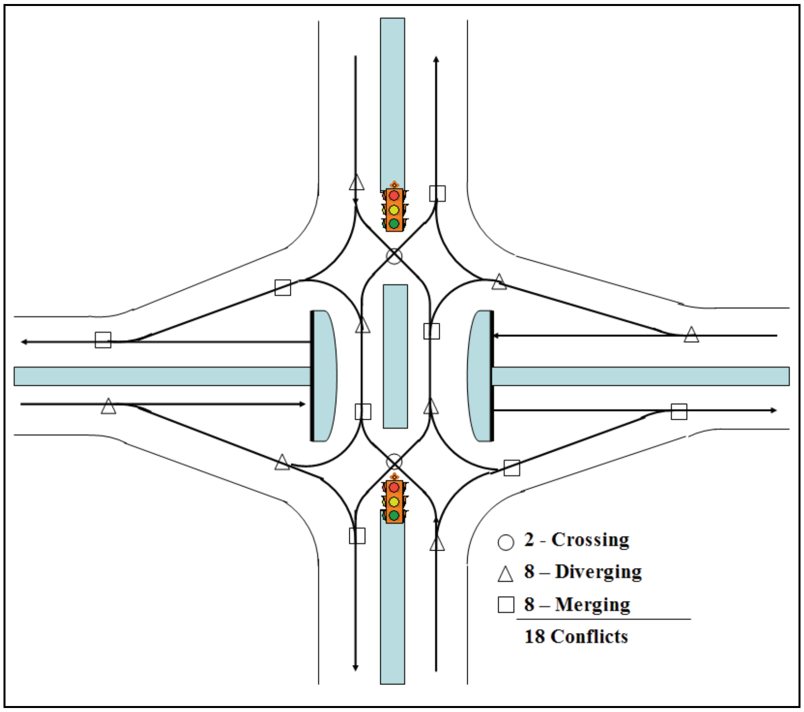
Exhibit 5-28 Conflict Points for a Single Point Urban Interchange



Divergent Diamond Interchange
The divergent diamond interchange has four ramps where vehicles that want to turn right may get on or off the roadway as shown in Exhibit 5-29. The divergent diamond is designed so that as traffic on the minor roadway approaches the interchange intersections, the opposing lanes change which side of the road is being used. This allows turn conflicts to be merge/diverge rather than crossing. Vehicles that want to turn left follow the appropriate traffic flow and merge into the receiving lane without interference of opposing traffic. The divergent diamond overlap allows vehicles to turn left at the designated signalized intersections reducing crossing paths and conflict points. This configuration also allows the use of fewer phases in the traffic signal operation.

Due to high speeds and high volumes, the divergent diamond must consider appropriate pedestrian crossings. There have been designs that show pedestrian crossings over the ramps and down the middle of the minor leg.

Exhibit 5-29 Conflict Points for a Divergent Diamond Interchange

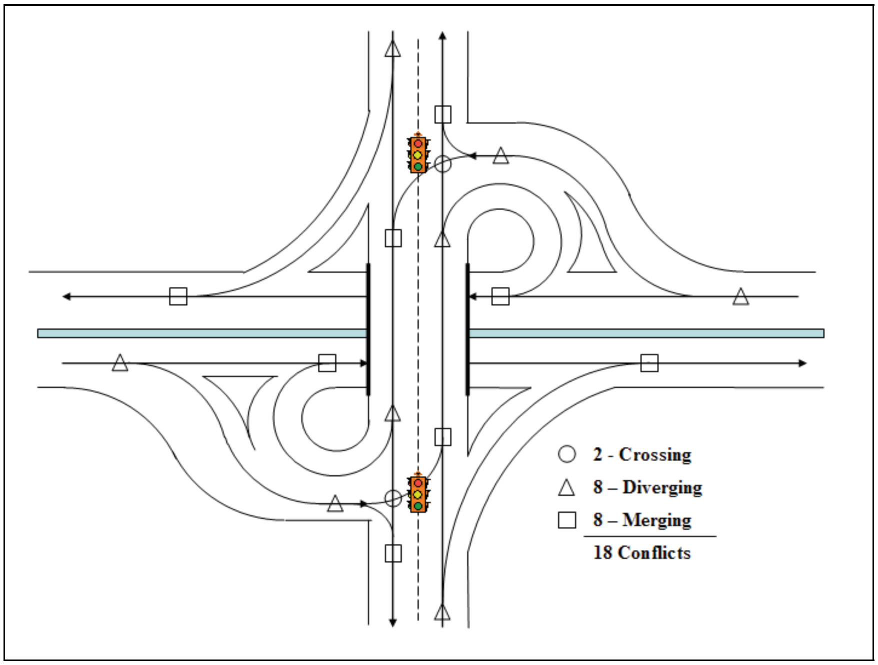


Partial Cloverleaf Interchange

The partial cloverleaf, shown in Exhibit 5-30, reduces conflict points by providing elevated ramps to maneuver on and off the roadway. The on-ramps are free flow but the off ramps are controlled by traffic signs or signals. Vehicle paths cross only for left turning traffic from the ramps. There is potential for weaving conflicts if this interchange has two or more lanes in each direction.

Although a full cloverleaf configuration is a possible design, Oregon and many other states no longer use them because of the short distance between the merging/diverging paths of the ramps. If pedestrian crossings were present, they would direct people over the ramps where the speeds are lowest and drivers have adequate sight distance.

Exhibit 5-30 Conflict Points for a Partial Cloverleaf Interchange



Conflict Point Summary Table

Exhibit 5-31 is a compilation of geometric alternatives and types of conflict points. The conflict points in the table account for one vehicle path for every movement and should only be used for general guidance. Every segment, intersection or interchange is unique and should not be assumed to have the generic characteristics of this table.

Exhibit 5-31 shows the number of crossing, diverging, merging and weaving conflict points. It sums the minor and major conflict points and shows the number of potential pedestrian conflict points. Turning movements and through movements were counted for the pedestrian conflicts at each potential location for a crosswalk. Total vehicle-vehicle conflict points range from two for the right-in-right-out intersection to thirty-two for the four-leg intersection. The number of conflict points reflects the number of approaches, permitted movements, and geometric alternatives. The conflict points do not reflect the number of lanes as the values provided in Exhibit 5-31 have only one lane per movement.

Exhibit 5-31 Conflict Points with One Vehicle Path per Movement

Geometry (Figure Number)	Crossing	Diverging	Merging	Weaving	Minor	Major	Total Conflicts	Pedestrian Conflicts
Weaving Sections								
Ramp Weave (Exhibit 5-4)	0	2	2	1	5	0	5	N/A
Three Lane Weave (Exhibit 5-3)	0	3	3	1	7	0	7	N/A
Unchannelized Intersections								
T-Intersection (Exhibit 5-6)	3	3	3	0	6	3	9	12
Four Leg Intersection of Two One-Way Roads (Exhibit 5-7)	1	2	2	0	4	1	5	8
Four Leg Int. of a Two-Way Road and a One-Way Road (Exhibit 5-9)	5	4	4	0	8	5	13	14
Four Leg Intersection (Exhibit 5-10)	16	8	8	0	16	16	32	24
Channelized Intersections								
Right-In-Right-Out (Exhibit 5-11)	0	1	1	0	2	0	2	8
J-Turn (Exhibit 5-19)	1	1	1	0	2	1	3	2
Four-Way Intersection (Exhibit 5-12)	0	2	2	0	4	0	4	12
One Left Turn Ingress (Exhibit 5-14)	1	3	3	0	6	1	7	9
One Left Turn Egress (Exhibit 5-17)	1	3	3	0	6	1	7	9
Single Lane Roundabout (Exhibit 5-13)	0	4	4	0	8	0	8	8
Two Left Turn Ingresses (Exhibit 5-15)	2	4	4	0	8	2	10	6
Two Left Turn Egresses (Exhibit 5-18)	2	4	4	0	8	2	10	6
Left Turn Egress and Ingress (Exhibit 5-16)	3	4	4	0	8	3	11	6
Indirect Left Turns								
J-Turn with Intersection (Exhibit 5-19)	5	4	4	0	8	5	13	7
Jughandle (Exhibit 5-20)	8	8	8	0	16	8	24	12
Grade-Separated Access								
Directional Interchange (Exhibit 5-24)	0	3	3	0	6	0	6	8
Divergent Diamond (Exhibit 5-29)	2	8	8	0	16	2	18	16
Partial Cloverleaf (Exhibit 5-30)	2	8	8	0	16	2	18	12
Diamond (Exhibit 5-26)	6	8	8	0	16	6	22	16
Split Diamond (Exhibit 5-27)	4	10	10	0	20	4	24	16
Single-Point Urban Interchange (Exhibit 5-28)	8	8	8	0	16	8	24	16

Left-Turn Flyover (Exhibit 5-25)	12	8	8	0	16	12	28	21

5.6 Sight Distance

The length of roadway ahead that is visible to a driver is often referred to as “sight distance." The amount of visible roadway needed by a driver at any given time depends on the maneuvers or decisions that must be made at that moment. The four basic categories of sight distance are:

1. Intersection Sight Distance (Desirable)
2. Stopping Sight Distance (Minimum)
3. Decision Sight Distance
4. Passing Sight Distance

While each of these is briefly described below, intersection and stopping sight distance are most frequently examined in traffic analysis. For additional information on sight distance refer to ODOT’s Highway Design Manual.

- **Intersection sight distance (desirable)** is considered adequate when drivers at or approaching an intersection have an unobstructed view of the entire intersection and of sufficient lengths of the intersecting highways to permit the drivers to anticipate and avoid potential collisions. Sight distance must be unobstructed along both approaches at an intersection and across the corners to allow the vehicles simultaneously approaching to see each other and react in time to prevent a collision. Intersection sight distance should be obtained at every road approach, whether it be a signalized intersection or private driveway. In no case should the sight distance be lower than safe stopping sight distance (minimum).
- **Stopping sight distance (minimum**) is the minimum distance required for a vehicle traveling at a particular design speed to come to a complete stop after an obstacle on the road becomes visible. Stopping sight distance is normally sufficient to allow an alert and prudent driver to come to a hurried stop under normal circumstances.
- **Decision sight distance** should be provided at locations where multiple information processing, decision making and corrective actions are needed. Sample locations where decision sight distance is needed include unusual intersection or interchange configuration and lane drops.
- **Passing sight distance** is the minimum distance required for a vehicle to safely and comfortably pass another vehicle. If adequate passing sight distance opportunities cannot be accommodated in the project design, passing lanes or climbing lanes should be considered. See Chapter 6.

5.7 Multi-Modal Analysis

This section left intentionally blank while under development.

5.8 Other Analysis Issues/Procedures

While working on various types of projects, a number of situations may arise requiring analysis methodologies not discussed in this manual. Under these circumstances the ODOT Region Traffic staff or the Transportation Planning Analysis Unit should be contacted for a recommendation. Note: Region Traffic and TRS are responsible for work zone and pavement design analysis. If ODOT does not have a preferred analysis methodology to offer, there are a number of technical resources available for consultation. Non-standard analysis submissions shall include thorough documentation of assumptions, methods and calculations and engineer's stamp. A listing of several common resources for transportation analysis techniques is included in Appendix A.

6 SEGMENT ANALYSIS

6.1 Purpose

For analysis purposes, roadway facilities are separated into categories that are specific to traffic flow type: Uninterrupted and Interrupted traffic flow.

This chapter presents commonly used segment (uninterrupted flow) analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Topics covered include:

- Freeways
- Multi-Lane Highways
- Two-Lane Highways

6.2 <u>Freeways</u>

The analysis of freeways is generally broken down into the major components of the freeway system including basic freeway segments, ramps and ramp junctions and weaving segments. The analysis procedures used for each of these components are described below.

6.2.1 Basic Freeway Segments

Basic freeway segments include the portions of freeway where flow is not influenced by the diverging, merging, or weaving associated with ramp/freeway connections. The common methodology used for analyzing basic freeway segment operations is from Chapter 23 of the *HCM*. The primary factors that affect operations on basic freeway segments include: lane widths, lateral clearance, the number of lanes, interchange density, heavy vehicles, grades and driver familiarity. For a complete description of the analysis methodology, refer to Chapter 23 of the *HCM*.

While the *HCM* methodology uses level of service as a performance measure (based on vehicle density in passenger cars per mile per lane), volume/capacity ratios can be calculated from this analysis for comparison against ODOT's adopted mobility standards by following the steps listed below.

1. Assuming level of service E/F threshold represents capacity, determine the segment capacity by interpolating between the values for "maximum service flow rate" at level of service E displayed in Exhibit 23-2 of the *HCM* for the appropriate free-flow speed. Free-flow speed will be either calculated by this methodology assumed to be 5 mph greater than posted, or observed in the field.
2. Divide the calculated flow rate (v_p) by the interpolated capacity to obtain a volume/capacity ratio. Note: The units are passenger cars per hour per lane (pcphpl), not vehicles per hour.

6.2.2 Ramps and Ramp Junctions

The analysis associated with operations at ramp junctions with the freeway mainline typically involves the effects of vehicles either merging onto or diverging from the mainline. The common methodologies used for analyzing these movements are those from Chapter 25 of the *HCM*. These methodologies focus on an influence area of 1,500 feet (downstream from ramp if merging and upstream from ramp if diverging). It should be noted that while the *HCM* methodology defines the influence area of merging or diverging traffic to be within 1,500 feet, the effects can extend outside of this area. The analysis for merging and diverging areas is discussed further below.

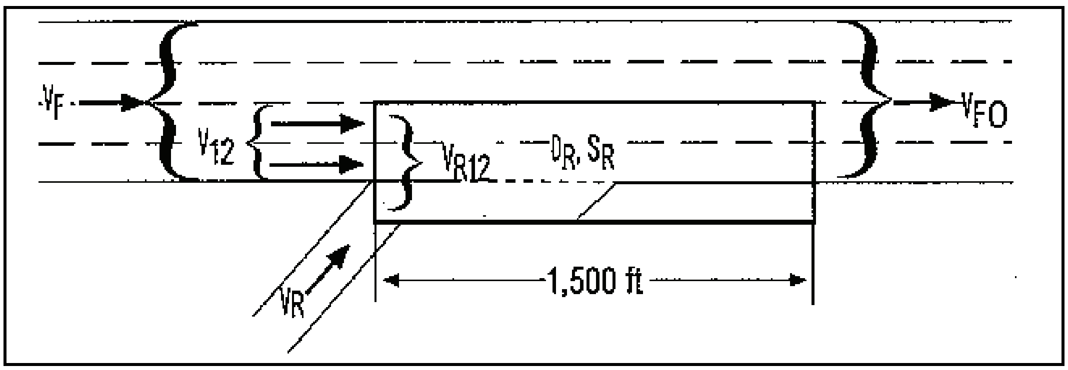
Merging Analysis

Merging analysis is often conducted at freeway on-ramps where vehicles from the ramp are entering a lane used by mainline traffic. In following the *HCM* methodology for merging analysis, there are three primary steps:

1. Predicting the flow rates entering lanes 1 and 2.
2. Determining capacity.
3. Determining level of service. Note that the performance measure of level of service is not used by ODOT and, therefore, this step will not be discussed.

The primary factors influencing the flow rates in lanes 1 and 2 (v_{12}) immediately upstream of the merge influence area are the total freeway flow rate approaching the merge area (v_F), the total ramp flow rate (v_R), the length of the acceleration lane and the ramp free-flow speed at the point of merging. The total flow rate entering the merge influence area (v_{R12}) is calculated by adding the flow rate remaining in lanes 1 and 2 (v_{12}) and the total ramp flow rate (v_R), as illustrated in Exhibit 6-1.

Exhibit 6-1 Freeway Merging Variables



Once the total flow rate entering the merge influence area (v_{R12}) has been calculated, it can be divided by the maximum desirable flow rate entering the merge influence area (4600 passenger cars per hour) to obtain a volume to capacity ratio for the merge influence area. When total flow rates for merge influence areas exceed capacity, locally high densities will occur, but freeway queuing will not always form as a result because mainline traffic will typically shift into the outermost lanes to avoid the merging traffic. Freeway queues are more likely to result in these situations where there are only two lanes for mainline traffic, forcing all vehicles to pass through the merge influence area. The *HCM* attempts to account for the amount of V_{12} traffic with the equations on *HCM* Exhibit 25-5. These equations are based on variables such as acceleration length, distance to next ramp, ramp volume, etc.

In addition to determining the volume to capacity ratio of the merge influence area, the volume to capacity of the downstream basic freeway segment should be checked to ensure the added traffic from the ramp does not create a downstream bottleneck. In cases where the total departing freeway flow rate (v_{FO}) is greater than the capacity of the downstream freeway segment (see Section 6.2.1), queues will form immediately downstream that will result in failure at the ramp connection, regardless of the whether flow rate entering the merge influence area has exceeded its capacity or not.

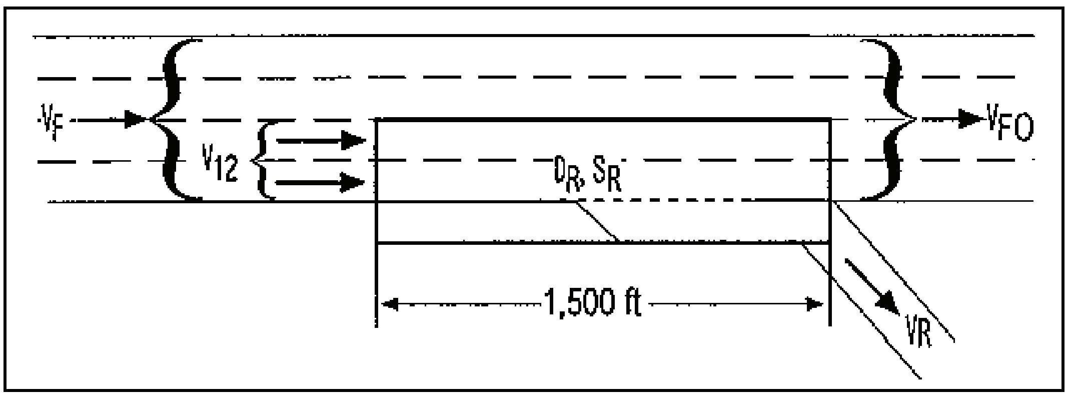
Exhibit 25-7 in the *HCM* displays capacities for merge areas including downstream freeway segment capacities (taken from Basic Freeway Segment chapter), as well as merge influence area capacities (where the maximum v_{R12} is always 4600 passenger cars per hour).

Diverging Analysis
Diverging analysis is often conducted at freeway off-ramps where vehicles from the mainline are departing to the ramp from a lane used by mainline traffic. The *HCM* methodology for diverging analysis is similar to that discussed above for merging, with three primary steps:

1. Predicting the approaching freeway flow in lanes 1 and 2.
2. Determining capacity.
3. Determining the density of flow within the ramp influence area. This step will not be discussed as the density is used to determine the performance measure of level of service, which is not used by ODOT.

For diverging analysis, the approaching flow rate in lanes 1 and 2 (v_{12}) is predicted for a point immediately upstream of the deceleration lane and includes the ramp flow rate (v_R) as illustrated in Exhibit 6-2. Models for predicting v_{12} can be found in Exhibit 25-12 of the *HCM*.

Exhibit 6-2 Freeway Diverging Variables



The primary cause of failure in diverge areas is inadequate capacity of an exit leg, whether on the freeway itself or the off-ramp. Capacities for downstream freeway legs can be obtained from Exhibit 25-14 (taken from Basic Freeway Segment chapter) from the *HCM*, and off-ramp capacities can be obtained from Exhibit 25-3. With these capacities known, volume to capacity ratios can be calculated by dividing the downstream freeway flow rate (v_{FO}) by the downstream freeway leg capacity and the ramp flow rate (v_R) by the ramp capacity.

Failure in diverge areas can also occur when the capacity of the freeway segment within the diverge area is exceeded. Capacities for upstream freeway segments can be obtained from Exhibit 25-14 (same as for downstream freeway segments) from the *HCM*. With this capacity known, a volume to capacity ratio can be calculated by dividing the freeway flow rate upstream of the diverge (v_F) by the capacity of the upstream freeway segment.

In addition to these conditions, the flow rate entering lanes 1 and 2 (v_{12}) immediately upstream of the deceleration lane should be checked to see if it exceeds the maximum desirable level. A volume to capacity ratio for this area can be calculated by dividing the approaching flow rate

(v_{12}) by the maximum desirable flow rate of 4400 passenger cars per hour (Exhibit 25-14 of *HCM*). Unlike the other conditions described above, the condition where the flow rate entering lanes 1 and 2 exceeds the maximum desirable level may create locally high densities, but may not always result in freeway queuing because mainline traffic will typically shift into the outermost lanes to avoid the diverging traffic. Freeway queues are more likely to result in these situations where there are only two lanes for mainline traffic, forcing all vehicles to pass through the diverging area.

6.2.3 Weaving Segments

Weaving Configurations

Another necessary step before the analysis can be conducted is the determination of the weaving type, which is based on the number of lane changes required of each weaving movement. The *HCM* methodology identifies three types of geometric configurations for weaving areas. Each of these types of configurations is described below, with diagrams provided in Exhibit 6-4.

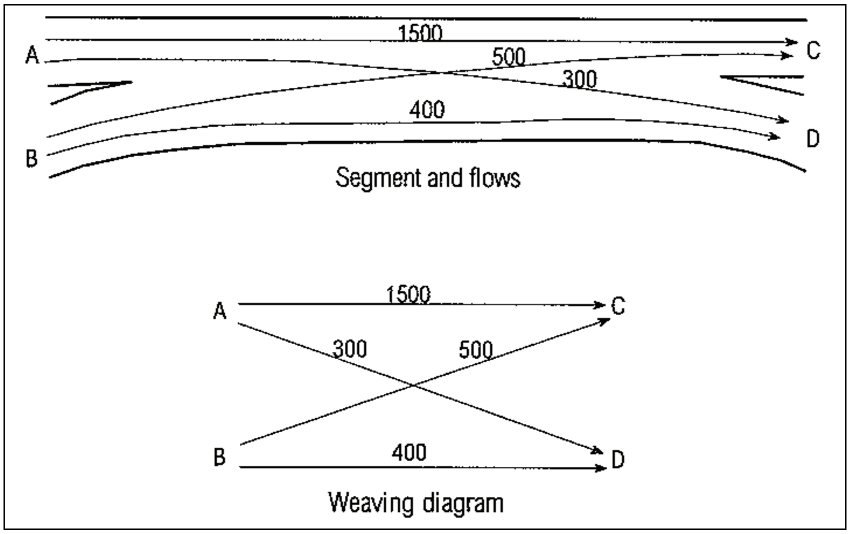
- **Type A**: Weaving vehicles in both directions must make one lane change to successfully complete a weaving maneuver.
- **Type B**: Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make one lane change to successfully complete a weaving maneuver.
- **Type C**: Weaving vehicles in one direction may complete a weaving maneuver without making a lane change, whereas other vehicles in the weaving segment must make two or more lane change to successfully complete a weaving maneuver.

Typically weaving segments are formed when merge areas are followed closely by diverge areas (within 2,500 feet) and the two are joined by an auxiliary lane requiring the crossing of two or more traffic streams traveling in the same general direction along a significant length of highway without the aid of traffic control devices. Note that when one-lane on-ramps are followed by one-lane off-ramps and the two are not connected by an auxiliary lane, weaving analysis is not conducted and the merge and diverge areas are analyzed independently using the procedures previously described. Recognition of configurations that could result in weaving is critical in highway operations analysis, as weaving areas require intense lane changing maneuvers that create a significant amount of turbulence. ODOT prefers the use of the *HCM* methodology for analyzing weaving maneuvers, but also supports the use of the Leisch Method in cases where engineering judgment suggests *HCM* results are not accurately reflecting conditions. For weaving areas greater than 2,500 feet use the more conservative of either the merge/diverge or Leisch methods.

The *HCM* discusses weaving concepts in Chapter 13 and the analysis methodology in Chapter 24. While most analysts will take advantage of the practicality of the Highway Capacity Software (HCS), which will perform all needed calculations to analyze weaving areas, it is important to have a basic understanding of weaving characteristics and key input parameters for use with HCS.

Weaving Diagrams
With a weaving area identified for analysis, a weaving segment diagram should be created to clearly identify the traffic flow rates associated with each movement, i.e., mainline to mainline, mainline to off-ramp, on-ramp to mainline, and on-ramp to off-ramp. An example of a weaving segment diagram is shown in Exhibit 6-3.

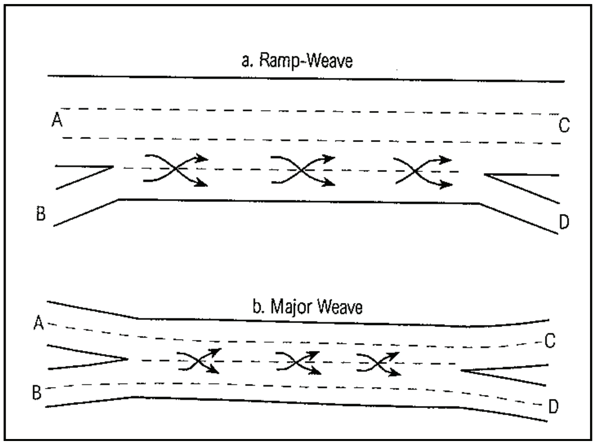
Exhibit 6-3 Weaving Diagram



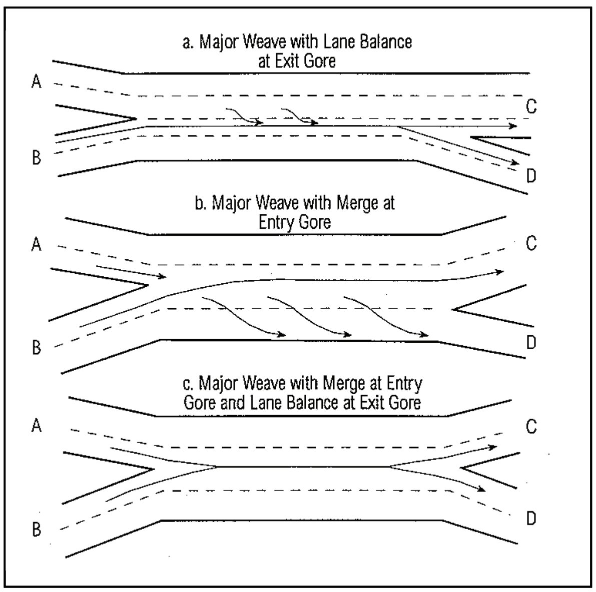
As can be seen, identifying the volume of traffic associated with each movement will require specialized data collection compared to typical counts that would only count traffic entering and leaving the area without noting its origin. Origin-destination surveys may provide the best data to use for a weaving analysis, but are not always practical to conduct and can be expensive. Some types of origin-destination studies, such as where traffic is stopped to conduct interviews, are more expensive than others such as license plate surveys. The less costly types of surveys are commonly through direct observation (usually recorded with video in the field and counted later) where all movements can be seen from one vantage point or through license plate identification. Another method sometimes available is the use of select-link output from a transportation demand model in combination with a common volume survey where a travel demand model has been created for the area. This method is especially useful for future scenarios where travel patterns may be different than current conditions. If either of these methods is not possible or practical for the particular area, the analyst may be required to apply engineering judgment in considering area characteristics such as land uses, topography and the area transportation network to create these movements from a common volume survey.

Weaving sections come in three configurations; Type A, B and C. Exhibit 6-4 shows the three types. Type A requires a lane change to get into or out of the auxiliary lane. Type A weaves are the most common type which occur mainly between interchanges that have a large portion of local trips that travel between them. High weaving volumes can cause Type A weaves to have poor operations. Type B weaves only require one lane change for either the mainline or ramp movement. These do not "trap" vehicles in the weaving section, so speeds are higher and operate much better than Type A weaves. Type C weaves require more than one lane change to perform the weaving maneuver and generally only operate well if the movement that must change lanes multiple times has a small volume. Type C weaves are relatively uncommon, are generally discouraged, but may exist in older highway alignments.

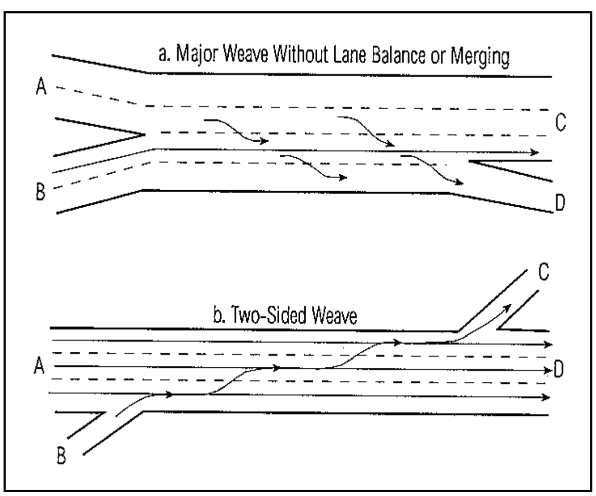
Exhibit 6-4 Weaving Configurations



Type A Configuration



Type B Configuration



Type C Configuration

Constrained vs. Unconstrained Conditions
Applying the weaving methodology, other geometric characteristics must be described including whether the weaving area is operating under constrained or unconstrained conditions and identifying the length of the weaving area. The determination of whether a weaving segment is operating under constrained or unconstrained conditions is based on the relationship between the number of lanes that must be used by weaving vehicles to achieve equilibrium with non-weaving vehicles (N_W) and the maximum number of lanes that can be used by weaving vehicles for a given configuration (N_W(max)). Where $N_W < N_W$(max), conditions are described as unconstrained because there are no impediments to weaving vehicles' ability to achieve equilibrium with non-weaving traffic. Where $N_W > N_W$ (max), conditions are considered to be constrained because weaving vehicles are not provided enough roadway width as would be needed to reach equilibrium. Under constrained operation weaving vehicles often experience operating conditions much worse than those experienced by non-weaving vehicles, while under unconstrained conditions weaving and non-weaving vehicles usually experience similar operating conditions.

The calculation of N_W and N_W(max) is determined by the configuration type, i.e., Type A, B, or C, and speeds of weaving and non-weaving vehicles. See Exhibit 24-7 in the *HCM*. When using the HCS to perform calculations, the analyst will only be required to determine the configuration type, free-flow speed and total number of lanes in the weaving section. However, an understanding of the characteristics of constrained and unconstrained conditions is important when analyzing weaving areas.

Weaving Length
Because weaving vehicles must execute all lane changes between the entry and exit gores, weaving lengths are measured from a point at the merge gore where the right edge of the freeway shoulder lane and the left edge of the merging lane are 2-feet apart to a point at the diverge gore where the two edges are 12-feet apart. Weaving lengths are limited to 2,500 feet in the *HCM* methodology. For weaving areas greater than 2,500 feet, use the more conservative of either the merge/diverge or Leisch methods.

Weaving Density
The key element of the *HCM* weaving analysis methodology is the calculation of the weaving area density, which is determined by incorporating weaving characteristics such as flow rate, configuration and free-flow speed. For a complete description of the density calculation refer to Chapter 24 of the *HCM*. The *HCM* uses the performance measure of level of service to rate weaving operations, which is directly related to the density calculated according to Exhibit 6-5.

Exhibit 6-5 Level of Service Criteria for Weaving Segments

Level of Service	Density (Passenger Cars/Mile/Lane)	
	Freeway Weaving Segment	Multi-Lane and Collector-Distributor* Weaving Segments
A	< 10.0	<12.0
B	10.0 – 20.0	12.0 – 24.0
C	20.0 – 28.0	24.0 – 32.0
D	28.0 – 35.0	32.0 – 36.0
E	35.0 – 43.0	36.0 – 40.0
F	>43.0	>40.0

* See page 24-19 of the HCM – research is unclear on applicability of LOS criteria to collector-distributor roads.

Weaving Capacity

While ODOT does not use level of service for evaluating facility performance, the density of the weaving section is still used to determine the volume to capacity ratio. If the capacity of the weaving section is equated to the level of service E/F threshold shown in Exhibit 6-5, then the capacity of a freeway weaving section would occur at a density of 43 passenger cars per mile per lane. The capacity in passenger cars per hour at this density can be found through the following iterative process.

1. Complete the analysis using the *HCM* methodology. While this methodology will produce a level of service, which is not needed, it will also produce a density.
2. The capacity of the weaving section will be equal to the total entering flow rate that results in a calculated density of 43 passenger cars per mile per lane (for freeways). Using the flow rates from the initial analysis, begin an iterative process by multiplying each movement flow rate by a common factor until the resulting density reaches, but does not exceed, 43 passenger cars per mile per lane.
3. Add the individual movement flow rates that produced the target density to obtain the total entering flow rate, which will be taken as the weaving section capacity.

The volume to capacity ratio for the section can now be calculated by dividing the original total entering flow rate by the capacity (total entering flow rate resulting in target density). This process of iteration will typically require fewer than ten attempts. The same procedure can be used for weaving analysis of non-freeway facilities, but a different target density for the capacity will be required, as shown in Exhibit 6-5 for multi-lane and collector-distributor roadways.

In addition to v/c ratio, the weaving section volume ration (VR) and speeds should be reported. The VR is the ratio of the weaving flow rate to the total flow rate. The *HCM* provides recommended upper limits on volume ratios. The difference between weaving and non-weaving speeds is a form of speed differential, which is preferred to be 10 mph or less for safety. Conditions exceeding these values should be examined using more detailed analysis methods such as simulation.

Example 6-1 Weave Capacity Example

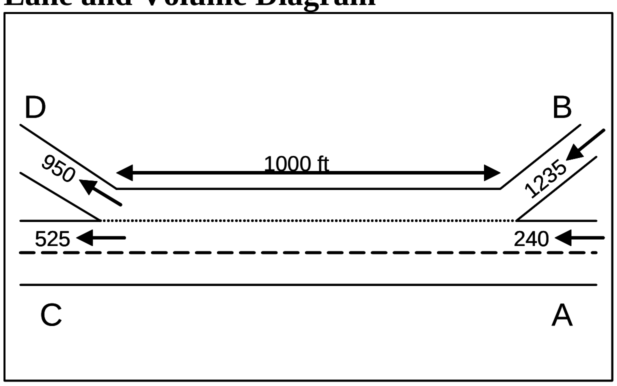
Given: Type A weave

- 12 ft lanes
- 6 ft lateral clearance
- 1000 ft weaving distance
- 35 mph posted speed
- Multilane highway segment
- 5% Trucks
- PHF = 0.95
- Driver population factor = 0.95
- Volumes in vehicles per hour
- Weaving and non-weaving flow distributions

Find: Volume-to-Capacity ratio for weaving section

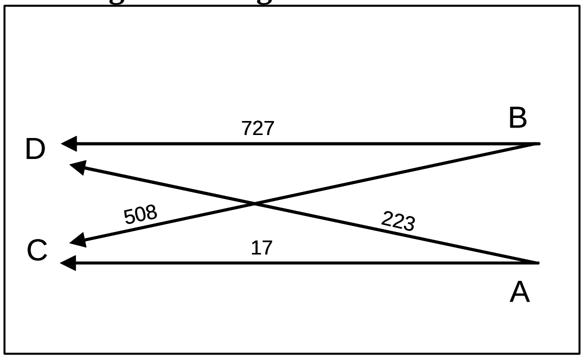
This example problem is based off of an actual project alternative. The lane and volume diagram shows the layout of the Type A weaving section and the volumes in vehicles per hour. The weaving section was created between a free-right turn at "B" and a loop off-ramp at "D" on a multilane roadway at an interchange.

Lane and Volume Diagram



The lane volumes were converted into weaving (A-D and B-C) and non-weaving (B-D and A-C) volumes as shown below. In this case, future distributions were available from a cumulative analysis procedure. Other sources of weaving volumes include field collected origin-destination data such as by tracking vehicle license plates. Where a travel demand model is present, select link runs can help estimate weaving movements.

Weaving Flow Diagram



The given information is then input into the *HCM* weaving procedure. The *HCM* result is a flow rate (in passenger cars per hour) of 1673 pc/h with a corresponding density of 17.41 pc/mi. The target density is 40 pc/mi, which is the density at capacity (v/c = 1.00) for a multilane or collector-distributor roadway. The table below was then iteratively created by multiplying the flow rates by a common factor until the density was as close as possible to 40 pc/mi.

Multiplied Flow Rates

Iteration	Factor	A-C (pc/h)	B-D (pc/h)	A-D (pc/h)	B-C (pc/h)	Flow rate (pc/h)	Density (pc/h)
1	1x	19	825	253	576	1673	17.41
2	2x	38	1650	506	1152	3346	47.83
3	1.5x	57	1238	380	864	2539	33.76
4	1.8x	34	1485	455	1037	3011	41.92
5	1.7x	32	1402	430	979	2843	39.04
6	1.74x	33	1436	440	1002	2911	40.20
7	1.73x	33	1427	438	996	2894	39.91
8	1.735x	33	1431	439	999	2902	40.05
9	**1.733x**	**33**	**1430**	**438**	**993**	**2899**	**39.99**

As can be seen from the last line in the table, the target density was reached at a flow rate of (rounded) 2900 pc/h. The 2900 pc/h flow rate is taken as the capacity in the v/c calculation. The sum of the original weaving and non-weaving flow rates is taken as the volume in the v/c calculation. The resulting v/c ratio would be:

Weaving v/c = 1673 / 2900 = 0.58

This v/c ratio is of an acceptable level. However, in doing the calculations it was found that the VR of 0.50 exceeds the maximum allowed by the methodology (0.45). Note: “c” at the end of *HCM* Exhibit 24-8 indicates that 3-lane type A segments do not operate well at volume ratios above .45, and may have poor operations and localized queuing. In addition, the difference between the weaving speeds (27 mph) and the non-weaving speeds (40 mph) is greater than 10 mph, which indicates a much greater potential crash risk. Simulation afterwards confirmed the poor operations as predicted even though the v/c ratio was acceptable.

It should also be noted, if using the HCS to perform calculations, that this program will provide warnings on the output sheet regarding limitations of this methodology that may not be reflected in the analysis results. It is the analyst’s responsibility to check these conditions to be sure the analysis results are valid. In addition, as with all types of analysis procedures, the analyst should verify that the results obtained appear to be reasonable for the given scenario. If they are not, the assumptions and input parameters should be reevaluated for errors. Should the results continue to appear inaccurate after making these types of adjustment, the analyst may consider applying a different methodology.

6.3 Multi-Lane Highways

Analysis procedures for uninterrupted-flow multi-lane highways are provided in Chapter 21 of the *HCM*. Highways analyzed with this procedure must maintain a minimum of two travel lanes in each direction, would typically have direct access allowed through driveways and at-grade intersections, and must maintain uninterrupted flow. Highways with access limited to on-ramps and off-ramps should be analyzed using the Basic Freeway Segment methodology. In addition, highways experiencing interrupted flow from influences such as traffic signals and on-street parking should be analyzed using a different methodology, such as the Urban Streets methodology from the *HCM*.

These procedures are very similar to those previously described for basic freeway segments, with slightly different input data needs. The most notable differences include the need to account for median type and access density. For a complete description of the analysis methodology, refer to Chapter 21 of the *HCM*.

While the *HCM* methodology uses level of service as a performance measure (based on vehicle density in passenger cars per mile per lane), volume/capacity ratios can be calculated from this analysis for comparison against ODOT's adopted mobility standards by following the steps listed below. Note that separate volume/capacity ratios must be calculated for each direction of travel.

1. Assuming level of service E/F threshold represents capacity, determine the segment capacity by interpolating between the values for "maximum service flow rate" at level of service E displayed in Exhibit 21-2 of the *HCM* for the appropriate free-flow speed. Free-flow speed will be either calculated by this methodology or assumed.
2. Divide the calculated flow rate (vp) by the interpolated capacity to obtain a volume/capacity ratio.

6.4 <u>Two-Lane Highways</u>

The *HCM* provides procedures for the operational analysis of two-lane highways modeled as two-way or directional segments, using level of service (LOS) and volume to capacity (v/c) ratios as performance measures. The application of these procedures on ODOT facilities is discussed below. For a complete description of the methodology, refer to Chapter 20 of the *HCM*.

6.4.1 Two-Way vs. Directional Analysis

While the ability to analyze two-lane highways as two-way or directional segments is offered by the *HCM*, only the analysis of ODOT facilities as directional segments is considered acceptable. The two-way analysis averages the two directions together, which can result in a combined v/c ratio that is within the adopted standard where the v/c ratio for one of the directions fails to meet the standard. In reality the v/c ratio of the highway is controlled by the highest direction, and should be reported as such.

Furthermore, the two-way analysis is not compatible with the multi-lane analysis. Because of this, it is possible to analyze a two-lane highway before and after widening to four lanes and obtain a higher v/c ratio for the four-lane condition. This inconsistency is corrected if the two-lane highway is analyzed by direction.

6.4.2 Performance Measures

The *HCM* defines the LOS for two-lane highway analysis by the percent time-spent-following and average travel speed. However, ODOT has no established standard for the amount of time-spent-following that is acceptable, or how such a standard might be different for various classifications of highways. Time spent following can be used for relative comparisons among alternatives. These measures can also understate performance in developed areas where driver expectations may be consistent with slower travel speeds and restricted passing opportunities. In addition, the calculation of the performance measures of percent time-spent-following and average travel speed have a large amount of uncertainty and error and, therefore, should not be considered reliable. As a result, only the v/c ratio will be considered as an acceptable measure of performance for two-lane highway segments.

The capacity of a two-lane highway is generally assumed to be 1,700 passenger cars per hour per direction of travel, with a maximum of 3,200 passenger cars per hour per direction of travel for both directions combined. To calculate the v/c ratio for a directional segment, the passenger car equivalent peak 15-minute flow rate is divided by the appropriate capacity.

6.4.3 Passing and Climbing Lanes

Both passing and climbing lanes are low-cost improvements that can be very effective in improving the operation of two-lane highways and can reduce the need to widen highways to four lanes. The *HCM* includes methodologies for analyzing these types of facilities in Chapter 20.

When analyzing either passing or climbing lanes it must be determined whether a no-passing restriction will be placed on opposing traffic in the area of the added lane. If passing by opposing traffic will not be allowed, the operations of opposing traffic must be reanalyzed to include this restriction.

While the methodologies described below can be used to evaluate the operations of passing and climbing lanes, the appropriate locations and lengths to use for design should be determined through the use of ODOT's HDM.

Passing Lanes

Passing lanes are typically used where there may be inadequate passing opportunities, either because of sight distance limitations or as traffic volumes approach capacity. By providing a safe place to pass, passing lanes tend to reduce unsafe passing maneuvers. In addition to improving operations in the segment containing the passing lane, operations of the highway downstream of the passing lane may also be improved for up to several miles before queues begin to reform. Exhibit 20-23 in the *HCM* shows the general relationship between the directional flow rate and the length of the downstream roadway affected. The *HCM* methodology is applicable to directional segments of two-lane highways that include the entire passing lane, and should also include the full effective downstream length (Exhibit 20-23), if possible.

A critical part of passing lane analysis using the *HCM* methodology includes dividing the analysis segment into four regions.

1. Upstream of the passing lane.
2. The passing lane, including tapers.
3. Downstream of the passing lane, but within its effective length.
4. Downstream of the passing lane, but beyond its effective length.

When using the Highway Capacity Software (HCS) to perform calculations, only the total segment length, length upstream of the passing lane and length of the passing lane are needed for input. The program will automatically calculate the other lengths based on these lengths and the directional flow rate. As with the Two-Lane Highway analysis, a volume to capacity ratio for a directional segment must be obtained by dividing the passenger car equivalent peak 15-minute flow rate by the appropriate capacity. For a complete description of the remaining analysis assumptions and methodology, see Chapter 20 in the *HCM*.

The analysis methodology in the *HCM* for passing lanes is intended to be applied to highways on level or rolling terrain only. Added lanes on mountainous terrain or on specific grades should be analyzed as climbing lanes.

Climbing Lanes

Climbing lanes are similar to passing lanes, but are generally used where grades cause unreasonable reductions in operating speeds of some vehicles. An unreasonable reduction in operating speeds is typically considered to occur where speed differentials of more than 10 mph are created. These lanes increase the capacity of a two-lane highway by providing a specific lane for slower vehicles to travel in while climbing an extended grade. This enables faster vehicles to pass these slower vehicles safely without having to leave the main travel lane. While climbing lanes are typically thought of as being associated with upgrades, they can also be applied to downgrades where heavy vehicles must drive in a low gear to avoid speeding out of control.

When analyzing the downgrade direction, passenger car equivalents for trucks operating at crawl speeds are available in Exhibit 20-18 of the *HCM*. For all other heavy vehicles, the passenger car equivalents in the *HCM* for level terrain should be used (Exhibit 20-9).

6.4.4 Other Analysis Procedures

Because of the deficiencies in the *HCM* procedures noted above, and the fact that volume to capacity ratios do not describe two-lane highway operations very well on their own, new methods and performance measures for evaluating two-lane highway operations are being considered. As an example, the Florida Department of Transportation has developed a modified version of the *HCM* methodology that more accurately reflects performance in developed areas by creating a unique class (Class III) for two-lane highways through developed areas, selecting percent free-flow speed as the performance measure and establishing new LOS thresholds to better reflect driver expectations in these areas. While ODOT has not accepted this methodology, it does represent one possible approach that will require further research.

7 INTERSECTION ANALYSIS

7.1 Purpose

This chapter presents commonly used intersection (interrupted flow) analysis procedures and identifies specific methodologies and input parameters to be used on ODOT projects. Topics covered include:

- Turn Lane Criteria
- Intersection Capacity Analysis
- Traffic Signal Warrants
- Estimating Vehicle Queue Lengths

7.2 Turn Lane Criteria

Proposed left or right turn lanes at unsignalized intersections and private approach roads must meet the installation criteria contained in the Highway Design Manual (HDM). Meeting the criteria does not require a turn lane to be installed. Engineering judgment must be used to determine if an installation would be safe and practical. The ODOT Traffic Manual provides further guidance on the use of right and left turn lanes.

7.2.1 Left Turn Lane Criteria – Unsignalized Intersections

Purpose

A left turn lane improves safety and increases the capacity of the roadway by reducing the speed differential between the through and the left turn vehicles. Furthermore, the left turn lane provides the turning vehicle with a potential waiting area until acceptable gaps in the opposing traffic allow them to complete the turn. Installation of a left turn lane must be consistent with the access management strategy for the roadway.

Left Turn Lane Evaluation Process

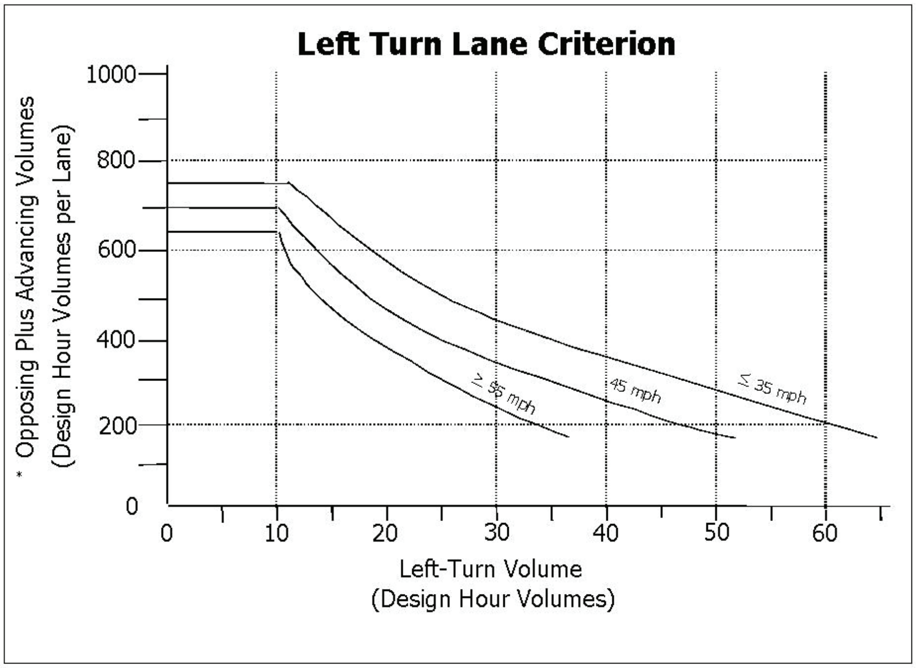
- A left turn lane should be installed, if criteria 1 (Volume) or 2 (Crash) or 3 (Special Cases) are met, unless a subsequent evaluation eliminate it as an option; and
- The Region Traffic Engineer must approve all proposed left turn lanes on state highways, regardless of funding source; and
- Complies with Access Management Spacing Standards; and
- Conforms to applicable local, regional and state plans.

Criterion 1: Vehicular Volume

The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a left turn lane. The volume criteria is determined by the Texas Transportation Institute (TTI) curves in Exhibit 7-1.

The criteria is not met from zero to ten left turn vehicle per hour, but indicates that careful consideration be given to installing a left turn lane due to the increased potential for accidents in the through lanes. While the turn volumes are low, the adverse safety and operations impacts may require installation of a left turn. The final determination will be based on a field study.

Exhibit 7-1 Left Turn Lane Criterion (TTI)



*(Advancing Volume/Number of Advancing Through Lanes) + (Opposing Volume/Number of Opposing Through Lanes)

Criterion 2: Crash Experience

The crash experience criteria are satisfied when:

1. Adequate trial of other remedies with satisfactory observance and enforcement has failed to reduce the accident frequency; and
2. A history of crashes of the type susceptible to correction by a left turn lane (such as where a vehicle waiting to make a left turn from a through lane was struck from the rear); and
3. The safety benefits outweigh the associated improvement costs; and
4. The installation of the left turn lane does not adversely impact the operations of the roadway.

Criterion 3: Special Cases

1. **Railroad Crossings**: If a railroad is parallel to the roadway and adversely affects left turns, a worst case scenario should be used in determining the storage requirements for the left turn lane design. The left turn lane storage length depends on the amount of time the roadway is closed, the expected number of vehicle arrivals and the location of the crossing or other obstruction. The analysis should consider all of the variables influencing the design of the left turn lane and may allow a design for conditions other than the worst case storage requirements, providing safety is not compromised.
2. **Passing Lane**: Special consideration must be given to installing a left turn lane for those locations where left turns may occur and other mitigation options are not acceptable.
3. **Geometric/Safety Concerns**: Consider sight distance, alignment, operating speeds, nearby access movements and other safety related concerns.

4. **Non-Traversable Median**: As required in the Median Policy, a left turn lane must be installed for any break in a non-traversable median.
5. **Signalized Intersection**: Consideration shall be given to installing left turn lanes at a signalized intersection. The State Traffic Engineer shall review and approve all proposed left turn lanes at signalized intersection locations on the state highway system.
6. **Other Conditions**: Other surrounding conditions, such as a drawbridge, could adversely affect left turns and must be treated in a manner similar to that for railroad crossings.

Evaluation Guidelines

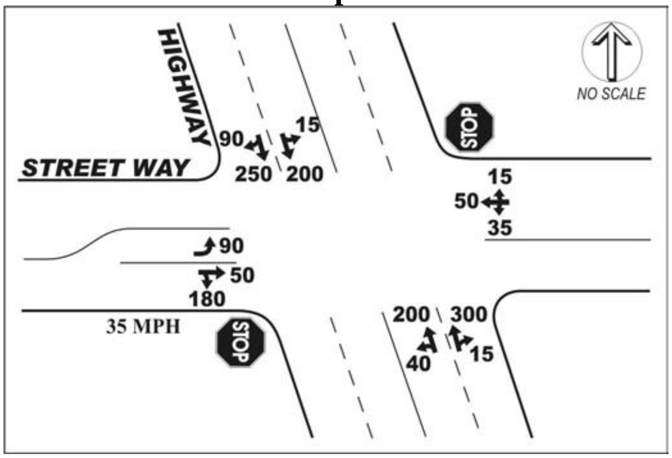
1. The **evaluation** should indicate the installation of a left turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the left turn lane should not be installed or, if already in place, not allowed to remain in operation.
2. **Alternatives Considered**: List all alternatives that were considered, including alternative locations. Briefly discuss alternatives to the left turn lane considered to diminish congestion/delays resulting in criteria being met.
3. **Access Management**: Address access management issues such as the long term access management strategy for the state roadway, spacing standards, other accesses that may be located nearby, breaks in barrier/curb, etc.
4. **Land Use Concerns**: Include how the proposed left turn lane addresses land use concerns and transportation plans.
5. **Plan**: Include a plan or diagram of proposed location of left turn lane.
6. **Operational Requirements**: Consider storage length requirements, deceleration distance, desired alignment distance, etc. For signalized intersections, installing a left turn lane must be consistent with the requirements in the Traffic Signal Guidelines.

Example 7-1 Left Turn Lane Criterion Example

<u>Left Turn Volume Criterion Example</u>

Volume Criterion Example shown below shows an unsignalized intersection with a shared through-right lane and a shared through-left lane on the Highway. The peak hour volumes and lane configurations are included in the figure. The 85th percentile speed is 45 mph and the intersection is located in a city with a population of 60,000. Do the NB and SB left turn movements meet the volume criterion?

Volume Criterion Example



- **Southbound**: The southbound advancing volume is 555 (90 + 250 + 200 + 15) and the northbound opposing volume is 515 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Exhibit 7-1 is determined using the equation:

  y-axis volume = ((Advancing Volume/Number of Advancing Lanes) +
  (Opposing Volume/Number of Opposing Lanes)) y-axis
  = (555/2 + 515/2) = 535

  To determine if the southbound left turn volume criteria is met, use the 45 mph curve in Exhibit 7-1, 535 for the y-axis and 15 left-turns for the x-axis. The volume criterion is not met in the southbound direction.

- **Northbound**: The northbound advancing volume is 555 (40 + 200 + 300 + 15) and the southbound opposing volume is 540 vehicles (the opposing left turns are not counted as opposing volumes). The volume for the y-axis on Exhibit 7-1 is (555/2+ 540/2) = 548. To determine if the southbound left turn volume criteria is met, use the 45 mph curve in Exhibit 7-1, 548 for the y-axis and 40 left-turns for the x-axis. The volume criterion is met in the northbound direction.

7.2.2 Right Turn Lane Criteria – Unsignalized Intersections

Purpose
The purpose of a right turn lane at an unsignalized intersection is to improve safety and to maximize the capacity of a roadway by reducing the speed differential between the right turning vehicles and the other vehicles on the roadway.

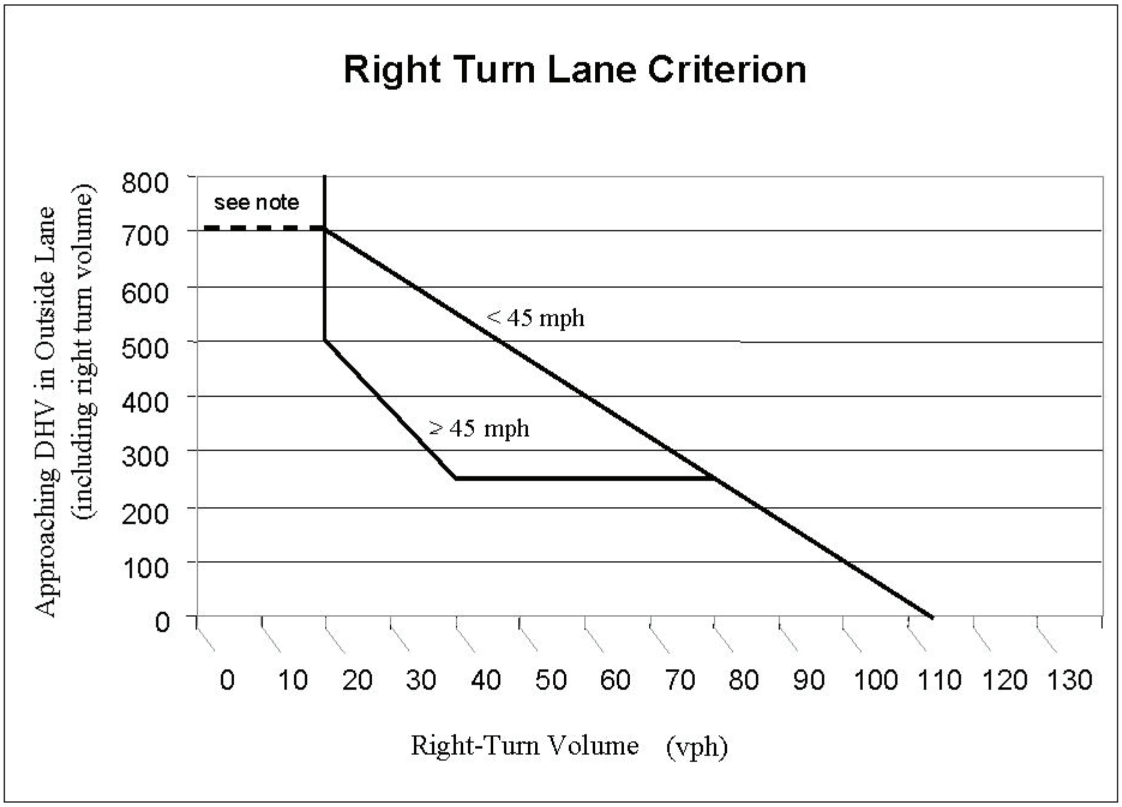
Right Turn Lane Evaluation Process

1. A right turn lane should be installed, if criteria 1 (Volume) or 2 (Crash) or 3 (Special Cases) are met, unless a subsequent evaluation eliminates it as an option; **and**
2. The Region Traffic Engineer must approve all proposed right turn lanes on state highways, regardless of funding source; **and**
3. Complies with Access Management Spacing Standards; **and**
4. Conforms to applicable local, regional and state plans.

<u>Criterion 1: Vehicular Volume</u>

The vehicular volume criterion is intended for application where the volume of intersecting traffic is the principal reason for considering installation of a right turn lane. The vehicular volume criteria are determined using the curve in Exhibit 7-2.

Exhibit 7-2 Right Turn Lane Criterion



Note: If there is no right turn lane, a shoulder needs to be provided. If this intersection is in a rural area and is a connection to a public street, a right turn lane is needed.

Criterion 2: Crash Experience

The crash experience criterion is satisfied when:

1. Adequate trial of other remedies with satisfactory observance and enforcement has failed to reduce the accident frequency; **and**
2. A history of crashes of the type susceptible to correction by a right turn lane; **and**
3. The safety benefits outweigh the associated improvements costs; **and**
4. The installation of the right turn lane minimizes impacts to the safety of vehicles, bicycles or pedestrians along the roadway.

Criterion 3: Special Cases

1. **Railroad Crossings**: If a railroad is parallel to the roadway and adversely affects right turns, a worst case scenario should be used in determining the storage requirements for the right turn lane design. The right turn lane storage length depends on the amount of time the roadway is closed, the expected number of vehicle arrivals and the location of the crossing or other obstruction. The analysis should consider all of the variables influencing the design of the right turn lane and may allow a design for conditions other than the worst case storage requirements, providing safety is not compromised.
2. **Passing Lane**: Special consideration must be given to installing a right turn lane for those locations where right turns may occur and other mitigation options are not acceptable.
3. **Geometric/Safety Concerns**: Consider sight distance, alignment, operating speeds, nearby access movements and other safety related concerns.
4. **Other Conditions**: Other surrounding conditions, such as a drawbridge, could adversely affect right turns and must be treated in a manner similar to that for railroad crossings.

Evaluation Guidelines

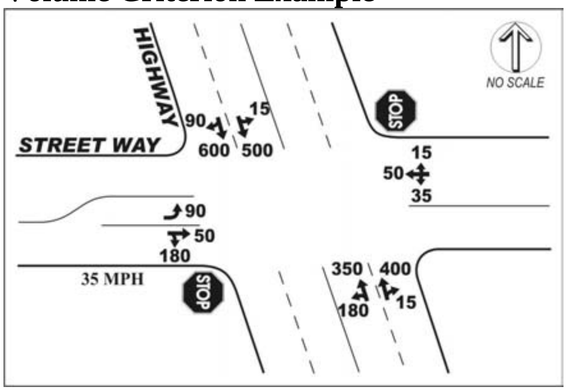
1. The **evaluation** should indicate the installation of a right turn lane will improve the overall safety and/or operation of the intersection and the roadway. If these requirements are not met, the right turn lane should not be installed or, if already in place, should be reevaluated for continued use.
2. **Alternatives Considered**: List all alternatives that were considered, including alternative locations. Briefly discuss alternatives to the right turn lane considered to diminish congestion/delays resulting in criteria being met.
3. **Access Management**: Address access management issues such as the long term access management strategy for the state roadway, spacing standards, other accesses that may be located nearby, breaks in barrier/curb, etc.
4. **Land Use Concerns**: Include how the proposed right turn lane addresses land use concerns and transportation plans.
5. **Plan**: Include a plan or diagram of proposed location of right turn lane.
6. **Operational Requirements**: Consider storage length requirements, deceleration distance, desired alignment distance, etc. For signalized intersections, installing a right turn lane must be consistent with the requirements in the Traffic Signal Guidelines.

Example 7-2 Right Turn Lane Criterion Example

Right Turn Vehicular Volume Criterion Example

Volume Criterion Example shown below shows an unsignalized intersection with a shared through-right lane and a shared through-left lane on the Highway. The peak hour volumes and lane configurations are included in the figure. The 85th percentile speed is 45 mph and the intersection is located in a city with a population of 60,000. Determine if a NB or SB right turn lane meets the criteria.

Volume Criterion Example



The northbound outside lane has 400 through vehicles and 15 right turning vehicles for a total of 415 vehicles. Using the 45 mph curve in Exhibit 7-2, along with 415 approaching vehicles and 15 right turning vehicles we find that the vehicular volume criterion is not met.

The southbound outside lane has 600 through vehicles and 90 right turning vehicles for a total of 690 vehicles. Using the 45 mph curve in Exhibit 7-2, along with 690 approaching vehicles and 90 right turning vehicles we find that the vehicular volume criterion is met.

7.2.3 Criteria for Turn Lanes at Signalized Intersections

Turn lanes at signalized intersections are determined differently than at unsignalized intersections. At signalized intersections a left turn lane is always desirable, while a right turn lane is generally determined based on signal capacity needs. At signalized intersections, installation of turn lanes must be consistent with the requirements in ODOT's Traffic Signal Policy and Guidelines and the Traffic Manual and approval must be received.

7.3 Intersection Capacity Analysis

7.3.1 Functional Area of Intersection

Addendum A

7.3.2 Effects of Upstream or Downstream Bottlenecks

Intersection analysis can be affected by upstream and downstream bottlenecks on the roadway network. If there is an upstream bottleneck it could restrict the flow of vehicles, ultimately reducing the potential for vehicles to access a study intersection. This potential reduction in vehicle volume at the intersection could result in a lower v/c ratio, indicating that "additional" capacity is available at the intersection. Improving areas that bottleneck could create new areas that fail, but previously indicated available capacity due to the original bottleneck. The analyst should be aware of the potential for an improvement to push a problem elsewhere.

Downstream bottlenecks can have a similar effect by producing a queue spillback, which would prevent vehicles from passing through a study intersection upstream. In this situation, the unserved vehicles will queue beyond the study intersection, but will not be captured as part of the demand when a vehicle count is collected. This low vehicle count at the study intersection could result in a capacity analysis that shows a lower v/c ratio than would be calculated if the bottleneck were not occurring.

7.3.3 Peak Demand Exceeds Operational Capacity

In general, analysis of existing conditions should not render results for v/c ratio calculations of greater than 1.0. This would indicate that more vehicles actually proceeded through an intersection than there is available capacity for. If a v/c ratio of greater than 1.0 is calculated for existing conditions, the default parameters used in analysis should be checked for reasonableness. A common cause of this is the use of default saturation flow rates that do not reflect actual conditions. If the existing v/c ratio calculated is greater than 1.10, the local field data should be checked and possibly additional data collected to refine the analysis. Also check the parameters that are used to calculate the adjusted saturation flow rate (PHF, lane utilization, etc.).

During future year analysis a v/c ratio calculation may result in a value higher than 1.0. This condition may result from a latent demand of vehicles at an intersection. This should be considered as a demand-to-capacity ratio (d/c) rather than an actual v/c ratio and would indicate conditions where mitigation could be considered to improve intersection operations.

7.3.4 Actual Versus Theoretical Conditions

When analysis is conducted on an intersection, the analysis is typically representative of isolated intersection operations. In actuality multiple factors may play a part in the operations of the intersection. These could be factors such as upstream or downstream intersections, coordinated signal systems, closely spaced intersections, etc. These factors should be considered when conducting signalized intersection analysis to help replicate what is actually occurring in the field.

7.3.5 Unsignalized Intersection Capacity

Capacity analysis for unsignalized intersections should generally follow the established methodology of the current HCM for both two-way and all-way stop control.

Two-Way Stop Control

For two-way stop control, the HCM employs a procedure for analyzing unsignalized intersections that is primarily based on an established hierarchy of intersection movements (based on assigned ROW) and a gap acceptance model. The major components of the gap acceptance model include the critical gap and follow-up time; where the critical gap is the minimum time interval in the major street traffic stream that allows intersection entry for one minor street vehicle and the follow-up time is the time between the departure of one vehicle from the minor street and the departure of the next vehicle using the same major street gap under a condition of continuous queuing on the minor street.

Substitution for the default values of critical gap and follow-up times used in the HCM shall only be permitted after conducting a thorough field investigation and obtaining ODOT approval.

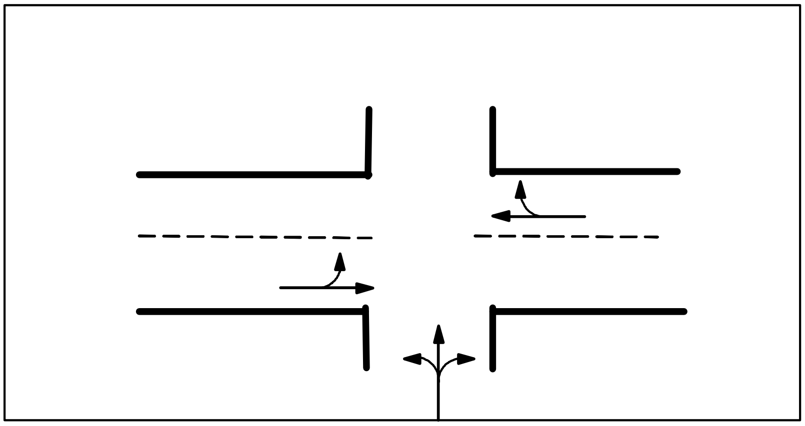
At two-way stop intersections, the controlling movement (usually a minor street left turn) often controls the overall intersection performance. Therefore, the v/c ratio for that movement will typically be the one reported and evaluated against the adopted mobility standard. This is especially important to recognize when analyzing two-way stop-controlled intersections where the very low v/c ratios for the unimpeded, high-volume major street movements will overshadow the higher v/c ratios for the lower-volume minor street movements. In these situations the unimpeded v/c ratio is often very low, even though the minor street movements are near or over capacity. However, as there may be times when the mainline v/c ratio is near the mobility standard, it should always be acknowledged before deferring to minor street movements.

Special Note

For intersections where the minor street is one-way: Synchro 6 and 7 do not use the proper gap times for an intersection with a one-way minor street, such as at an interchange ramp terminal. Synchro 6/7 are using the gap times appropriate for a four-legged intersection with four approaches; however, one-way minor street intersections have four legs, but only three approaches. See

Exhibit 7-3.

Exhibit 7-3 Two-Way Stop Control Intersection



The critical gap times (t_c) need to be changed for the minor street left turn only. The value is different depending on how many lanes are on the major street.

Critical Gap tc(s)

- Two Lane Major Street = 6.4
- Four/Six Lane Major Street = 6.8

All other critical gap times stay the same. After the value is changed it will be in red to indicate a user-overridden value. Deleting the value out and pressing “enter” will revert the value back to the default setting.

All-Way Stop Control
For all-way stop controlled intersections, the HCM procedure is based on an analysis of each approach independently. The procedure determines the capacity of each approach, which is used to calculate v/c ratios. The highest v/c ratio approach will be the one reported and evaluated against the adopted mobility standard.

7.3.6 Roundabouts

Roundabouts are increasing in popularity as an intersection form with less control. Research has shown roundabouts reduce crashes and vehicle delay. They are most effective with equal traffic on all approaches or with one-way streets (ramp terminals). However, they cannot supply as much capacity as traffic signals with multiple through or turn lanes and do not provide smooth progression of arterial traffic flows. The ODOT Traffic Manual and HDM contain roundabout guidelines, standards and siting criteria. Roundabout capacity analysis will employ the NCHRP Report 572 method.

TPAU has adopted the NCHRP Report 572 method for roundabouts. Unlike the earlier Highway Capacity Manual (HCM) Lower and Upper methods, the NCHRP Report 572 method is based on a statistically correct number of sites and observations so that definitive conclusions could be made. Procedures will likely be in the next update of the HCM 2000.

Report 572 shows U.S. drivers use roundabouts more conservatively than international drivers. Thus, the resulting roundabout capacity is lower than international calculation methods. Past use of SIDRA and German G2 methods was conservative, but correct, as they are relatively close to the NCHRP Report 572 resultant capacity.

NCHRP Report 572 Method
The general capacity equation is as follows:

$$C = A \bullet \exp\left(-B \bullet V_c\right)$$

Where

C = Entry capacity (passenger cars per hour; pc/h)
A = Coefficient
B = Coefficient
V_c = Circulating (conflicting) flow (pc/h)

A and B coefficient equations:

$$A = 3600 / t_f \qquad B = \frac{\left[(t_c - t_f) / 2 \right]}{3600}$$

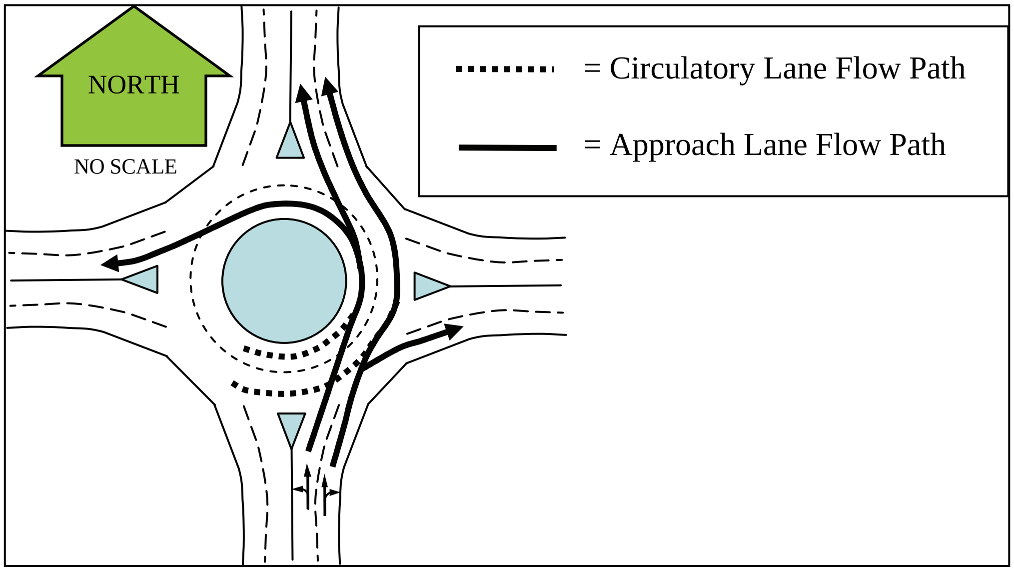
where

t_f = Follow-up headway (s)
t_c = Critical Headway (s)

t_f = Follow-up headway (s)
t_c = Critical Headway (s)

Conflicting flow is defined as the circulating flow that conflicts with the subject entry flow (Exhibit 7-4). Entry flow must yield to circulating flow. NCHRP research recommends circulating flows up to 1,200 pc/h for a single lane roundabout and from 200 pc/h to about 1,800 pc/h for a multilane roundabout.

Exhibit 7-4 Roundabout Circulatory and Approach Entry Flow



Average values of the A and B coefficients are used in the general equation. This equation can be customized to fit driver behavior in an area. When changing driver familiarity over time, NCHRP research found little correlation between t_c and t_f. Research also showed changes in the A and B coefficients are likely to be small. Obtaining t_c and tf values for A and B requires a statistically correct number of observations at multiple roundabouts in the same general area, an intensive effort.

Heavy Vehicle Factors and Passenger Cars Units

Volumes and resulting capacities in vehicles per hour (vph) are converted to passenger cars per hour (pc/h), using a heavy vehicle factor with a passenger car equivalent for trucks, ET. The heavy vehicle factor is calculated with the truck percentage and ET. The PHF and heavy vehicle factor are then used to calculate the entry/circulating flows.

At signals, the truck equivalency factor generally has a default value of 2.0, depending on approach conditions, HCM 2000 (page 16-10). The appendices to the NCHRP Report 572 state that the passenger car equivalent for heavy vehicles should be 2.0; roundabouts are no more efficient than signals in respect to the heavy vehicle flow rate. Roundabouts may even be worse than signals if heavy (long) vehicles can not merge into gaps in circulatory flow. A value of 2.0 should be used unless approach conditions warrant a different value, in such cases use the HCM, Chapter 20 (Exhibit 20-9) for two-lane highways and Chapter 21 (Exhibit 21-8) for multilane highways. In the HCM 2000, Exhibit 20-9, trucks must be subtracted out of the count (vph) in order to look up ET with passenger cars per hour (pc/h) flow rates. Total vehicles per hour should then be adjusted by the appropriate peak hour factor (PHF) from a recent count.

The 2003 HDM (page 9 - 36) states that roundabouts should not have high truck volumes, nor be located on grades that limit visibility or complicate construction. The 2003 HDM (9 - 36) also states that roundabout traffic should mostly comprise of commuter and local traffic. Therefore, the percent of trucks, buses and recreational vehicles should be low.

For level grade, the HCM 2000 requires the use of Equation 20-4 or 21-4.

$$F_{HV} = \frac{1}{1 + P_T(E_T - 1) + P_R(E_R - 1)}$$

where

P_T = portion of trucks in the traffic stream, expressed as a decimal
P_R = portion of RVs in the traffic stream, expressed as a decimal
E_T = passenger-car equivalent for trucks, obtained from Exhibit 20-9
E_R = passenger-car equivalent for RVs, obtained from Exhibit 20-9

For roundabout analysis, ODOT considers all heavy vehicles, such as recreational vehicles and buses, to be trucks in analysis, a conservative approach. According to the HCM 2000, combining recreational vehicles and buses with the truck classification is acceptable when the truck and bus percentage is five times the percentage of recreational vehicles. The HCM 2000, Chapter 12: Highway Concepts, states this combination is acceptable (at any ratio) if the relative proportions of recreational vehicles, trucks and buses are not known. As counts with recreational vehicles counted separately become available, this may be revisited.

With trucks, buses and recreation vehicles combined, the HCM 2000 heavy vehicle factor equation simplifies to:

$$F_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

P_T = portion of trucks in the traffic stream, expressed as a decimal
E_T = passenger-car equivalent for trucks, obtained from Exhibit 20-9

The heavy vehicle factor is then used to convert vehicles per hour (veh/h) into passenger cars per hour (pc/h) in the general form shown below. There are several forms of this equation in the HCM 2000.

$$V_p = \frac{V}{PHF \bullet F_{HV}}$$

where

V_p = 15 minute passenger car equivalent flow rate (pc/h)
V = hourly volume (veh/h)
PHF = Peak Hour Factor

For general use at single lane roundabouts, the capacity v/c ratio and control delay equations are:

Capacity

$$C = 1130 \bullet \exp\left(-B \bullet V_c\right)$$

where

$C =$ Entry capacity (passenger cars per hour; pc/h)
V_c = Circulating (conflicting) flow (pc/h)
B = Coefficient, 0.0010 for single lane roundabouts, 0.0007 for multilane

V/C Ratio

$$V/C_{leg} = \frac{Volume\ldots(pc/h)}{Capacity\ldots(pc/h)} = \frac{Entry\ldots Flow}{Entry\ldots Capacity}$$

Control Delay

$$d = \frac{3600}{C} + 900T\left[\frac{V}{C} - 1 + \sqrt{\left(\frac{V}{C} - 1\right)^2 + \frac{\left(\frac{3600}{C}\right)\frac{V}{C}}{450T}}\right]$$

where

d = Average control delay (s/veh)
C = Capacity of subject lane (veh/h)
T = Time period: T = 1 for actual one hour analysis, T = 0.25 if using PHF
V = Flow in subject lane (veh/h)

With Control Delay calculated, it is then used to determine LOS and queue lengths.

Multilane Roundabout
For multilane roundabouts, the critical entry lane is used. The critical entry lane, the entry lane with the highest volume, is determined by field observation and/or traffic count. If only one entry lane is available (planned), the volume is assigned to that lane. If multiple lanes are available, the volume is distributed based on field observations (data). In the case of an existing one lane approach, the NCHRP Report 572 recommends an equal through path distribution between the two future entry lanes (except for bypass lanes). It is better to assign lane distributions based on field observations (data). Traffic counts of intersections with two lane streets (existing) will obviously not show lane distributions for four lane streets (future need). Driveway counts of attractions near the intersection can be of use in predicting lane utilization. Location and characteristics of the area should be considered, such as nearby attractions and driver type.

The critical lane is generally the right lane of entry. The critical lane volume calculation determines the entry lane flow in the v/c and delay equations. The entire flow, in both circulatory lanes, is used to calculate circulating (conflicting) flow. Entering vehicles generally yield to vehicles in both circulatory lanes. Multilane roundabouts may not have two lanes completely circulate 360 degrees around the roundabout, having single lane circular sections. Approaches in these sections should use the single lane coefficient when calculating entry capacity.

Bypass Lanes
The bypass lane volume is subtracted out of the appropriate right turn volume. If the bypass lane flow yields or merges into an exiting flow (or lane) on a leg, then the capacity of the bypass lane should be calculated. The exiting flow is used as the circulating or conflicting flow and the bypass lane volume must yield as the entry flow. Use of the single or multilane capacity equation depends on how many lanes exit the roundabout. No calculation is necessary if the bypass lane enters as an add-lane on the exit leg.

V/C Ratio & Level of Service
After C or C_{crit} is obtained, the volume-to-capacity (v/c) ratio of the legs can be calculated.

$$V/C_{leg} = \frac{Volume\ldots(pc/h)}{Capacity\ldots(pc/h)} = \frac{Entry\ldots Flow}{Entry\ldots Capacity}$$

There is no overall v/c for a roundabout; all approach legs must be calculated. The reported v/c is the highest approach leg v/c calculated. The maximum v/c allowed is 0.80 on state highways. If an approach leg v/c ratio is over 0.80, the roundabout should not be constructed. However, the limiting v/c ratio is determined by the State Traffic Engineer (with consultation from Region Traffic) based on functional class, highway designation, traffic characteristics and system continuity.

Level of service (LOS) for a non-state highway roundabout follows the same approach delay methodology used for unsignalized intersections as shown in HCM 2000, Chapter 17. The individual approach LOS thresholds are also the same as unsignalized intersections. As with v/c, an individual LOS for each approach should be calculated, with the highest LOS controlling. Applicability or flexibility of local LOS standards should be considered.

Queuing

The NCHRP Report 572 method uses the same queuing model used in unsignalized intersections in the HCM 2000. This queuing model drastically underestimates the queues of major street left turns and minor street right turns at unsignalized intersections. A minor right turn at a roundabout is equivalent to the unsignalized intersection version with slightly less delay at the yield-controlled approach. The queuing model might not predict a higher level of delay, since the likelihood of a vehicle being stopped on approach is higher with larger circulating peak hour flows.

TPAU has researched queuing methods and adopted the Two-Minute Rule for queues at roundabouts, as used for unsignalized intersections, until further evaluation is made. Unmodified volumes, in vehicles per hour (vph), should be used instead of passenger car (pc/h) for queuing calculations.

Logical Analysis and Design Progression

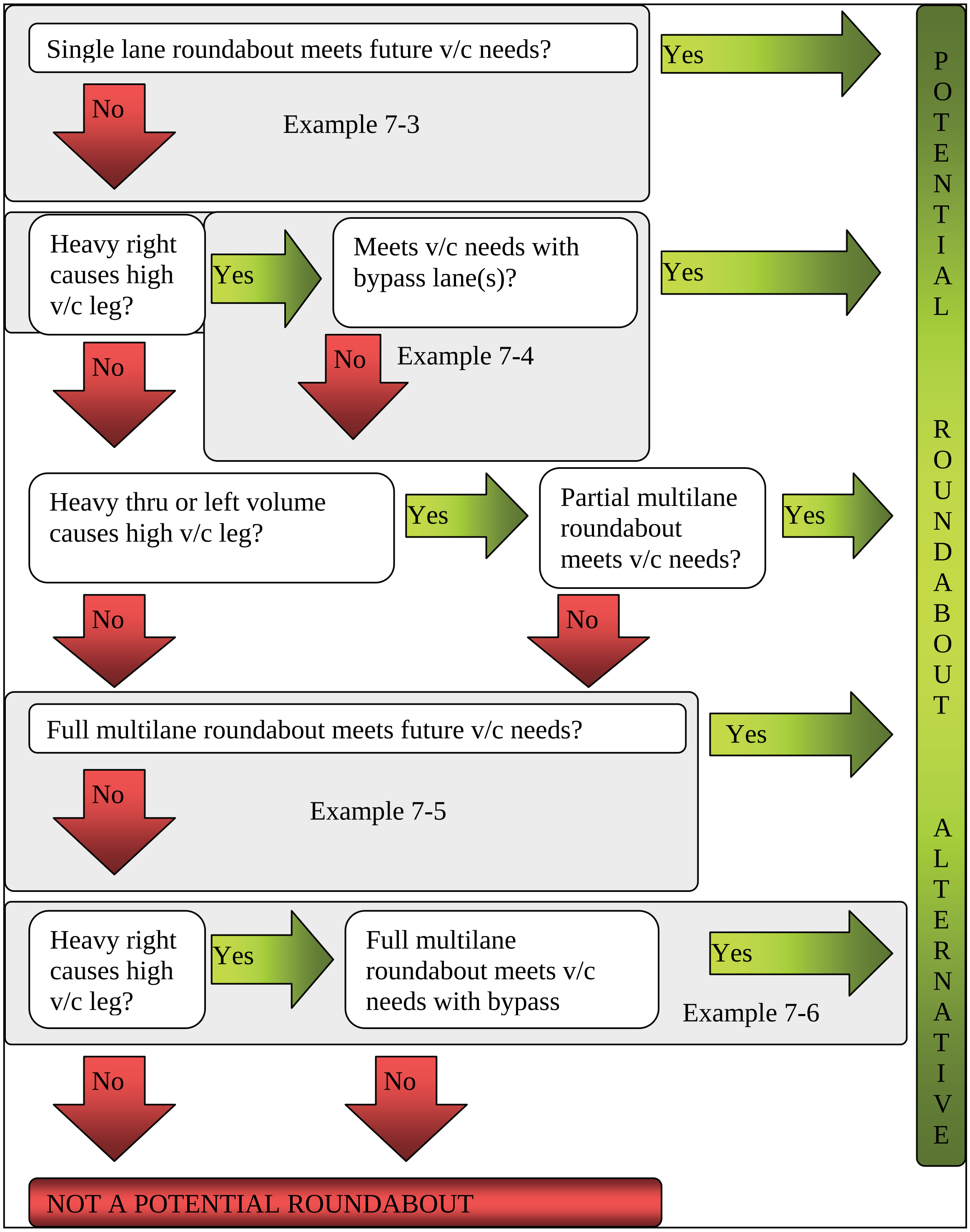
Start your analysis by considering a single lane roundabout with future volumes 20 years beyond the build date. If an approach v/c is over 0.80 due to right turn volumes, then a bypass lane should be analyzed for that movement. When the bypass lane traffic merges into an exiting lane, then the capacity of the bypass lane should be calculated (see Example 7-4). The bypass lane capacity is calculated, much like one leg of a roundabout, with the lane exiting the roundabout having the right of way (as with circulating flow) and the bypass lane yielding (approach lane flow) when there is no add lane. If the high v/c on an approach is not due to a heavy right turn movement, then a partial multilane roundabout should be considered (not all of the circulating lanes must have more than one lane). A full multilane roundabout should be analyzed next. If a multilane roundabout approach has a v/c over 0.80 due to a heavy right turn movement, then consider a bypass lane. A heavy left or thru movement causing a high v/c ratio indicates a roundabout is a poor alternative, based on volume and capacity. Exhibit 7-5 shows the design progression in flow chart form.

Roundabout Siting Considerations
Along with the analysis of a roundabout, the HDM 2003 states that proposed roundabouts:

- Should not have more than 4 approach legs.
- Should meet acceptable v/c ratios for the proposed design life.
- Should have approach roadway posted speeds 35 mph or less.
- Should have normal circular geometry.
- Should have similar or balanced volumes on all approach legs.
- Should be at an intersection of two highways with roughly the same functional classification or no more than one level of difference.
- Should be mostly commuter and local traffic (not freight).
- Should not have high pedestrian volumes.
- Should not have high volumes of large trucks.
- Should not be located within an interconnected signal system.
- Should not be in locations where exiting vehicles would be interrupted by queues from signals, railroads, drawbridges, ramp meters or by operational problems created by left turns, accesses, etc.
- Should not be located where grades or topography limit visibility or greatly complicate construction.

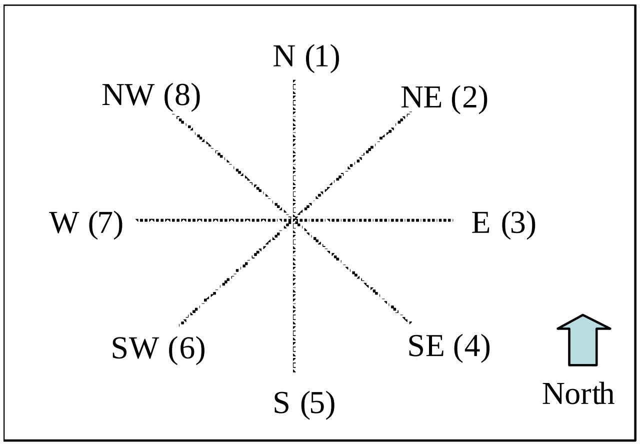
For roundabouts on State Highways, remember that roundabouts deflect vehicle paths in order to reduce speed, HCM 2000 (Chapter 10). Speed into and through roundabouts is controlled by physical features of the roundabout and not by signs or pavement markings. Consideration should be made to the proximity of a fire or police station or emergency route. Signals can offer signal preemption to ensure emergency vehicles pass through an intersection uninhibited, but roundabouts do not provide as much control.

Exhibit 7-5 Roundabout Volume/Capacity Analysis and Design Progression



Roundabout Calculator Spreadsheets
Single and multilane roundabout calculator spreadsheets have been developed to expedite capacity and queuing calculations. These roundabout spreadsheets are available on the TPAU analysis software webpage: http://www.oregon.gov/ODOT/TD/TP/TAS.shtml. Eight leg positions are available for geometric flexibility (Exhibit 7-6). Volumes are entered (Exhibit 7-7) in a "from" (top of column) "to" (left side) format to assign volumes to legs. ET, PHF and percent trucks are input for each leg. The spreadsheet calculates the circulating flows and entry lane capacity. Entry leg v/c ratio, approach control delay, LOS and the 95th percentile queue (using the two minute rule) are calculated for each leg with entry volumes entered.

Exhibit 7-6 Geometric Flexibility Diagram, directions numbered

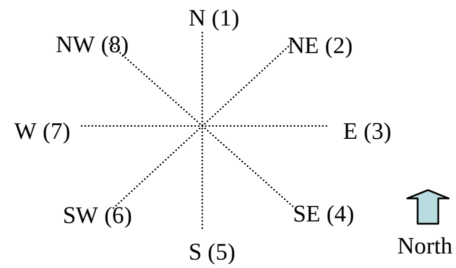


Even though each spreadsheet can handle up to eight leg positions, the 2003 HDM states that roundabouts should not have more than four approach legs and normal circular geometry.

Single and multilane roundabout calculator spreadsheets are shown in Exhibit 7-7 and Example 3. Making an entry other than the default PHF, ET or the truck percentage will highlight the entry background yellow, showing changes to the spreadsheet (Exhibit 7-7, under Volume Characteristics).

Exhibit 7-7 Sample Single Lane Calculation Spreadsheet

General & Site Information	
Analyst:	Count Carsin Circle
Agency/Company:	State Highway DOT
Date:	1/9/2008
Project Name:	City desires a
Intersection:	Mill Street and Elm
Analysis Time Period:	4 to 5 PM (with PHF)
Jurisdiction:	City / ODOT
Year:	20 years beyond Build



Volumes		Roundabout Approach/Entry Legs							
		N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
Input	N (1), vph	0		25		100		300	
Volumes	NE (2), vph								
to Leg #	E (3), vph	15		0		600		425	
	SE (4), vph								
	S (5), vph	55		45		0		100	
	SW (6), vph								
	W (7), vph	65		235		70		0	
	NW (8), vph								
Output	Total Vehicles	135	0	305	0	770	0	825	0

Volume Characteristics	N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
% Trucks	4.0	0.0	5.0	0.0	7.0	0.0	1.5	0.0
E_t	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
PHF	0.89	0.92	0.92	0.92	0.92	0.92	0.90	0.92
F_{HV}	0.962	1.000	0.952	1.000	0.935	1.000	0.985	1.000

Entry/Conflicting Flows		N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
Flow to Leg #	N (1), pc/h	0	0	29	0	116	0	338	0
	NE (2), pc/h	0	0	0	0	0	0	0	0
	E (3), pc/h	18	0	0	0	698	0	479	0
	SE (4), pc/h	0	0	0	0	0	0	0	0
	S (5), pc/h	64	0	51	0	0	0	113	0
	SW (6), pc/h	0	0	0	0	0	0	0	0
	W (7), pc/h	76	0	268	0	81	0	0	0
	NW (8), pc/h	0	0	0	0	0	0	0	0
	Entry flow, pc/h	158	0	348	0	896	0	930	0
	Conflicting flow, pc/h	401	884	536	1731	835	1064	133	559

Results	N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
Entry Capacity, pc/h	757	NA	661	NA	490	NA	989	NA
Leg v/c ratio	0.21		0.53		1.83		0.94	
Control Delay, s/pc	6.0		11.3		395.0		31.7	
LOS	A		B		F		D	
95th Percentile Queue (ft)	221	0	508	0	1320	0	1292	0

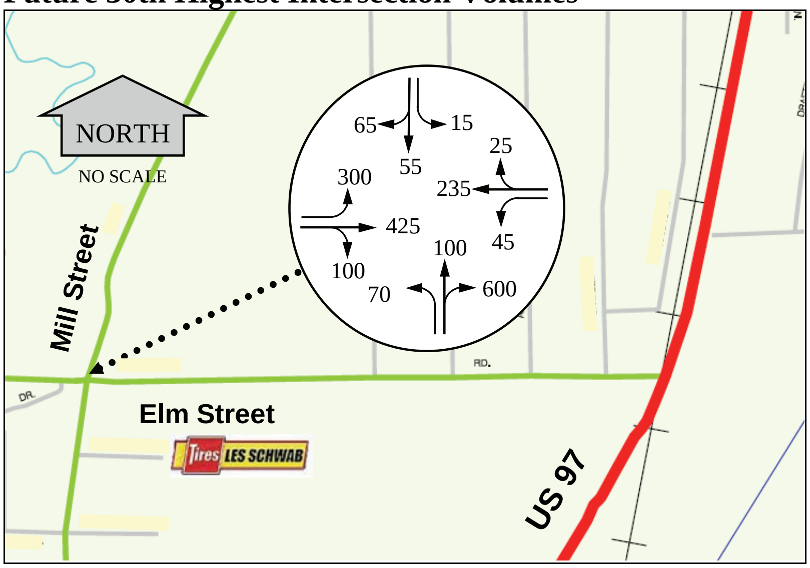
Example 7-3 Single Lane Roundabout Calculation

A roundabout is proposed for the intersection of Mill Street and Elm Street. A single lane roundabout will be analyzed in this example. The figure below shows the intersection volumes 20 years beyond the build date.

Volume

The first step is to take the 30th highest hour intersection volumes (see figure below) and enter them into the spreadsheet (see following section of spreadsheet), traveling from approach legs (top row) to exit legs (left column).

Future 30th Highest Intersection Volumes



Entry Leg Volume Table

Volumes		Roundabout Approach/Entry Legs							
		N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
Input	N (1), vph	0		25		100		300	
Volumes	NE (2), vph								
to Leg #	E (3), vph	15		0		600		425	
	SE (4), vph								
	S (5), vph	55		45		0		100	
	SW (6), vph								
	W (7), vph	65		235		70		0	
	NW (8), vph								
Output	Total Vehicles	135	0	305	0	770	0	825	0

Heavy Vehicle Factors and Passenger Cars Units
The PHF and percent trucks for the intersection should be attained from a recent count. The default value for ET for each approach will be used. The heavy vehicle factor is then calculated with the truck percentage and ET (see section of spreadsheet below). Using the north leg heavy vehicle factor calculation as an example, with trucks, bus and recreation vehicles combined, the HCM 2000 Two-Lane Highways Equation 20-4 simplifies to:

$$F_{HV} = \frac{1}{1 + P_T(E_T - 1)} = \frac{1}{1 + 0.04 \bullet (2.0 - 1)} = 0.962$$

where

P_T = portion of trucks in the traffic stream, expressed as a decimal
E_T = passenger-car equivalent for trucks, obtained from Exhibit 20-9

FHV Calculation in Volume Characteristics

Volume Characteristics	N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
% Trucks	4.0	0.0	5.0	0.0	7.0	0.0	1.5	0.0
E_t	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
PHF	0.89	0.92	0.92	0.92	0.92	0.92	0.90	0.92
F_{HV}	0.962	1.000	0.952	1.000	0.935	1.000	0.985	1.000

The PHF and heavy vehicle factor are used to calculate the entry/circulating flows (see section of spreadsheet below). The flows are calculated in passenger car equivalents with the following equation.

$$V_p = \frac{V}{PHF \bullet F_{HV}} = \frac{15 veh/h}{0.89 \bullet 0.962} = 18\, pc/h$$

where

V_p = 15 minute passenger car equivalent flow rate (pc/h)
V = hourly volume (veh/h)
PHF = Peak Hour Factor

Circulating Flow Output from Spreadsheet

Entry/Conflicting Flows	N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
Flow to Leg # N (1), pc/h	0	0	29	0	116	0	338	0
NE (2), pc/h	0	0	0	0	0	0	0	0
E (3), pc/h	18	0	0	0	698	0	479	0
SE (4), pc/h	0	0	0	0	0	0	0	0
S (5), pc/h	64	0	51	0	0	0	113	0
SW (6), pc/h	0	0	0	0	0	0	0	0
W (7), pc/h	76	0	268	0	81	0	0	0
NW (8), pc/h	0	0	0	0	0	0	0	0
Entry flow, pc/h	158	0	348	0	896	0	930	0
Conflicting flow, pc/h	401	884	536	1731	835	1064	133	559

Capacity and V/C Calculation
The capacity and v/c calculations are:

Capacity

$$C = 1130 \bullet \exp\left(-B \bullet V_c\right)$$

where

C = Entry capacity (passenger cars per hour; pc/h)
V_c = Circulating (conflicting) flow (pc/h)
B = Coefficient, 0.0010 for single lane roundabouts

The north leg, N (1), capacity is:

$$C_{N(1)} = 1130 \bullet \exp(-0.0010 \bullet 401) = 757 \text{ pc/h}$$

With the capacity known, the volume to capacity ratio (V/C) of the north leg is:

$$V/C_{N(1)} = \frac{Volume\ldots(pc/h)}{Capacity\ldots(pc/h)} = \frac{Entry\ldots Flow}{Entry\ldots Capacity} = \frac{158}{757} = 0.21$$

Control Delay
The average control delay equation is:

$$d = \frac{3600}{C} + 900T\left[\frac{V}{C} - 1 + \sqrt{\left(\frac{V}{C} - 1\right)^2 + \frac{\left(\frac{3600}{C}\right)\frac{V}{C}}{450T}}\right]$$

where

d = Average control delay (s/veh)
C = Capacity of subject lane (veh/h)
T = Time period: T = 1 for actual one hour analysis, T = 0.25 if using PHF
V = Flow in subject lane (veh/h)

$$d = \frac{3600}{757} + 900(0.25)\left[\frac{158}{757} - 1 + \sqrt{\left(\frac{158}{757} - 1\right)^2 + \frac{\left(\frac{3600}{757}\right)\frac{158}{757}}{450(0.25)}}\right] = 6.0 \text{ s/veh}$$

The results table calculates entry capacity, v/c ratio, control delay, LOS and 95th percentile queue (see below). The v/c ratio of the S (5) and W (7) legs do not meet standard.

Results Table

Results	N (1)	NE	E (3)	SE	S (5)	SW (6)	W	NW
Entry Capacity, pc/h	757	NA	661	NA	490	NA	989	NA
Leg v/c ratio	0.21		0.53		1.83		0.94	
Control Delay, s/pc	6.0		11.3		395		31.7	
LOS	A		B		F		D	
95th Percentile Queue	221	0	508	0	1320	0	1292	0

The south approach v/c over 1.0 is unacceptable. To mitigate this high v/c, a bypass lane should be considered because of the heavy right turn causing the high approach v/c (Example 7-4).

Example 7-4 Single Lane Roundabout with Bypass Lane Calculation

For this example, the south leg (5), with a heavy right turn volume, will be analyzed for a bypass lane, subtracting the approach right turn volume (following Exhibit 7-5 flow chart). This subtraction will lower the S (5) approach leg volume from 770 to 170 (see highlighted cell in spreadsheet below).

Entry Leg Volume Table

Volumes		Roundabout Approach/Entry Legs							
		N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
Input	N (1), vph	0		25		100		300	
Volume	NE (2), vph								
to Leg #	E (3), vph	15		0		0		425	
	SE (4), vph								
	S (5), vph	55		45		0		100	
	SW (6), vph								
	W (7), vph	65		235		70		0	
	NW (8), vph								
Output Total Vehicles		135	0	305	0	170	0	825	0

The truck percentage of the S (5) approach should be checked as it may have changed. The 600 vehicles (including trucks) no longer enter the roundabout, check the count for the truck percentage of the movement. The combined truck percentage for the through and left movements should be used. The section of the spreadsheet below shows the percent trucks input change and the change in the heavy vehicle factor as a result of removing some (from 770 to 170) of the vehicles approaching the roundabout (see above).

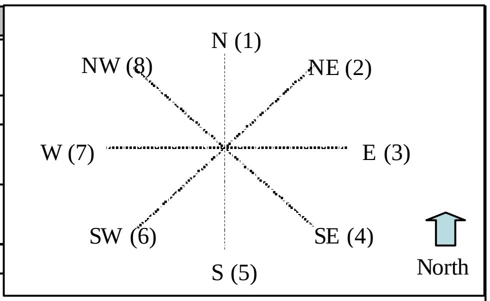
Volume Characteristics with Bypass Lane Volume Removed

Volume Characteristics	N (1)	NE (2)	E (3)	SE (4)	S (5)	SW (6)	W (7)	NW (8)
% Trucks	4.0	0.0	5.0	0.0	4.0	0.0	1.5	0.0
E_t	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
PHF	0.89	0.92	0.92	0.92	0.92	0.92	0.90	0.92
F_{HV}	0.962	1.000	0.952	1.000	0.962	1.000	0.985	1.000

As already stated, if the bypass lane flow yields or merges into an exiting flow (or lane) on a leg, the capacity of the bypass lane should be calculated. The exiting flow is used as the circulating or conflicting flow and the bypass lane volume yields as the entry flow. Use of the single or multilane capacity equation depends on how many lanes exit the roundabout. Calculation of the bypass lane capacity is not necessary if the bypass lane enters an add-lane on the exit leg. The PHF should also be checked, but in this case it remains the same.

Single lane Spreadsheet, Bypass Lane Volume Removed

General & Site Information	
Analyst:	Count Carsin Circle
Agency/Company:	ODOT
Date:	current date
Project Name:	City wants roundabout
Intersection:	Mill and Elm Street
Analysis Time Period:	4 to 5 PM (with PHF)
Jurisdiction:	City/ODOT
Year:	20 years beyond build



Volumes		Roundabout Approach/Entry Legs							
		N (1)	**NE (2)**	**E (3)**	**SE (4)**	**S (5)**	**SW (6)**	**W (7)**	**NW (8)**
Input	N (1), vph	0		25		100		300	
Volumes	NE (2), vph								
to Leg #	E (3), vph	15		0		0		425	
	SE (4), vph								
	S (5), vph	55		45		0		100	
	SW (6), vph								
	W (7), vph	65		235		70		0	
	NW (8), vph								
Output	Total Vehicles	135	0	305	0	170	0	825	0

Volume Characteristics	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
% Trucks	4.0	0.0	5.0	0.0	4.0	0.0	1.5	0.0
E_t	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
PHF	0.89	0.92	0.92	0.92	0.92	0.92	0.90	0.92
F_{HV}	0.962	1.000	0.952	1.000	0.962	1.000	0.985	1.000

Entry/Conflicting Flows	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
Flow to Leg # N (1), pcu/h	0	0	29	0	113	0	338	0
NE (2), pcu/h	0	0	0	0	0	0	0	0
E (3), pcu/h	18	0	0	0	0	0	479	0
SE (4), pcu/h	0	0	0	0	0	0	0	0
S (5), pcu/h	64	0	51	0	0	0	113	0
SW (6), pcu/h	0	0	0	0	0	0	0	0
W (7), pcu/h	76	0	268	0	79	0	0	0
NW (8), pcu/h	0	0	0	0	0	0	0	0
Entry flow, pcu/h	158	0	348	0	192	0	930	0
Conflicting flow, pcu/h	399	879	531	1027	835	1064	133	556

Results	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
Entry Capacity, pcu/h	758	NA	665	NA	490	NA	989	NA
Leg v/c ratio	0.21		0.52		0.39		0.94	
Control Delay, s/pcu	6.0		11.2		12.0		31.7	
LOS	A		B		B		D	
95th Percentile Queue (ft)	221	0	508	0	279	0	1292	0

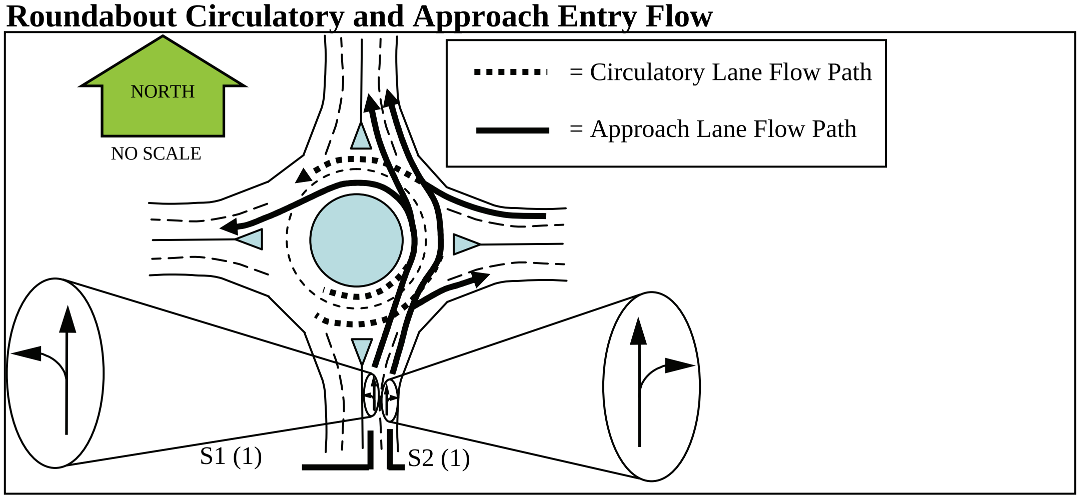
The v/c of leg S (5) was over 1.0 (Results Table), but is now acceptable at 0.39 (see above) with a bypass lane taking the right turn movements out of the roundabout. The west leg approach still has a v/c over 0.80, due to a heavy through and left turn volume (see above). The next step of the analysis progression would be to look at a partial multilane roundabout (see flowchart). The next example skips right to the full multilane roundabout analysis.

Example 7-5 Multi-Lane Roundabout Example

A heavy through and/or left turn movement cannot be improved with a bypass lane. Therefore, this example will now consider a multilane roundabout alternative. While this example has jumped steps to a full multilane analysis, one should first consider a partial multilane roundabout. For partial multilane roundabouts, with a single lane section, both analysis methods are required (both spreadsheets). Application depends upon the number of lanes in each section.

Volume
A key difference in the multilane spreadsheet is the volume inputs for each of two approach lanes (see figure and spreadsheet below). There are two lanes (columns) for each leg. The lane with a 1 next to the geometric leg, S1 (1), is the inside approach lane and the lane with a 2, S2 (1), is the outside approach (curb) lane. The geometric leg, S and the number in parenthesis, (1), designate the approach for both lanes. Approaches with only one lane should consistently use one lane.

Roundabout Circulatory and Approach Entry Flow



Lane distribution of the right turn movements should be to the outside lane and left turn movements to the inside lane (see above). For this example, field observation indicates lane distribution of 65 percent of the approach lanes through movements using the outside lane (see below).

Multilane Approach Leg Volume Table

Volumes	Roundabout Approach/Entry Legs							
	N1 (1)	N2 (1)	NE1 (2)	NE2 (2)	E1 (3)	E2 (3)	SE1 (4)	SE2 (4)
Volumes N (1),vph	0	0			0	25		
to Leg # NE (2),vph								
E (3), vph	0	15			0	0		
SE (4), vph								
S (5), vph	19	36			45	0		
SW (6), vph								
W (7), vph	65	0			82	153		
NW (8), vph								
Entry Volume, vph	84	51	0	0	127	178	0	0
	S1 (5)	S2 (5)	SW1 (6)	SW2 (6)	W1 (7)	W2 (7)	NW1 (8)	NW2 (8)
N (1), vph	35	65			300	0		
NE (2), vph								
E (3), vph	0	600			149	276		
SE (4), vph								
S (5), vph	0	0			0	100		
SW (6), vph								
W (7), vph	70	0			0	0		
NW (8), vph								
Entry Volume, vph	105	665	0	0	449	376	0	0

The highest entry lane volume of the two approach lanes is the critical lane volume (see spreadsheet below). This critical lane volume is adjusted by the heavy vehicle factor and PHF (next spreadsheet) to attain the critical lane entry flow (see Results Table). Heavy Vehicle Factor depends upon passenger car equivalents for trucks and the truck percentage.

Exhibit 7-8 Critical Lane Volumes

Critical Lane Volumes	N	NE	E	SE	S	SW	W	NW
N (1), vph	0	0	25	0	65	0	300	0
NE (2), vph	0	0	0	0	0	0	0	0
E (3), vph	0	0	0	0	600	0	149	0
SE (4), vph	0	0	0	0	0	0	0	0
S (5), vph	19	0	0	0	0	0	0	0
SW (6), vph	0	0	0	0	0	0	0	0
W (7), vph	65	0	153	0	0	0	0	0
NW (8), vph	0	0	0	0	0	0	0	0
Entry Volume, vph	84	0	178	0	665	0	449	0

Heavy Vehicle Factors and Passenger Cars Units
Truck percentage (see spreadsheet below) is kept the same for both lanes. One should have classifier counts for each approach lane or local data of similar existing roundabouts in the site area before altering the multilane spreadsheet and entering a truck percentage for each lane.

There is no change in PHF attained from the count. ET also remained the same as in the single lane roundabout. With the PHF, ET and truck percentage, the heavy vehicle factor is calculated (see below).

Volume Characteristics with Bypass Lane Volume Removed

Volume Characteristics	N	NE	E	SE	S	SW	W	NW
PHF	0.89	0.92	0.92	0.92	0.92	0.92	0.90	0.92
E_t	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
% Trucks	4.0	0.0	5.0	0.0	4.0	0.0	1.5	0.0
F_{hv}	0.962	1.000	0.952	1.000	0.962	1.000	0.985	1.00

Just as the single lane roundabout, the heavy vehicle factor calculation is:

$$F_{HV} = \frac{1}{1 + P_T(E_T - 1)}$$

where

P_T = portion of trucks in the traffic stream, expressed as a decimal
E_T = passenger-car equivalent for trucks, obtained from Exhibit 20-9

The original volume is then factored by the PHF and the heavy vehicle factor to attain the entry flows (passenger car units), some of which are summed to calculate circulating flows (see below).

Entry/Circulating Flows

Entry/Conflicting Flows		N	NE	E	SE	S	SW	W	NW
Flow to	N (1), pcu/h	0	0	29	0	113	0	338	0
Leg #	NE (2), pcu/h	0	0	0	0	0	0	0	0
	E (3), pcu/h	18	0	0	0	678	0	479	0
	SE (4), pcu/h	0	0	0	0	0	0	0	0
	S (5), pcu/h	64	0	51	0	0	0	113	0
	SW (6), pcu/h	0	0	0	0	0	0	0	0
	W (7), pcu/h	76	0	268	0	79	0	0	0
	NW (8), pcu/h	0	0	0	0	0	0	0	0
	Conflicting flow, pcu/h	399	879	531	1706	835	1064	133	556

The circulating flow is then used in the following equation to attain critical entry capacity.

Capacity

$$C = 1130 \bullet \exp\left(-B \bullet V_c\right)$$

where

C = Entry capacity (passenger cars per hour; pc/h)
V_c = Circulating (conflicting) flow (pc/h)
B = Coefficient, 0.0007 for multilane

The north leg, N, critical entry capacity is:

$$C_{crit_N} = 1130 \bullet \exp(-0.0007 \bullet 399) = 855 \text{ pc/h}$$

With the capacity known and the factored entry volume, the volume to capacity ratio (V/C) of the north leg is:

$$V/C_{legN} = \frac{Volume\ldots(pc/h)}{Capacity\ldots(pc/h)} = \frac{Critical\ldots Entry\ldots Flow}{Critical\ldots Entry\ldots Capacity} = \frac{98}{855} = 0.12$$

Control Delay
The average control delay equation is:

$$d = \frac{3600}{C} + 900T\left[\frac{V}{C} - 1 + \sqrt{\left(\frac{V}{C} - 1\right)^2 + \frac{\left(\frac{3600}{C}\right)\frac{V}{C}}{450T}}\right]$$

where

d = Average control delay (s/veh)
C = Capacity of subject lane (veh/h)
T = Time period:T = 1 for actual one hour analysis,T = .025 if using PHF
V = Flow in subject lane (veh/h)

$$d_N = \frac{3600}{855} + 900(0.25)\left[\frac{98}{855} - 1 + \sqrt{\left(\frac{98}{855} - 1\right)^2 + \frac{\left(\frac{3600}{855}\right)\frac{98}{855}}{450(0.25)}}\right] = 4.8 \text{ s/veh}$$

These equations are used in comprising the results table (below). The critical lane entry flow is the critical lane volume factored by the heavy vehicle factor and the PHF. The queue calculation is simply a lookup on a volume table, based on the two minute rule, with car lengths factored by the truck percentage.

Results Table

Results	N	NE	E	SE	S	SW	W	NW
Crit. Entry Capacity pcu/h	855	NA	779	NA	630	NA	1029	NA
Crit. Lane Entry Flow pcu/h	98	0	203	0	797	0	506	0
Leg v/c ratio	0.12		0.26		1.27		0.49	
Control Delay s/pcu	4.8		6.2		148.0		6.8	
LOS	A		A		F		A	
95th Percentile Queue ft	138	0	296	0	1156	0	703	0

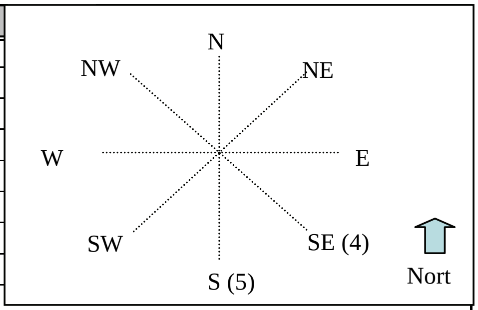
Be sure to follow the flow chart and adhere to the siting criteria. The high v/c ratio of the south leg, with a high right turn movement, was alleviated with the use of a bypass lane (Example 7-4). The high v/c ratio of the west leg required a multilane roundabout due to the large volume of through and left turn movements (Example 7-5). Example 7-5 shows an improvement of the south leg v/c from 1.70 to 1.27, which is still not acceptable.

Example 7-6 Multi-Lane Roundabout with Bypass Lane Calculation

Since the south leg v/c of Example 7-5 is still unacceptably high, Example 7-6 will analyze a south leg bypass. The spreadsheet results are shown below for a south leg bypass on a multilane roundabout, taking the large right turn volume out of the leg.

Multilane Spreadsheet with a South Leg Bypass

General & Site Information	
Analyst:	Count Carsin
Agency/Company:	State Highway
Date:	1/9/2008
Project Name:	City desires roundabout
Intersection:	Mill and Elm Street
Analysis Time Period:	4 PM to 5 PM (with PHF)
Jurisdiction:	City / ODOT
Year:	20 years beyond Build



Volumes	N1 (1)	N2 (1)	NE1 (2)	NE2 (2)	E1 (3)	E2 (3)	SE1 (4)	SE2 (4)
	Roundabout Approach/Entry Legs							
Volumes N (1), vph	0	0			0	25		
to Leg # NE (2), vph								
E (3), vph	0	15			0	0		
SE (4), vph								
S (5), vph	19	36			45	0		
SW (6), vph								
W (7), vph	65	0			82	153		
NW (8), vph								
Entry Volume, vph	84	51	0	0	127	178	0	0
	S1 (5)	**S2 (5)**	**SW1 (6)**	**SW2 (6)**	**W1 (7)**	**W2 (7)**	**NW1 (8)**	**NW2 (8)**
N (1), vph	35	65			300	0		
NE (2), vph								
E (3), vph	0	0			149	276		
SE (4), vph								
S (5), vph	0	0			0	100		
SW (6), vph								
W (7), vph	70	0			0	0		
NW (8), vph								
Entry Volume, vph	105	65	0	0	449	376	0	0

Critical Lane Volumes	N	NE	E	SE	S	SW	W	NW
N (1), vph	0	0	25	0	35	0	300	0
NE (2), vph	0	0	0	0	0	0	0	0
E (3), vph	0	0	0	0	0	0	149	0
SE (4), vph	0	0	0	0	0	0	0	0
S (5), vph	19	0	0	0	0	0	0	0
SW (6), vph	0	0	0	0	0	0	0	0
W (7), vph	65	0	153	0	70	0	0	0
NW (8), vph	0	0	0	0	0	0	0	0
Entry Volume, vph	84	0	178	0	105	0	449	0

Volume Characteristics	N	NE	E	SE	S	SW	W	NW
PHF	0.89	0.92	0.92	0.92	0.92	0.92	0.90	0.92
E_t	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
% Trucks	4.0	0.0	5.0	0.0	4.0	0.0	1.5	0.0
F_{hv}	0.962	1.000	0.952	1.000	0.962	1.000	0.985	1.000

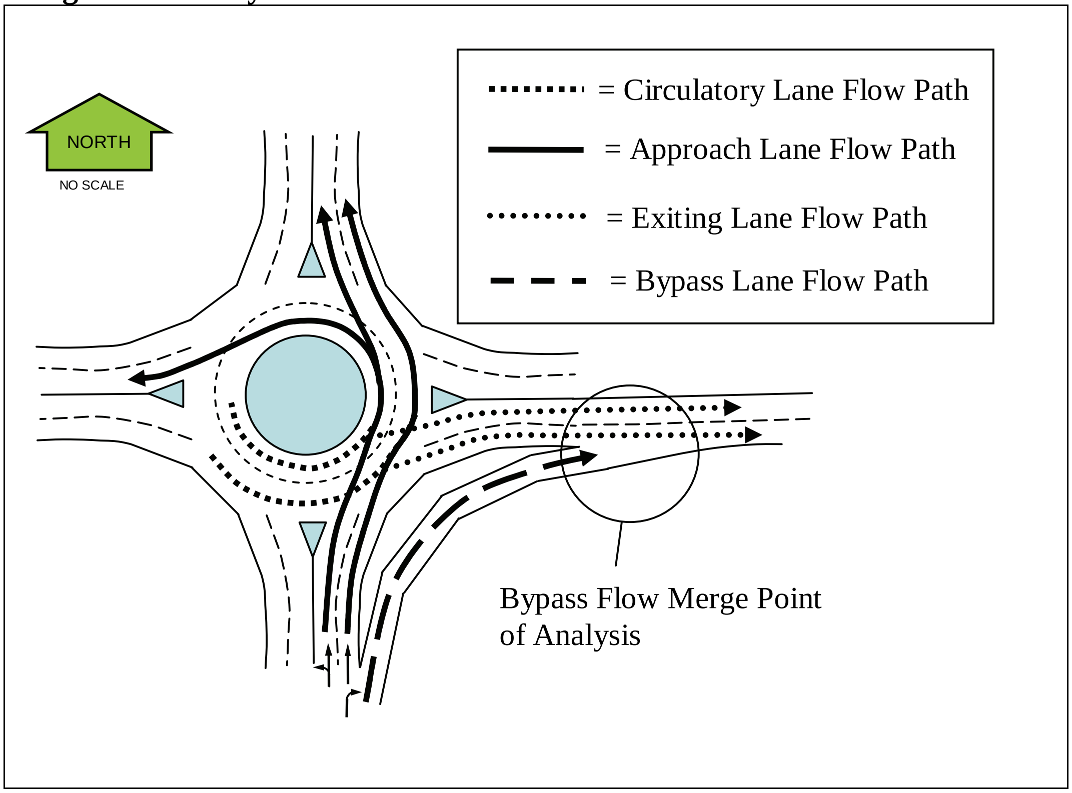
Volume Characteristics	N	NE	E	SE	S	SW	W	NW
PHF	0.89	0.92	0.92	0.92	0.92	0.92	0.90	0.92
E_t	2.0	2.0	2.0	2.0	2.0	2.0	2.0	2.0
% Trucks	4.0	0.0	5.0	0.0	4.0	0.0	1.5	0.0
F_{hv}	0.962	1.000	0.952	1.000	0.962	1.000	0.985	1.000
Entry/Conflicting Flows	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
Flow to N (1), pcu/h	0	0	29	0	113	0	338	0
Leg # NE (2), pcu/h	0	0	0	0	0	0	0	0
E (3), pcu/h	18	0	0	0	0	0	479	0
SE (4), pcu/h	0	0	0	0	0	0	0	0
S (5), pcu/h	64	0	51	0	0	0	113	0
SW (6), pcu/h	0	0	0	0	0	0	0	0
W (7), pcu/h	76	0	268	0	79	0	0	0
NW (8), pcu/h	0	0	0	0	0	0	0	0
Conflicting flow, pcu/h	399	879	531	1027	835	1064	133	556
Results	**N**	**NE**	**E**	**SE**	**S**	**SW**	**W**	**NW**
Crit. Entry Capacity	855	NA	779	NA	630	NA	1029	NA
Crit. Lane Entry Flow	98	0	203	0	119	0	506	0
Leg v/c ratio	0.12		0.26		0.19		0.49	
Control Delay	4.8		6.2		7.0		6.8	
LOS	A		A		A		A	
95th Percentile Queue ft	138	0	296	0	172	0	703	0

The LOS and v/c ratios are now acceptable. If the bypass lane emptied into an add lane and all results were acceptable, then the analysis would be complete. However, it is desired that the bypass lane merge into the lane that exits the roundabout. The bypass lane flow merging into the exit leg is now analyzed, see figure below for location. The capacity of the East exit leg to accept the bypass volume must now be calculated.

The bypass volume is analyzed as a leg of a single leg approach to a roundabout. The bypass flow must yield to the exiting flow, just as the approach flow of a roundabout must yield to the circulatory flow.

The spreadsheet below calculates the capacity of the East leg to accept the bypass volume. The bypass lane v/c ratio is unacceptable at 0.97. The LOS is unacceptable at E (see spreadsheet below), with a queue of 700 feet. Such a queue length will have an adverse affect on the roundabout and the surrounding system. If a roundabout is still desired as a potential alternative, then an add lane must be built alongside the two east leg exiting lanes.

Merge Point Analysis Location

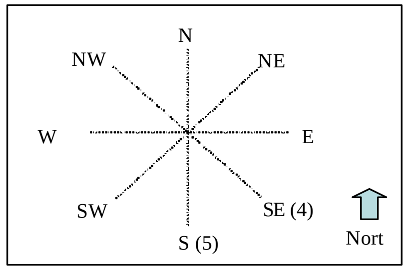


Bypass Merge Point Analysis without Add Lane Calculation

Heavy right still causes high v/c (Example 7-6)

Bypass Lane Merge Point Analysis
of dual exit lanes and a single bypass
lane

Bypass from Leg: S (5)
to the leg adjacent in the counterclockwise direction



Volumes	Circulatory Exit leg Flow (inner)	Circulatory Exit leg Flow (outer)	Approach Bypass Lane Flow	
Sum of E (exiting E inner)	440			Total inner Circulatory Flow (all 8 2cells)
Sum of E (exiting E outer)	600			Total outer Circulatory Flow (all 8 1cells)
Critical exiting Lane Volume	600	greater volume		but do not remove flow from roundabout
S to E right turn volume		Removed roundabout volume	600	Remove this from roundabout volume
Volume Characteristics	**Exit leg**		**Bypass**	
% Trucks	2.0	2.0	7.0	
E_t	2.0	2.0	2.0	
PHF	0.92	0.92	0.92	
F_{HV}	0.980		0.935	
Entry/Conflicting Flows				
Entry Flow			698	
Conflicting Flow	665			
Results				
Entry Capacity, pc/h	709			
Bypass Lane v/c ratio	0.98			
Control Delay, s/pc	49.0			
LOS	E			
95th Percentile Queue (ft)	955			

7.3.7 Signalized Intersection Analysis

Signalized intersection control can generally be classified into three categories; pre-timed, semi-actuated and fully-actuated operations. A pre-timed signal has the cycle length, phases, green times and change phases all preset to be constant for every cycle. A semi-actuated signal operates by designating a “main street” that is served until actuation from the “side street” occurs. Under this type of operation the cycle length and green times may vary based on vehicle demand. ODOT has effectively upgraded all formerly semi-actuated intersections to fully actuated. A fully-actuated signal allows detection on all legs and phases of the intersection and cycle lengths and green times are determined based on the demand for each movement.

In addition to the type of signal operating, each signalized intersection has characteristics associated with it related to how the timing of a signal is allocated over a cycle. These characteristics relate to phases, intervals, change intervals, green time, lost time, yellow and all-red clearance times and effective green time. All of these characteristics can be part of signalized operations and can affect the overall intersection operations. For more information on characteristics of signals and signal operations analysis refer Chapter 16 of the HCM.

Saturation Flow Rates
As previously discussed in Chapters 3-5, saturation flow rates are critical components in the analysis of signalized intersection capacity and can be defined as the flow in vehicles per hour that can be accommodated by a lane group assuming that the green phase is displayed 100 percent of the time. Saturation flow rates can be measured in the field or calculated by applying adjustment factors to a default “ideal” saturation flow rate. For more information regarding the calculation and application of saturation flow rates, refer to Chapter 3.

Signalized Intersection v/c Ratio
For signalized intersections, the OHP v/c ratio is based on the overall intersection v/c ratio, not the movement v/c ratio as explained in Action 1F of the OHP. The intersection v/c ratio is also known as the critical v/c ratio or Xc in the HCM. The intersection v/c ratio is not generally affected by the approach green times (except in cases with shared left turns). See HCM equation 16-8 below.

$$Xc = \sum \left(\frac{v}{s}\right)_{ci} \left(\frac{C}{C-L}\right)$$

where:

Xc = critical v/c ratio for intersection
$\sum \left(\frac{v}{s}\right)_{ci}$ = summation of flow ratios for all critical lane groups i
C = cycle length(s)
L = total lost time per cycle, computed as lost time, tL, for critical path of movement(s)

Analysis Procedures Regarding Signal Timing
Capacity analysis of signalized intersections should be performed in accordance with the methods and default parameters contained in this manual. ODOT has established the following criteria for traffic impact studies with regard to the timing chosen for the capacity analysis of signalized intersections. ODOT reserves the right to reject any operational improvements that in its judgment would compromise the safety and efficiency of the facility.

Phase Splits
Thirteen seconds is the lowest maximum green split that should be used. Clear documentation of the selected maximum splits for each phase must be provided in the analysis. The total side street splits should not be greater than the highway splits. Except in cases where the analyst is directed otherwise by ODOT staff, the splits are considered optimized when they yield the lowest overall intersection v/c ratio. This optimization should be done for each capacity analysis.

Non-Coordinated Signals
Cycle lengths and phase splits should be optimized to meet an ideal level of service, queuing and/or volume to capacity ratio for a non-coordinated traffic signal intersection. If simulation is going to be needed, existing signal timing will be necessary for the calibration process. Otherwise, unless directed to do so by ODOT staff, the use of the existing timing is not required. The cycle length for the analysis should not exceed 60 seconds for a two-phased traffic signal, 90 seconds for a three-phased traffic signal (e.g., protected highway left turns and permissive side streets left turns) or 120 seconds for a four or more phased traffic signal. The signal cycle length should cover the pedestrian clearance time for all crosswalks. For information on pedestrian crossings, see ODOT Traffic Signal Policy and Guidelines.

Signals in Coordinated Signal System
At the start of a project, ODOT staff will determine whether the analysts should use the existing signal timings for all analysis scenarios or develop optimized timings for the coordinated system. The existing timings may need to be used to calibrate a simulation model. If the existing timings are to be used in the analysis, Region traffic shall provide timing files, timing sheets or Synchro files of the existing settings. If optimized timings are to be developed, those settings are subject to approval by ODOT and those conditions become the baseline for all comparisons.

The following settings should be optimized for each analysis scenario when the analyst is asked to use optimum coordination settings.

- Cycle Length
- Phase Length (Splits)
- Phase Sequence (Lead/Lag Left Turns)
- Intersection Offsets

The optimum settings must meet the criteria established in OAR 734-020-0480 as it relates to progression analysis while also attempting to find the lowest v/c ratio for each intersection. This OAR only applies when modifications are proposed to a signal which would affect the settings of the coordination plans. Examples of these modifications are changes in cycle length, decreased green time for mainline, additional phases, longer crosswalks and intersection relocation. Note: If Synchro is to be used to optimize a series of coordinated intersections review Section 7.3.8 and ensure that all necessary data is entered. If SimTraffic will be eventually used, ensure that Section 7.3.8 and Chapter 8 is followed.

Future Signals
For future signals, left turns should be assumed to have the appropriate phasing (i.e., permitted, protected-permitted or protected only) according to the criteria for left turn treatment contained in the current ODOT Traffic Signal Policy and Guidelines. The Region Traffic Section and the Traffic-Roadway Section should be consulted any time a new signal is proposed. It should always be considered that while new traffic signals provide a benefit to some users, the capacity of the mainline is typically cut in half by new signal installations and improper or unjustified signals can increase the frequency of rear-end collisions, delays, disobedience of signal indications and the use of less adequate routes.

Signal Timing Sheets

If it is desired to closely match the current traffic operations, the timing parameters installed in the signal controller need to be used in the analysis. The field timing parameters are recorded on the signal timing sheets located in the signal cabinet. Signal timing sheets should be obtained from the Region Traffic office as they generally have the most recent copies from the signal cabinet. Signal timing changes frequently, so the analyst should make sure to have the most recent version. For the analyst, not all of the included sheets are necessary, but it is important that all of the needed sheets are obtained. The following shows the important sheets (Exhibit 7-9 through Exhibit 7-15, Sheets 2, 3, 6, 7 and 8. Sheets 4 and 5 are required if multiple timing plans exist) and what to look for on each sheet. The example signal timing sheet used to illustrate this section is the intersection of US 97 (Bend Parkway) and Pinebrook Boulevard in Bend.

Sheet 2 – Phase Rotation Diagram

The phase rotation diagram shows how the signal operates through its cycle. This diagram is needed so the signal is entered correctly into Synchro or other program. For complicated phasing the diagram is an invaluable source. Exhibit 7-9 shows a phase rotation diagram for US 97 and Pinebrook Boulevard, which is a two-phase signal. Many timing sheets, especially the electronic ones, are missing the phase rotation diagram. Contact the appropriate Region Traffic section to obtain.

Exhibit 7-9 Signal Timing Sheet 2

SHEET 2

Date sheet in effect: Date sheet voided:

Location: Hwy 97 @ Pinebrook

Table Numbers refer to Trafficview & Translink

TABLE 3

Clock, EV and Misc. (C + Key)									
Function	Key	Clock Location C + 3 uses Call / Active Display Sunday = 1							
Year	0								
Month	1								
Date	2								
Day of Week	3								
Hour	4								
Minute	5								
Second	6								
1/10 Second	7								
		Phase Number							
		1	2	3	4	5	6	7	8
	8								
EVD Phases	D			X					X
Handicap Ped	E								

TABLE 3

Preemption Data (E+Key)			
Function	Key	Parameter	Timing
EVA	0	Delay	0
	1	Minimum	1
EVB	2	Delay	0
	3	Minimum	1
EVC	4	Delay	0
	5	Minimum	1
EVD	6	Delay	0
	7	Minimum	1
Overlaps	8	Red Revert	5.0
Railroad	9	Delay	
	A	Minimum	

		Phase Number							
		1	2	3	4	5	6	7	8
RR Clear Ph	B								
RR Permit	C								
RR OL Permit	D								
Nema Hold Ph	E								
	F								

TABLE 6 (also see sheet 6)

Miscellaneous (D + Code)			
Function	Code	Value	Notes
Floating Ped	2E		0 = Off 1 = On (Ph. 7 & 8 Not permitted)
ID Number	2F	061	Range 0 to 253 (1)
Coordination Ped Recalls	3E	1	0 = Recall 1 = No Recall
Rest in WALK	3F		0 = Off 1 = ON
Advance Warning End of Green	4E		Extend time for green after sign turns on (2) (5)
Advance Warning	4F		Delay time for sign after ... yellow (2) (5)
			...l red ...d flash
			... 1 = Flash ... Flash Red
NEMA Inputs	66		Non zero value reassigns C1 inputs. (3)
Bus Delay	6D		Delay time before preemption (4)
Bus Timer # 1	6E		Extension of max green for phases 2 & 6 (Free operation)
Bus Timer # 3	6F		Force off time for Ph 4 & 8 (only Free operation)
JHK Protocol	76		0 = no 0.1 = yes
JHK Area No. & 1st digit local	7D		Area No. 0 - 7 and Local 001 - 510 (1)
EV minimum timed Start / end of call	7E		0 = at start of call 1 = at end of call
EV On Indicators	7F		0 = Off, 1 = Flash, 5 = solid indication (5)

Notes

(1) JHK ID no. is formed by Area no. (0 to 7) and 3 digit Local no. (001-510). Left most digits entered as x.x in location 7D and rightmost as xx in location 2F

(2) See Sheet 6, Location B+0+E

(3) C1 pins 54, 63, 64, 75, 76, and 77. See sheet 6, Location B+0+D.

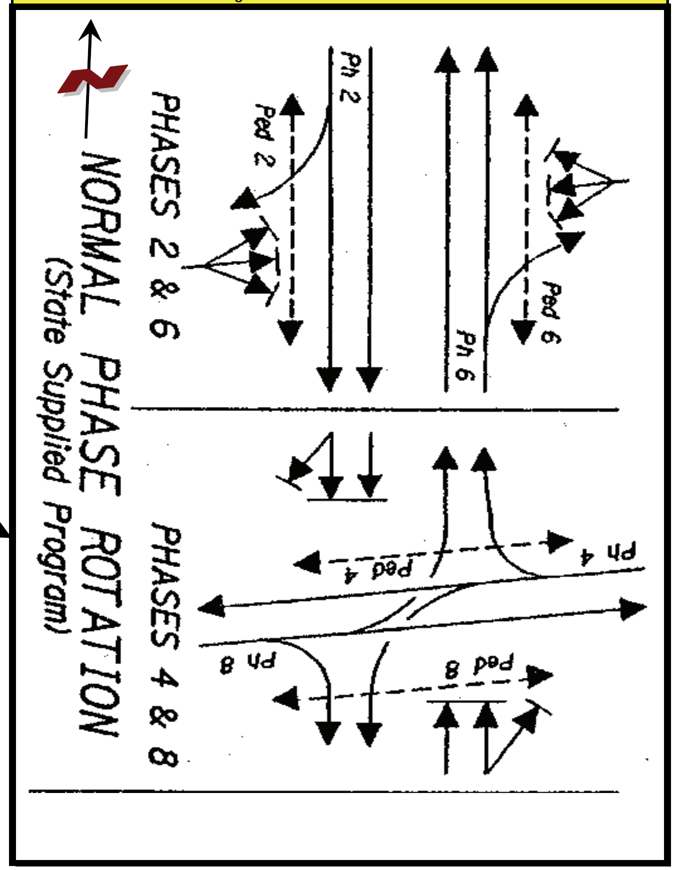
(4) Entering 25.5 in this location is the only way of disabling bus preempt.

(5) Ped yellow outputs, C1-35, 36, 37, and 38 are used by Rt. Turn Overlaps, EV on indicators, TOD/DOW programmable outputs, Fiber Optic sign for RR flash yellow clearance, and Advance Warning sign operation.

Phase Rotation Diagram

Phase Rotation Diagram

T. M. S. Dwg. Nos:



Sheet 3 – Table 1 Phase Functions

Table 1 (Exhibit 7-10) shows the basic phasing properties and Exhibit 7-11 shows the pedestrian timings and the advanced actuated phasing properties needed for signalized analysis and simulation programs. Vehicle Recall (Key =0) shows what phases will appear for at least a minimum amount of time in each cycle the signal would return to if there is no demand on the side street. Permitted Phase (Key=4) shows what phases are present at this intersection. Overlap A-D (Key A-D) shows what phases operate together on each of the overlap outputs on the controller. If there are no checked boxes in this section, then there are no overlapping phases, but there may be signal heads displaying outputs from two phases such as the common vertical five-section right-turn signal head.

Sheet 3 – Table 1 Phase Timing

For non-coordinated signals, the cycle length and phase splits can be determined from the Phase Timing portion of Table 1. If multiple timing plans exist then they will be listed on Sheet 4 and/or Sheet 5. The only values that are needed to determine splits and cycle lengths from this portion of Table 1 are the maximum greens (Key = ph + 0), max 2 greens (Key = ph +1), yellow time (Key = ph + C) and all-red time or red clear (Key = ph + D).

The cycle length of actuated signals will vary from cycle to cycle depending on the vehicle demand. Synchro's phase splits include yellow and all-red, which is different from the maximum green on the timing sheet. Synchro also forces the maximum greens to add up perfectly to the cycle length. Therefore, the maximum cycle length needs to be proportionally adjusted down to match with Synchro's cycle length (the cycle length that is entered into the program). The maximum cycle length can be determined by summing the maximum greens (or max 2 greens if those are used in the analysis hour) and the yellow/all-red for each phase. The max green values on Sheet 3 are just that, i.e., maximum green times. The total maximum split used in Synchro will be the sum of the max green (or max 2 green), yellow and all-red. To convert the Sheet 3 timing into Synchro-compatible timing, the following is done.

1. Add up the Synchro cycle lengths from Sheet 3 by summing the maximum greens.
2. Add the yellow time and all-red time to the cycle length calculated in Step 1 to obtain the maximum cycle length.
3. The Synchro phase lengths are calculated by dividing the green + yellow + all-red time for a phase by the maximum cycle length. This ratio is then multiplied by the Step 1 Synchro cycle length.
4. Repeat for each phase.

The sum of the Synchro phases should add up to the Step 1 cycle length.

Exhibit 7-10 Signal Timing Sheet 3 – Basic Phase Settings

Hwy 97 @ Pinebrook

Vehicle Recall, Permitted Phases & Overlaps

TABLE 1 Page 0

Phase Functions (0+Key)

Function	Key	1	2	3	4	5	6	7	8
		Phase Number *							
Veh Recall	0		X				X		
Ped Recall	1								
Red Lock	2								
Yellow Lock	3								
Permit Phase	4		X		X		X		X
Ped Phases	5		X		X		X		X
Lead Phases	6	X		X		X		X	
Double Entry	7				X				X
Sequential	8								
Start Green	9		X				X		
OLA=	A								
OLB=	B								
OLC=	C								
OLD=	D								
Exclusive	E								
Sim Gap	F		X				X		

TABLE 2 Page 0

Miscellaneous (C+F+Key)

Function	Key	Value	
Page ID	0	0	Keys 8 through F use Call/Active Display
	1		
	2		
	3		
OLA Red	4		
OLB Red	5		
OLC Red	6		
OLD Red	7		

Function	Key	1	2	3	4	5	6	7	8
		Phase Number							
RT OLE	8								
RT OLF	9								
Red Rest	A								
Max Recall	B								
Flash Green	C								
	D								
Advance WALK	E								
Restrictive Ph	F								

* Shown on Call/Active Display

TABLE 1 Page 0

Phase Timing (Ph. No. + Key)

Interval	Key	1	2 Southbound Hwy 97	3	4 Westbound Pinebrook	5	6 Northbound Hwy 97	7	8 Eastbound Pinebrook
		Phase Number							
Max Green	0		50		30		50		30
Max2 / HFDW	1		40		35		40		35
Walk	2		5		5		5		5
Flashing DW	3		21		21		22		25
Max Initial	4		20		5		20		[illegible]
Min Green	5		10		5		10		
TBR	6		10		5		10		
TTR	7		20		5		20		
Observe Gap	8								
Passage	9		5.2		3.5		5.2		
Min Gap	A		3.2		1.0		3.2		
Add per Act	B		1.5				1.5		
Yellow	C		4.0		4.0		4.0		4.0
Red Clear	D		1.0				1.0		
Red Revert	E		5.0		5.0		5.0		5.0
Walk 2	F								

To observe timing for an individual phase:
Enter C + A + F for Ring A (Phase 1-4) or enter C + B + F for Ring B (Phase 5-8)

Page I.D. 0

Phase Conditions as shown on Free Display

00 Initial Entry
02 WALK
03 Flashing DW
05 Min Green
08 Rest
09 Passage
0B Added Initial
0C Yello
0D Red
0E Red
11 Gap Out
12 Force Off
14 Max Out
15 Red Revert Timed out

TABLE 2 Page 0

Miscellaneous (9+Key)

Parameter		Key	Value	Notes
Short Pwr Dn		0		Clock Correction Speed up 1 - 9 Slow down 11 - 19
Long Power Dn		1		
Preemption Delay Types	EVA	2		Preemption Delay Types: Hold 1 Latch 2 Both 3 Neither 0
	EVB	3		
	EVC	4		
	EVD	5		
	RR	6		
OLD	Green	E		
	Yellow	F		

Maximum Green and Max 2 Green Times

Sheet 8 indicates when each is in effect.

Keyboard Entries when not in Free Display

A Advance
B Back
C Clear Display
D Column Advance
E Enter and Advance
F Free Display

Yellow and All-red Time

Phase Data Copy
C + x + C + y + D
x From Phase (x cannot be 3 or 8)
y To Phase(s) - up to 3

SHEET 3

Exhibit 7-11 Signal Timing Sheet 3 - Advanced Phase Settings

Date sheet in effect:

Date sheet voided:

Location: Hwy 97 @ Pinebrook

TABLE 1 Page 0

Phase Functions (0+Key)

Function	Key	1	2	3	4	5	6	7	8
		Phase Number *							
Veh Recall	0		X				X		
Ped Recall	1								
Red Lock	2								
Yellow Lock	3								
Permit Phase	4		X		X		X		X
Ped Phases	5		X		X		X		X
Lead Phases	6	X		X		X		X	
Double Entry	7				X				X
Sequential	8								
Start Green	9		X				X		
OLA=	A								
OLB=	B								
OLC=	C								
OLD=	D								
Exclusive	E								
Sim Gap	F		X				X		

TABLE 2 Page 0

Miscellaneous (C+F+Key)

Function	Key	Value
Page ID	0	0
	1	
	2	

Keys 8 through

OL

OL

OL

OL

Actuated Phasing Settings for Timing Plans and Simulation

	Key	1	2	3	4	5	6	7	8
RT OLE	8								
RT OLF	9								
Red Rest	A								
Max Recall	B								
Flash Green	C								
	D								
Advance WALK	E								
Restrictive Ph	F								

* Shown on Call/Active Display

SHEET 3

TABLE 1 Page 0

Phase Timing (Ph. No. + Key)

Interval	Key	1	2 (Southbound Hwy 97)	3	4 (Westbound Pinebrook)	5	6 (Northbound Hwy 97)	7	8 (Eastbound)
		Phase Number							
Max Green	0		50		30		50		30
Max2 / HFDW	1		40		35		40		35
Walk	2		5		5		5		5
Flashing DW	3		21		21		22		25
Max Initial	4		20		5		20		5
Min Green	5		10		5		10		5
TBR	6		10		5		10		5
TTR	7		20		5		20		5
Observe Gap	8								
Passage	9		5.2		3.5		5.2		3.5
Min Gap	A		3.2		1.0		3.2		1.0
Add per Act	B		1.5				1.5		
	C		4.0		4.0		4.0		4.0
	D		1.0				1.0		
	E		5.0		5.0		5.0		5.0
	F								

Walk and Flashing Don't Walk times

To observe timing for an individual phase :
Enter C + A + F for Ring A (Phase 1-4) or enter C + B + F for Ring B (Phase 5-8)

Page I.D. 0

Phase Conditions as shown on Free Display

00	Initial Entry	0C	Yellow
02	WALK	0D	Red Clear
03	Flashing DW	0E	Red Revert
05	Min Green	11	Gap Out
08	Rest	12	Force Off
09	Passage	14	Max Out
0B	Added Initial	15	Red Revert Timed out

TABLE 2 Page 0

		Key	Value	
Short Pwr Dn		0		Clock Correction Speed up 1 - 9 Slow down 11 - 19
Long Power Dn		1		
Preemption Delay Types	EVA	2		Preemption Delay Types:
	EVB	3		
	EVC	4		Hold 1 Latch 2 Both 3 Neither 0
	EVD	5		
	RR	6		
Ped Inhibit		7		Usually should be 0
OLA	Green	8		Overlap Yellow Time should always be specified
	Yellow	9		
OLB	Green	A		
	Yellow	B		
OLC	Green	C		
	Yellow	D		
OLD	Green	E		
	Yellow	F		

Keyboard Entries when not in Free Display

A	Advance	D	Column Advance
B	Back	E	Enter and Advance
C	Clear Display	F	Free Display

Reinitialization
D + 1 + F + 1 + E
(Use **only** when in flash)

Phase Data Copy
C + x + C + y + D
x From Phase (x cannot be 3 or 8)
y To Phase(s) - up to 3

Example 7-7 Signal Phase Splits

Example values for Sheet 3 are (Exhibit 7-10):

- Vehicle Recall = Phases 2 and 6 (US 97)
- Permitted Phases = 2, 4, 6 and 8. From the phase rotation diagram in Exhibit 7-9 it is seen that Phase 2 and 6 on US 97 go together and Phase 4 and 8 on Pinebrook go together.
- Overlaps = No overlapping phases

If this signal was not coordinated (it isn't) then the maximum cycle length would be the maximum greens plus the yellow times plus the all-red times. In checking Sheet 8 (Exhibit 7-15), it is found that the max 2 green time is in effect starting at 4:30 PM, so the max 2 green time will be used to calculate the cycle length.

Maximum Cycle length = Max 2 green for Phase 2 and 6 + Max 2 green for Phase 4 and 8 + yellow x 2 phases + all-red x 1 phase = 40 + 35 + (4 x 2) + 1= 84 seconds.

Synchro phase split conversion:

1. Synchro Cycle length = 40 + 35 = 75 s
2. Maximum cycle length = 75 + 4(2) +1 = 84 s
3. Synchro Phase 2&6 = ((40 + 4 + 1) / 84) x 75 = 40 s
4. Synchro Phase 4&8 = ((35 + 4) / 84) x 75 = 35 s
5. Check = 40 +35 = 75 s = Step 1 cycle length

In the above example the differences in the phase splits are small, resulting in Synchro splits that are the same as the timing sheet splits. The splits are different if the maximum greens were used instead of the max 2 greens, as shown below.

1. Synchro Cycle length = 50 + 30 = 80 s
2. Maximum cycle length = 80 + 4(2) +1 = 89 s
3. Synchro Phase 2&6 = ((50 + 4 + 1) / 89) x 80 = 49 s
4. Synchro Phase 4&8 = ((30 + 4) / 89) x 80 = 31 s
5. Check = 49 +31 = 80 s = Step 1 cycle length

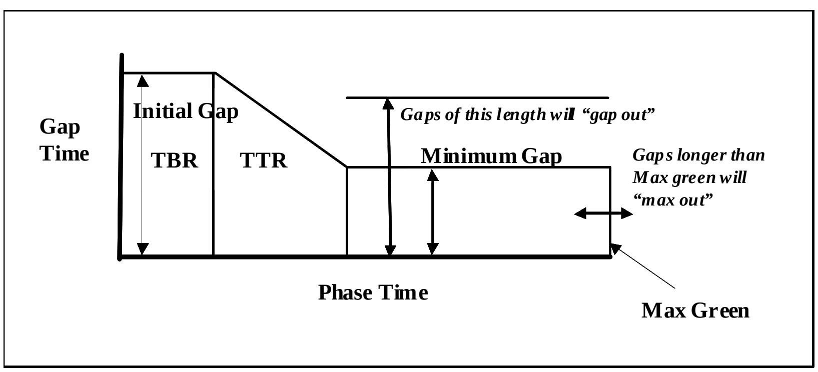
For most new actuated signals, additional settings need to be pulled from Table 1. Pedestrian settings can have a large impact on signal operation and the resulting intersection v/c especially if there are a large number of pedestrian calls per hour on an approach. For creating a calibrated simulation, the actual pedestrian timing should be used as shown in Table 1 (Key= ph + 2 and Key = ph + 3) If the timing is not known, the ODOT standard walk time is 7.0 seconds with the curb-to-curb flashing don't walk time based on a 4.0 ft/s walk time.

Table 1 also covers the actuated signal phasing parameters that are needed for creating timing plans and calibrated simulations. These five parameters are:

- **Minimum Green** (Key= ph + 5) - Minimum green time that a signal indication will occur for once the phase is served..
- **Time Before Reduce** (TBR) (Key= ph + 6) – Time elapsed before gap time is reduced
- **Time To Reduce** (TTR)(Key = ph + 7) - Time elapsed during gap time reduction to minimum.
- **Passage** (Key = ph +9) – This is the time that a phase is initially extended after a call is placed on a vehicle approach. Also known as initial gap.
- **Minimum Gap** (Key = ph + A) – Gap time after reduction until end of phase.

Exhibit 7-12 shows the progression of the gap time from when a green indication starts at the initial gap in the TBR period down to the minimum gap time. During the TTR period, the initial gap time is reduced down to the minimum gap time as specified on the timing sheet. If during the minimum gap time, the minimum gap is exceeded, then the signal will turn yellow (also known as a “gap out”). If vehicles keep approaching, the passage time will extend the green time to the maximum green time and then turn yellow (also known as a “max out”). Having a signal gap out is preferable, as dilemma vehicles (vehicles that either quickly accelerate or decelerate under yellow) can occur under max out conditions.

Exhibit 7-12 Actuated Gap Time



Sheet 6 – Table 6 Operation

Table 6 indicates whether or not the signal is ever coordinated over the course of a day or week. If Mode (Key = B+0+4) is a non-zero value, then the intersection is coordinated. The intersection may or may not be in coordination during the analysis periods. The actual times that coordination plans are in effect are entered on Sheet 8 of the local controller or on Table 5 of the On-Street Master Controller. Exhibit 7-13 shows that the example intersection is coordinated, but is not the master.

Exhibit 7-13 Signal Timing Sheet 6

SHEET 6

Date sheet in effect:

Date sheet voided:

Location: Hwy 97 @ Pinebrook

TABLE 6 (Also see sheet 2)

Operation (B + 0 + Key)

Key	Parameter	Value
0	Present Plan	
1	Time of Day Plan	
2	Hardwire Plan	
3	MODEM Plan	
4	Mode (0 - 4 see right)	3
5	Master (0 - 4 see right)	0
6	Master Cycle Clock	
7	Local Cycle Clock	
8	Local Timer	
9		
A		
B		

Mode: 0 = Free; 1 = TBC; 2 = Hardwire; 3 = Modem; 4 = TM System

Master: 0 = Off; 1 = Modem Master; 2 = Hardwire Master; 3 = 1 + 2; 4 = TM Master

Key		Phase Number 1	2	3	4	5	6	7	8
C									
D	NEMA CNA								
E	Adv. Warn.								
F	MRI Phases		X		X		X		X

OSM ? Y [ ] N [X]

OSM Location

Powers Rd.

Coordination Mode and Master Type

Function Code Index

Function		Time Clock On	Time Clock Off	Manual On	Manual Off	Notes
Outputs	A	71	81			*Phase 2 ped yellow (C1-35) (1)*
Outputs	B	72	82			*Phase 6 ped yellow (C1-36) (1)*
Outputs	C	73	83			*Phase 4 ped yellow (C1-37) (1)*
Outputs	D	74	84			*Phase 8 ped yellow (C1-38) (1)*
TOD Red Rest		25	24			*See Sheet 10 at B + C + D to set phases*
TOD Max Recall		27	26			*See Sheet 10 at B + A + E to set phases*
TOD Ped Recall		29	28			*See Sheet 10 at B + B + E to set phases*
WALK 2		55	54			*Use WALK 2 times set on Sheets 3, 4, 5*
Plan No.		1 - 18		1 - 18	0	*Sets operation to coordination plans on Sheet 7*
Free		20		20		*Sets operation to fully actuated*
Flash		19 or 33	32	19 or 33	0	*Sets operation to flash*
Max 2		129	128	129	0	*Use Max 2 times set on Sheets 3, 4, 5*
Det. Count 15		131	130			*Log Detector Counts - 15 min. intervals*
Det. Count 60		132	130			*Log Detector Counts - 60 min. intervals*
[illegible]		[illegible]				*Clear Detector Count Log*
[illegible]		[illegible]	[illegible]			*Enable Detector Diagnostics and log*
[illegible]		[illegible]	[illegible]			*Enable Detector Diagnostic without log*
Clear Det Diag.		138				*Clear Detector Diagnostic Log*
Send Real Time		199		199		*Modem master only*
Time Transfer		100		100		*Implements Page 0*
Time Transfer		101		101		*Implements Page 1*
Time Transfer		102		102		*Implements Page 2*
Page Copy				93	---	*Copies Page 0 data to Pages 1 & 2 Make sure Page 0 is the active Page*
Burn EEPROM				94	---	*Places active timing data into backup timing (Use reinitialization to place backup into active)*
Print Out				96	0	*Conect printer to C2 connector*

Note

(1) These C1 pins are used for other functions. See note (5) on Sheet 2.

Manual (D + 1 + E) [ ]

TABLE 13 (Also see Sheet 10)

Miscellaneous (E + F + Key)

Function	Key	Time
Railroad Max 2	0	
Ped Permissive Plan 1	1	
Ped Permissive Plan 2	2	
Ped Permissive Plan 3	3	
Ped Permissive Plan 4	4	
Ped Permissive Plan 5	5	
Ped Permissive Plan 6	6	
Ped Permissive Plan 7	7	
Ped Permissive Plan 8	8	
Ped Permissive Plan 9	9	
Number of Long Powerouts	A	
Number of Short Powerouts	B	
Failed Detector Number	C	
Max 2 On	D	
No Daylight Savings	E	
Revision Level	F	

TABLE 10 (Also see Sheet 12)

Sample Detectors (0 = off, 1 = on) (A + 3 + 9) [ ]

Sampling detectors are assigned using extended input codes on Sheet 11

For Protected / Permissive Left Turns

Left Turn Type (0, 1, or 2) (A + 3 + A) [ ]

0 = Off

1 = Left turn places call on cross street

2 = Left turn is omitted until cross street is serviced

Note: This feature works only with leading left turn phases 1, 3, 5, or 7. It is used to prohibit a green arrow from immediately following a green ball.

Sheet 7 – Table 7 Coordination Timing

If a signal operates in coordinated mode, then the timing shows up in Table 7. Timing values such as lead-lag settings on Sheet 7 override the values on Sheet 3. A signal controller will not exceed the max greens from Sheet 3 nor the force-offs (when the phase is forced “off” by the clock) on Sheet 7. The cycle length shown on Sheet 7 can be directly entered into Synchro. Using the force-offs the actual phase splits can be calculated. These values can also be directly entered into Synchro.

Exhibit 7-14 shows Table 7 for the example. In this case, Plan 2 with the 80 second cycle length is in operation during the afternoon peak. Read down the column. At 0 seconds Phases 2 and 6 are forced off. At 35 seconds Phases 4 and 8 are forced “off.” Phases 2 and 6 operate from 35 seconds around to 0 seconds on the clock (80 – 35 = 45 seconds). In this case Phase 2 and 6 are 45 seconds and Phase 4 and 8 are 35 seconds. Note how this is would be different if this intersection was not coordinated, as shown under Sheet 3.

Exhibit 7-14 Signal Timing Sheet 7

Date sheet in effect: Date sheet voided:

Location: Hwy 97 @ Pinebrook

TABLE 7 (1 of 2)

Hardwire Conversion	Dial Offset	1			2			3			
		1	2	3	1	2	3	1	2	3	
Parameter	**Key**	**Coordination Timing (B + Plan No. + Key)**									
		1	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	Plan Number
Cycle Length	0	70	80								
Forceoffs for Phase Indicated by Key No.	1										
	2	0	0								
	3										
	4	31	35								
	5										
	6	0	0								
	7										
	8	31	35								
Offset	9	45	48								
Permissive	A	2	2								
Max. Dwell	B	30	35								

Coordination Timing Plans

Plan #2 is used in the example.

Sheet 8 of the master controller shows when each plan is in effect.

Plan			1	2	3	4	5	6	7	8
1	C	Lead Phases	X		X		X		X	
	D	Coord. Phases		X				X		
	E	Perm. 2 Ph.								
	F	Min. Recall								
2	C	Lead Phases	X		X		X		X	
	D	Coord. Phases		X				X		
	E	Perm. 2 Ph.								
	F	Min. Recall								
3	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
4	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
5	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
6	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
8	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								
9	C	Lead Phases								
	D	Coord. Phases								
	E	Perm. 2 Ph.								
	F	Min. Recall								

TABLE 7 (2 of 2)

Parameter	**Key 2**	**Coordination Timing (B + D + Key 1 + Key 2)**									
		10	11	12	13	14	15	16	17	18	Plan Number
		7	**8**	**9**	**A**	**B**	**C**	**D**	**E**	**F**	**Key 1**
Cycle Length	0										
Forceoffs for Phase Indicated by Key No.	1										
	2										
	3										
	4										
	5										
	6										
	7										
	8										
Offset	9										
Permissive	A										
Max. Dwell	B										

Plan			1	2	3	4	5	6	7	8
10	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
11	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
12	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
13	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
14	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
15	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
16	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
17	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								
18	C	Lead Phases								
	D	Coord. Phases								
	E									
	F	Min. Recall								

SHEET 7

<u>Sheet 8 – Table 5 Time Clock Control</u>

Table 5 shows the times that various timing plans and max greens are in effect for a particular intersection. In the absence of timing sheets from an on-street master controller (noted as "OSM" on the front of the timing sheet), the analyst will have to contact Region Traffic to verify which timing plan on Sheet 7 is in effect during the desired analysis period. Generally, during the PM peak plan #2 is in effect. The master controller would indicate in Table 5 which coordination plan shown on Sheet 7 would be operating at any given time. The function codes in the right-hand column in Table 5 can tell the analyst what maximum green applies. Code 128 is for the maximum green while Code 129 is for the max 2 green. Codes 100, 101 and 102 apply to Page 0, 1, 2 (on Sheets 3, 4 or 5) respectively, so the analyst can determine what phase timing is in effect. Codes 131 and 132 are just to tell the controller to count the traffic volume data in 15-minute intervals or 60-minute intervals, respectively.

Exhibit 7-15 shows the timing plans in effect for the example intersection. The controller for this intersection is coordinated, but is not the master. If this signal was not coordinated, Code 129 would be indicated starting at 4:30 PM, in which case the max 2 green would be used for calculating the cycle length and phase splits.

If this controller was the master controller, an event would be listed showing when each plan went into effect. Event 7 has been added to the table to illustrate this.

Exhibit 7-15 Signal Timing Sheet 8

SHEET 8

Date sheet in effect:

Date sheet voided:

Location: Highway 97 @ Pinebrook

TABLE 5 (1 of 2)

Time Clock Control (A+Code)

Event No.	Day code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
1	80								81	82	83
		X	X	X	X	X	X	X	6	00	131
2	84								85	86	87
		X	X	X	X	X	X	X	8	00	132
3	88								89	8A	8B
		X	X	X	X	X	X	X	14	00	131
4	8C								8D	8E	8F
		X	X	X	X	X	X	X	18	00	132
5	90								91	92	93
			X	X	X	X	X		16	30	129
6	94								95	96	97
			X	X	X	X	X		19	00	128
7	98								99	9A	9B
			X	X	X	X	X		16	31	2
8	9C								9D	9E	9F
9	A0								A1	A2	A3
10	A4								A5	A6	A7
11	A8								A9	AA	AB
12	AC								AD	AE	AF
13	B0								B1	B2	B3
14	B4								B5	B6	B7
15	B8								B9	BA	BB
16	BC								BD	BE	BF

Time Clock Control (A+Code)

Event No.	Day code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
17	C0								C1	C2	C3
18											
19											
20	CC								CD	CE	CF
21	D0								D1	D2	D3
22											
23	D8								D9	DA	DB
24											
25											
26											
27											
28											
29	F0								F1	F2	F3
30	F4								F5	F6	F7
31	F8								F9	FA	FB
32	FC								FD	FE	FF

TABLE 5 (2 of 2)

Time Clock Control (D+8+Code)

Event No.	Day code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
33	80								81	82	83
									85	86	87
									89	8A	8B
36	8C								8D	8E	8F
	90								91	92	93
39	98								99	9A	9B
45	B0								B1	B2	B3
46	B4								B5	B6	B7
47	B8								B9	BA	BB
48	BC								BD	BE	BF

Time Clock Control (D+8+Code)

Event No.	Day code	S 1	M 2	T 3	W 4	T 5	F 6	S 7	Hour	Min.	Func
49	C0								C1	C2	C3
50	C4								C5	C6	C7
51	C8								C9	CA	CB
52	CC								CD	CE	CF
	D0								D1	D2	D3
	D4								D5	D6	D7
55	D8								D9	DA	DB
	DC								DD	DE	DF
	E0								E1	E2	E3
	E4								E5	E6	E7
	E8								E9	EA	EB
	EC								ED	EE	EF
61	F0								F1	F2	F3
62	F4								F5	F6	F7
63	F8								F9	FA	FB
64	FC								FD	FE	FF

Function 131: 15 minute counts
Function 132: 60 minute counts

Function 129: Turn on Max II Green times
Function 128: Turn on Max Green times

If this signal was the the master, then the coordination plan used would be shown like this.

Function 2: Start Coordination Plan #2
(Functions 1-20 reserved for calling coordination plans)

Event numbers are for reference only.

Local TOD "Free" will override any plan received via an interconnect line.

7.3.8 Software and Tools Available for Analysis

There are many software programs and tools available for traffic analysis. The following is a brief discussion on a few of the most common tools. For more information on the selection of the appropriate tool, see the FHWA Traffic Analysis Toolbox, Volume II: Decision Support Methodology for Selecting Traffic Analysis Tools.

Critical Movement Analysis
The critical movement analysis method is a sketch planning-level tool used to get a quick ballpark estimate of whether the existing or forecasted volumes at a signalized intersection will be under, near or over the intersection’s capacity. It is for estimation only, not used to report v/c ratios as a final product or to compare to mobility standards. The analysis requires the intersection approach volumes, number of lanes and lane geometry on each approach.

Each of the movement pairs in conflict at the intersection (e.g., the westbound left and the eastbound through movements) are the focus of the analysis. The total volume included in each conflict pair is calculated to find the highest (or critical movement pair) for each roadway. Where multiple lanes exist in a lane group, use available data on lane utilization; if there is no data on lane utilization, for this procedure assume an even distribution per lane.

The critical movement pairs for each roadway are then summed and compared with the following standards, as shown in Exhibit 7-16:

Exhibit 7-16 Intersection Performance Assessment by Critical Volume

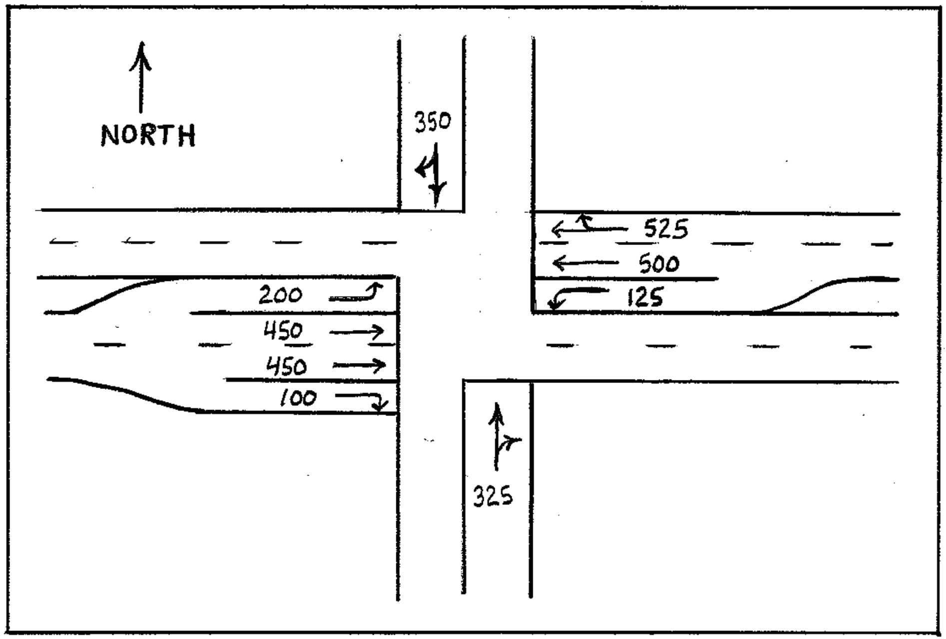
Sum of Critical Volumes (Vehicles/Hour/Lane)	Performance
0 to 1,200	Under Capacity
1,201 to 1,400	Near Capacity
1,401 and Above	Over Capacity

Critical movement analysis only estimates the intersection's ability to accommodate the projected volumes. It does not estimate vehicle delay, level of service or vehicle queue lengths.

Example 7-8 Critical Movement Analysis

The figure below illustrates the signalized intersection of a five-lane highway with a two-lane cross-street. For this intersection, conduct critical movement analysis.

Critical Movement Analysis Example



Solution:

For the east-west roadway, the conflict pairs include:

- 200 (EB LT) + 525 (WB TH/RT) = 725
- 200 (EB LT) + 500 (WB TH) = 700
- 125 (WB LT) + 450 (highest EB TH) = 575
- 125 (WB LT) + 100 (EB RT) = 225

The highest total volume in a conflict pair occurs for the EB LT and WB TH/RT. Therefore, the critical movement volume for the east-west roadway is 725 vehicles.

For the north-south roadway, the conflict pairs include:

- 350 (SB TH/RT) = 350
- 325 (NB TH/RT) = 325

For these approaches there are no conflicting movements, so the highest total volume on an approach is taken as the critical movement. Therefore, the critical movement volume for the north-south roadway is 350 vehicles.
The sum of the critical movement volumes for the intersection becomes:

725 (east-west) + 350 (north-south) = 1,075

Compared to the performance thresholds shown in Exhibit 7-16 this intersection is estimated to be operating under capacity.

Intersection Capacity Utilization

The Intersection Capacity Utilization (ICU) method is another signalized intersection sketch planning-level tool used to get a quick ballpark estimate of how much reserve capacity is available or how much the intersection is over capacity. It compares the current traffic volume to the intersection's ultimate capacity. It is for estimation only not used to report v/c ratios as a final product or to compare to mobility standards. The method sums the amount of time required to serve all movements at saturation for a given cycle length and divides by that reference cycle length. This method is similar to taking a sum of critical volume to saturation flow ratios (v/s), yet allows minimum timings to be considered. While it does not predict delay, it can be used to predict how often an intersection will experience congestion. The ICU method can provide reasonable estimates for intersection capacity conditions, but should not be used for detailed operational analysis.

The ICU is timing plan independent, yet has rules to insure that minimum timing constraints are taken into account. This removes the choice of timing plan from the capacity results. The ICU can also be used on unsignalized intersections to determine the capacity utilization if the intersection were to be signalized.

The ICU Level of Service (LOS) should not be confused with delay-based levels of service such as the HCM. Both are providing information about the performance of an intersection, but are measuring a different objective function. The ICU LOS reports out the amount of reserve capacity or capacity deficit. The delay based LOS reports out on the average delay experienced by motorists.

SIGCAP2, UNSIG10

SIGCAP2 is an ODOT developed computer program similar to the ICU. It is also based on the 1985 HCM. It is a sketch planning-level tool, timing plan independent, used to get a quick ballpark estimate of a signalized intersection v/c ratio. It is for estimation only, not used to report v/c ratios as a final product nor to compare to mobility standards. It can be used to estimate LOS C volumes for Environmental traffic data.

UNSIG10 is an ODOT written computer program that analyzes unsignalized intersections. It is a sketch planning-level tool used to get a quick ballpark estimate of whether and by what magnitude the existing or forecasted volumes at a signalized intersection will be under, near or over the intersection's capacity. It is for estimation only, not used to report v/c ratios as a final product nor to compare to mobility standards. It can be used to estimate LOS C volumes for Environmental traffic data.

Traffix

Traffix is a computer program that calculates level of service at isolated signalized and unsignalized intersections based on the HCM methods. There is no interaction between the intersections, similar to the Highway Capacity Software (next software covered). This program is frequently used for evaluating the impacts of proposed developments. It facilitates the process of trip distribution and assignment over a street network making it easier to test multiple development scenarios and different mitigation measures. The Traffix program uses Zones, Gates, Paths, Routes and Attractions to simulate an existing network and the addition of a potential development. The program can be used to develop both existing and future traffic volumes for several alternatives, evaluate potential signal timing (but not progression) and generate Level of Service and HCM reports for intersections (signalized and unsignalized). A Traffix file can be converted over to a Synchro file (some details don't transfer), saving time creating new files and inputting different volume scenarios.

Local jurisdictions often use Traffix to track various development proposals and to keep an inventory of their network. Traffix is often used for TIAs and similar analysis work. This tool is also used when working with cumulative analysis of small communities and small regional projects. Traffix may be a better tool for analysis in an area with several new developments or experiencing unusually fast growth that out paces historical growth rates.

There are some limitations of Traffix. Traffix does not use ODOT's accepted analysis procedure for roundabouts. The electronic files (input and output) will need to be provided. Screen prints may also be required to show various inputs. Traffix queue lengths must not be used for unsignalized intersections and may only be used for isolated signalized intersections where no simulation is being performed. Gates may be needed between attractions to show trips occurring between attractions. In a model based forecast, volumes should be post processed and are not considered to be when using a factor or multiplier in this program.

Highway Capacity Software

Highway Capacity Software (HCS) implements the procedures defined in the HCM for analyzing capacity and determining LOS for signalized intersections, unsignalized intersections, urban streets (arterials), freeways, weaving areas, ramp junctions, multi-lane highways, two-lane highways and transit. Intersection analysis is based on the methodologies presented in Chapters 16 and 17 of the HCM. While the HCS is a widely used tool, it can only accurately analyze intersections in an isolated environment, free from the effects of other intersections.

Synchro/SimTraffic

Synchro is a complete software package for modeling and optimizing traffic signal timings. Synchro implements both the Intersection Capacity Utilization (ICU) 2003 method for determining intersection capacity, as well as the methods of the HCM, Chapters 15, 16 and 17; Urban Streets, Signalized Intersections and Unsignalized Intersections and reports both results. For analysis of ODOT facilities, the signalized intersection v/c ratio or the unsignalized highest movement v/c ratio obtained from the HCM Signalized and Unsignalized reports shall be used.

Synchro is the preferred analysis tool for areas where surrounding intersection operations can influence each other, as it will consider the effects of the coded transportation network on each intersection. This software is also suggested for projects where traffic simulation will be desired, because the street network and operational parameters used can be directly transferred to the SimTraffic program or other simulation programs. ODOT has conducted extensive research on the use of Synchro for analyzing state facilities and has documented several procedures for implementation and default values, which are provided in the next section. NOTE: Many of these procedures also apply to other programs, such as HCS-Signals and should be used where applicable.

7.3.9 Synchro Settings

This section shows the ODOT Synchro settings organized by window. The Simulation Settings Window is only used by SimTraffic and is covered in Section 8.3. The bullet points below only cover the important inputs.

Lane Window

- **Ideal Saturated Flow Rate –** default is 1900; however, ODOT's default is 1750 (see Section 3.5.2). The best way to determine the Saturated Flow Rate is to measure it in the field. (See HCM 2000 Chapter 16, Appendix H)
- **Lane Utilization Factor, F_{LU}, –** is calculated by Synchro, but may be overridden and can have a large impact on the movement saturation flow rate. This factor shall be calculated by the analyst if groups of two or more lanes exist including through and turn lanes that might be affected by uneven lane distribution. Uneven lane distribution can either occur with nearby downstream turn movements or where through lanes drop or add.

  F_{LU} = 1/(n * (Proportion in Heaviest Travel Lane))

  where

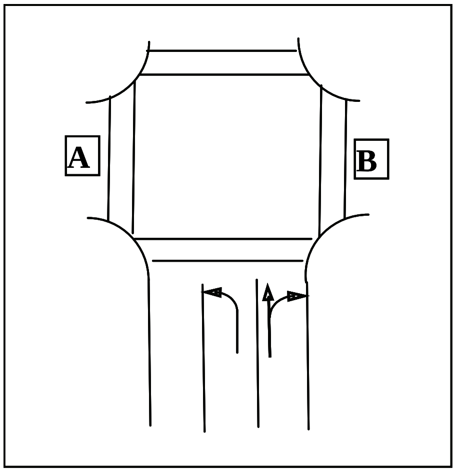
  n= number of lanes in lane group

Volume Window

- **Conflicting Peds –** enter the number of pedestrians that conflict with the permissive right turn movements and the permissive left-turn movements. This value will generate pedestrians in SimTraffic for unsignalized intersections, so this value should only be coded for actual pedestrian paths.

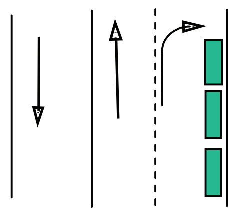
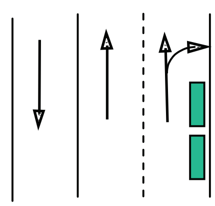
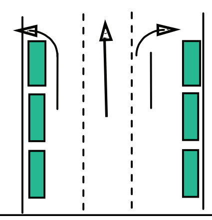
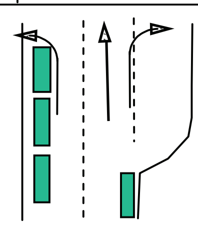
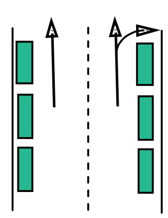
  In Exhibit 7-17, pedestrians in Crosswalk A conflict with the northbound lefts and pedestrians in Crosswalk B conflict with the northbound rights. Pedestrians will not reduce the saturation flow rate for protected turn movements or through movements.

Exhibit 7-17 Conflicting Pedestrian Movements



- **Peak Hour Factor** (PHF) – enter the peak hour factors. For current year analysis, use the actual PHF determined from the manual counts. For future year analysis, use the ODOT PHF default values unless the current PHF's are larger as shown in Section 5.3.
- **Heavy Vehicles** – enter the percentage of trucks and buses for the hour being analyzed for each approach or movement. These values should match the classification count information.
- **Adjacent Parking Lane** – if there is on street parking for this approach, check the box for the adjacent parking lane and enter the number of parking maneuvers per hour. Enter parking maneuvers for each lane group that is affected. Note that parking with zero parking maneuvers per hour is different from no parking as the adjacent parking lane still has an impact. Exhibit 7-18 shows typical parking scenarios and shows the affected lane groups.

Exhibit 7-18 Parking Coding

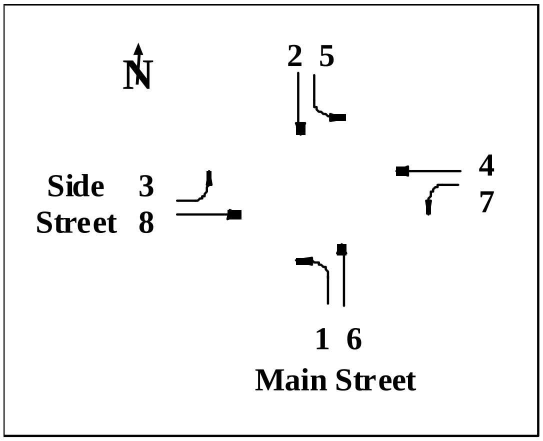
Lane Configurations		Affected Lane Groups		
		Left	Through	Right
	1 Through, 1 Right Long Storage			X
	2 Through		X	
	1 Left, 1 Through, 1 Right Long Storage	X		X
	1 Left Long Storage, 1 Through, 1 Short Right Storage	X	X	X
	2 Through		X	

- **Traffic From Mid-Block** – this is the proportion of traffic that comes from driveways and minor unsignalized intersections. This field should be used instead of trying to code multiple adjacent driveways which will result in excessive congestion.

Timing / Signing Window

- Exhibit 7-19 shows the ODOT default for signal phasing. This is different from the Synchro defaults. (Alternatively, if Main Street ran E-W, phase 6 would be WB.) This needs to be appropriately set for each intersection to have the signal operation work correctly.

Exhibit 7-19 Signal Phasing Diagram



- **Description** – This field can be used to record changes to settings when modifying alternatives, timing, calibrating simulations, or when reviewing other's files.

- **Controller Type:**
    - Actuated-Uncoordinated – This is the primary controller type used by ODOT in isolated situations. When analyzing for a new isolated signal, this is generally the correct controller type to assume;
    - Actuated-Coordinated – This is the primary controller type used by ODOT in progressed network situations;
    - Pretimed – This is used primarily in grid network situations (i.e. downtown networks) or older controllers on city streets;
    - Semi Actuated-Uncoordinated – No longer used by ODOT for permanent controller types, but it may be found on city or county facilities.
    - Unsignalized – Stopped controlled intersections;
    - Roundabouts – Synchro 7.0 will analyze single-lane roundabouts using the older HCM methodology which is not the same as ODOT's. Please refer to Section 7.3.6 for roundabout analysis procedures

  For existing networks, the type of controller can be determined by observation or through contacting your Region Traffic office. In new construction, the analysis will determine what controller type will be necessary.

- **Cycle Length (s) –** Good guidance for a maximum initial cycle length is determined by the number of phases: two phase = 60 s, three phase = 90 s, four phase =120 s.

- **Referenced to –** ODOT standard is set to “Beginning of yellow” For Type 170 signal controllers. Newer Type 2070 controllers use “Beginning of Green.” This specifies the phase the offset is referenced to.

- **Reference Phase –** the coordinated phases for an actuated signal. ODOT uses the mainline phases 2 and 6.

- **Yellow and All-Red Time –** Use the ODOT defaults as shown in Exhibit 7-20 when grades do not exceed 3 percent. For grades that do exceed 3 percent, the ITE formula below should be used for the yellow clearance intervals. Left turns may be treated as 25 mph approaches.

  ITE Yellow Clearance Intervals

  $$y = t + \frac{v}{2a + 2Gg}$$

  Where:
  y = length of the yellow interval, to the nearest 0.1 sec
  t = driver perception-reaction time, recommended as 1.0 sec
  v = velocity of approaching vehicle, in ft/sec (or m/sec)
  a = deceleration rate, recommended as, 10 ft/sec^2 (3.05 m/sec^2)
  g = acceleration due to gravity, 32 ft/sec^2 (9.8 m/sec^2)
  G = grade of approach (3% downgrade would appear as -0.03)

- **Lost Time Adjust –** ODOT default for lost time is 4.0 seconds (unless unusual conditions exist that would warrant a longer time). Synchro 7 redefined the lost time calculation so it is necessary to adjust the lost time up or down to match the default. The lost time adjustment is equal to the difference between the sum of the yellow and all-red times and the lost time default. See Exhibit 7-20 for the default lost time adjustments.

Exhibit 7-20 Recommended Yellow, All-Red & Lost Time Adjustment Values*

85th Percentile Speed (mph)	Yellow (s)	All Red (s)	Lost Time Adjustment (s)
25	3.5	0.5	0.0
30	3.5	0.5	0.0
35	4.0	0.5	-0.5
40	4.3	0.5	-0.8
45	4.7	0.7	-1.4
50	5.0	1.0	-2.0

55	5.0	1.0	-2.0

* These yellow and all-red values are generally applicable where downgrades are less than or equal to 3

Description – This field can be used to record changes to settings when modifying alternatives, timing, calibrating simulations, or when reviewing other's files.

- **Controller Type:**
  - Actuated-Uncoordinated – This is the primary controller type used by ODOT in isolated situations. When analyzing for a new isolated signal, this is generally the correct controller type to assume;
  - Actuated-Coordinated – This is the primary controller type used by ODOT in progressed network situations;
  - Pretimed – This is used primarily in grid network situations (i.e. downtown networks) or older controllers on city streets;
  - Semi Actuated-Uncoordinated – No longer used by ODOT for permanent controller types, but it may be found on city or county facilities.
  - Unsignalized – Stopped controlled intersections;
  - Roundabouts – Synchro 7.0 will analyze single-lane roundabouts using the older HCM methodology which is not the same as ODOT's. Please refer to Section 7.3.6 for roundabout analysis procedures

- **Lagging Phase? –** Checking this box will set this phase to lag the corresponding phases. This is generally for left turns, but can also be set for through and right turns provided that there are separate phases provided.

  Note: With "Dog-house" type permissive-protected left turn signals, lagging left turns are not allowed. Otherwise, a lagging left turn creates the "left-turn trap/yellow trap" where during the change from permissive movements in both directions to a lagging through phase in a single direction, the opposing movement does not stop as may be expected by a driver in the left-turn lane.

- **Recall Mode –** Defines what phases the signal can skip. There are four options: None, Min, Ped and Max. None will allow the phase to be skipped; Min requires the phase to occur for at least the minimum green and cannot be skipped. Ped requires a walk and flashing don't walk time to occur (i.e. in a downtown area) and Max (functionally equivalent to pre-timed) requires the phase to occur for the maximum green and cannot be skipped. Recall settings will only work correctly if consistent with mainline/side-street phase settings. Incorrect phase settings may result in a signal not giving green time to certain moves.

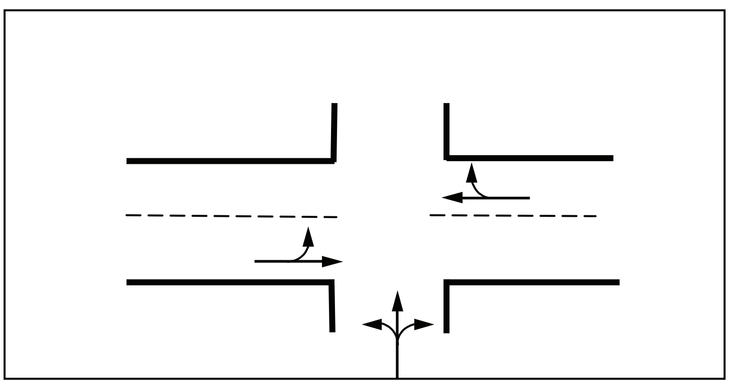
  For:
    - Major protected lefts & minor movements – Set to None;
    - Major through movements – Set to Min (Minimum).
    - Intersections that are at the junction of two progressed systems – Set to Min for all legs.

When the unsignalized controller type is selected, the Timing Window becomes the Signing Window. The below bullet points pertain to the Signing Window.

- **TWLTL Median** – Used to indicate whether a section is a two-way left-turn lane versus a regular median section. This will show the typical TWLTL striping on the screen, but Synchro will not analyze TWLTL operation. Using this setting will assume two-stage gap operation for the side-street even though this movement may not be compatible with a TWLTL. Two-stage gaps should only be coded if actual field observations show such behavior. New build alternatives should not be designed with two-stage gaps. However, the HCM considers wide medians where vehicles stop perpendicular to the mainline also to be a two-stage gap. Use this setting with caution.

- **Critical Gap(s)** – Leave the Synchro calculated default unless the unsignalized intersection is at an interchange ramp terminal or start of a one-way grid section where there are four legs but only three approaches. Synchro 7 does not use the proper gap times for an unsignalized intersection with a one-way minor street such as at an interchange ramp terminal. Synchro 7 is using the gap times appropriate for a four-legged intersection with four approaches, however, one-way minor street intersections have four legs but only three approaches (see Exhibit 7-21 below).

Exhibit 7-21 Four-Leg Three-Approach Intersection Illustration



The critical gap times (t_c) need to be changed for the minor street left turn only. The value is different depending on how many lanes are on the major street (see Exhibit 7-22). All other critical gap times stay the same. After the value is changed it will be in red to indicate a user-overridden value. Deleting the value out and pressing “Enter” will restore the value back to the default setting.

Exhibit 7-22 Critical Gaps for Four-Leg Three-Approach Intersections

	Two-Lane Major Street	**Four/Six-Lane Major Street**
Critical Gap t_c (s)	6.4	6.8

Phasing Window
Pedestrian Timing can have a significant impact on an intersection operation. Timing can be obtained from the signal timing sheets or the Region Traffic offices. Otherwise, the walk time is 7 seconds and the curb-to-curb "Flashing Don't Walk" time is generally calculated at 4 ft/sec for the length of the crosswalk. Areas with more pedestrians or older pedestrians may have different timings, so please check with Region Traffic or Traffic-Roadway Section.

Changing the defaults for the actuated signal phasing settings are only required if a calibrated simulation or actuated signal timing plans will be created. Exhibit 7-23 shows the required Phasing Window settings. These factors have a significant impact on the calibration. These settings can come either from signal timing (preferred) or from Exhibit 7-23.

These settings are defined as:

- Minimum Initial - Minimum green time
- Minimum Split - Minimum green + yellow + all-red + walk + flashing don't walk times. Leave higher values otherwise errors will result.
- Vehicle Extension (also known as "Passage" on a timing sheet) – Time that a detector extends the green time up to the maximum time available for that phase.
- Minimum Gap – Gap time after reduction until end of phase.
- Time Before Reduce (TBR) – Time elapsed before gap time is reduced
- Time To Reduce (TTR) - Time elapsed during gap time reduction to minimum.

Exhibit 7-23 ODOT Phasing Settings Defaults*

Parameter (s)	**Left Turns (s)**	**Mainline Through's (s)**	**Side Street Through's (s)**
Minimum Initial	4.0	10.0	6.0
Minimum Split	13.0 min.	14.0 min.	13.0 min.
Vehicle Extension	2.5	4.0	2.5
Minimum Gap	2.0	2.7	2.0
Time Before Reduce	8.0	10.0	8.0
Time To Reduce	4.0	13.0	4.0

*The values in this table are the general phasing settings from the Traffic-Roadway Section (TRS) except for the minimum gap for left turns and side streets. The TRS minimum gap values for these movements are 0.5 seconds. SimTraffic is too sensitive with the 0.5 second value because it does not allow enough time to adequately represent field conditions and a longer time is needed to prevent excessive queues and gap outs. The 0.5 second value should be retained for signal timing plan construction.

Note that these changes in the Phasing Window, especially minimum split times, might change the cycle length and maximum splits (especially for left turns and side streets), so the system optimization should be re-run.

Detector Window
If timing plans that involve actuated signals or a SimTraffic simulation needs to be created, the

Detector Window data must be entered. Synchro uses this data to model actuated signal operation. Correct detector settings are critical to a successful simulation in Version 7. If actuated signal operation or simulation is not going to be utilized, the Synchro default detector settings can be used.

- **Number of Detectors –** Enter in number of detectors (1 to 3) for a given lane type.
- **Detector Phases –** Phase that is triggered by detection zone. This value is carried over from the Timing Window.
- **Leading and Trailing Detectors –** Not used in Synchro 7 other than to maintain backwards compatibility with earlier versions. These values are automatically updated as more detailed detector position and size data is entered. The Leading Detector is the first detector that a vehicle encounters on an approach (furthest from the stop bar) while the Trailing Detector is the last detector on an approach and closest to the stop bar. **Warning: Do not modify these values if specific detector information is being entered as these will overwrite the detector data and create many errors.**
- **Detector Templates –** While all of the detector data can be added individually, it is very repetitive and the use of the Detector Template can quickly add the correct ODOT detector settings for new or existing signals in Synchro. The TPAU Analysis Tools web page has default Synchro template files that contain the full base ODOT detector settings.
- **Detector Position (ft) –** Enter distance to stop bar. Detector 1 is closest to the stop bar. Position varies by speed on approach and lane type.
- **Detector Type –** Synchro has three detector types – Call, Extend, or Call + Extend. ODOT uses the third type: Call + Extend (Cl+Ex). The detectors are capable of being both a “Call” or a “Extension” detector depending on the signal state. The call function occurs during the red time for phases that require a “call” for a green placed to the signal controller when a vehicle triggers the detector. The extend (or extension) function occurs during the green time and is used to extend the green time to allow vehicles to smoothly flow through the intersection approach.
- **Detector Size (ft) –** Enter size of detector. ODOT standard is a 6 foot diameter loop so enter “6” feet for all extension detectors. Detection on the side street is a pair of loops tied together, so these are coded as a single 16 foot detector.
- **Detector Delay –** Only used for side street exclusive right turn lanes to delay the detection call to the controller. This will limit the number of times that the side street phases will need to come up. This value is usually 10 seconds.

Detector Settings
There is a difference between ODOT detector placement standards which measure to the center versus Synchro which measures to the leading edge of the detector. Exhibit 7-24 shows the ODOT Synchro detector placement.

If turn bays are shorter than 72’ then eliminate Detector 2 as Synchro will give an error as the detection zone will exceed the storage length. Synchro may have trouble showing detectors in the Map Window in shorter turn bays, so it may be necessary to adjust the detector spacing or turn bay length. Short turn bays in the field likely only have a single detector (verify in the field).

If exclusive right-turn lanes on the mainline exist, there is only one detector at 137’ from the stop bar. If the turn bay is shorter than 137’ then adjust the distance to approximately 2/3 of the total bay distance, but do not put the detector at the extreme end of the turn bay as turning vehicles may miss the detector or through vehicles may trigger it.

Detectors for ramps are based on whether the ramp is a low speed ramp, such as a loop ramp (<45 mph), or a high speed ramp, such as a diagonal ramp (≥45 mph).

Exhibit 7-24 Synchro-Adjusted ODOT Detector Type and Position

Lane Type	Speed (mph)	Number of Detectors	Position (distance from stop bar to leading edge of detector, in feet)		
			Detector 1	Detector 2	Detector 3
Side or Left	All	2	2	72	
Right	All	1	137		
Ramp	<45	3	2	72	132
High-speed Ramp	45+	3	2	107	207
Mainline Through	25	1	137		
	30	1	177		
	35	2	107	217	
	40 & 45	2	157	317	
	50	2	187	377	
	55	2	222	447	

It is possible that detector type/placement differs in the field (i.e. call detectors at the stop bar on the mainline), so customization may be necessary. Local intersections can use the ODOT Side Street detector settings or default Synchro data, but should be field-checked for accuracy. Detector spacing can also be modified to fit video detection zones for intersections that have cameras.

Optimizing Signal Operations

Existing conditions (base year) need to be optimized if the timing did not come exclusively from timing sheets. The only exception is when a calibrated existing condition analysis is being used for simulation. Short-term analyses may require optimization even if timing sheets were used. All future no-build conditions and future build alternatives must optimize the signal timing, either as an isolated case or a signal system. Mainline phase orientation, reference phase, offset style, and recall settings are set appropriately before optimizing.

> **Note:** If at any time a change is made to intersection geometry, volumes, signal timing, etc., the system shall be re-optimized.

Existing field timing may or may not be fully optimized; it is often set to minimize motorist delay or queue lengths. For planning purposes or traffic impact studies, ODOT's practice is to optimize the timing for the best intersection v/c ratio. Movement v/c's should be relatively even on the intersection approaches. The cycle length and phase splits should be optimized for each analysis (existing, no-build and build alternatives) except during simulation calibration work, see Chapter 8. Generally, optimizing with longer cycle lengths and fewer vehicle phases will result in a lower v/c, however longer cycle lenths will increase queuing.

- **Intersection Cycle Length Optimization**– This algorithm optimizes the cycle length for a single intersection, based on delay.

- **Intersection Splits Optimization–** This algorithm optimizes the phase splits for a single intersection. This is a good place to start when optimizing the v/c for an intersection. Subsequent adjustments to movement green time may improve the v/c.

  Make sure that Lead/Lag Optimize is set properly. This will allow Synchro to optimize the phase sequence (leading or lagging operation for left turns) when the signal functions as part of a coordinated system. Lagging operation may be inappropriate for high volumes, five-section "doghouse" heads or other concerns. Flashing yellow left turn heads can operate either in leading or lagging mode.

  When optimizing for part (zone) or a whole network the system optimization should use the manual cycle length option so the best system cycle length can be found. An increment of 5 seconds should be used. System cycle lengths:
    - Should not exceed ODOT's 60-90-120 second cycle limits for 2, 3 and 4 phase signals, respectively.
    - Show promise for minimizing delay and stops and maximizing bandwidth
    - Should have a low number of "dilemma vehicles", i.e. a low number of vehicles expected to be caught near the intersection when a signal turns yellow.
    - May change the phase splits at the intersections. Verify that the new split times are acceptable.

The optimized network should have good progression between signals in the system. The quality of the progression will generally be determined by noting the size of the bandwidth for the selected cycle length. The bandwidth should be maximized as much as possible. OAR Division 20 should be reviewed to ensure that the resulting bandwidth is acceptable. Link speeds can be dropped five mph in order to check minimum bandwidth requirements.

The Time-Space diagram will need to be reviewed as Synchro optimizes for delay, not progression bandwidth. The analyst will need to drag the intersection offsets on the time-space diagram with a mouse to improve the arterial bandwidths (the link bands are not used). Bandwidths should be visible for both directions in most cases. Experimentation is often necessary to determine which intersections are critical for increasing the bandwidth. Arterial bandwidths should be maximized for each direction unless it is desired to have a larger bandwidth in a given direction (i.e. outbound commuter flow). In addition, leading/lagging settings in the Timing window should be reviewed for opportunities to improve the bandwidth. The analyst should also consider alternative system cycle lengths to maximize the bandwidth.

Required Synchro Output Reports
The following reports should be retained for file documentation about the no-build conditions or an alternative.

- **Lanes, Volumes, Timings Report** - This report contains the information that is used as inputs into Synchro. When reviewing a Traffic Impact Study (TIS), it is essential to have this report in order to verify the analysis results. The Synchro default reports are adequate.
- **Queues Report** - This report contains information about estimated queue lengths and

blocking. Synchro 7.0 reports two queue lengths:

- The 50th percentile queue is the largest queue length for a **typical** cycle;
- The 95th percentile queue is the maximum back of queue with **95th percentile** traffic volumes. This is the queue length used by ODOT to determine recommended storage lengths.
- **Note: Synchro calculates queue lengths as the maximum queue after only two cycles**. In conditions where the v/c > 0.70, Synchro queues may not accurately reflect queuing projections, especially if the intersection/node spacing is less than the estimated queuing. Watch for presence of “m” or “#” codes next to the queuing values. The “m” means that an upstream signal is metering the queue, so queues in this movement are actually shorter than they would be if there was not a constraint elsewhere. The “#” means that the 95th percentile volume is over capacity so queues shown are likely much longer, even up to twice shown. In these cases, the Synchro-based queues are not adequate and SimTraffic simulations are required for intersection queuing. In constrained analyses where the v/c ≥ 0.90, arrival rates become unstable and the estimated queue lengths in Synchro are unreliable and SimTraffic simulation-based queues are required.

- **HCM Signals or Unsignalized Report** - Synchro 7.0 will report out analysis of both signalized and stop controlled intersections. These follow the methodologies in Chapter 16 (Signalized) and Chapter 17 (Unsignalized) of the HCM 2000.
  - HCM Signals - This report generally follows the same outputs as in the HCM and the Highway Capacity Software (HCS). Some items may be different such as actuated green times which in turn may affect some of the calculations. Note that the HCM Signals Report will skip rows with unused or just default calculations.
  - HCM Unsignalized - Synchro 7.0 will analyze two-way and all-way stop controlled intersections following HCM 2000 methodology. The effect of upstream traffic signals is now included in the Synchro analysis.
  - HCM Roundabout - Synchro 7.0 will now analyze single-lane roundabouts using HCM 2000 methodology; however this is not compatible with the current ODOT roundabout methodology.

7.4 Traffic Signal Warrants

Because the presence of traffic signals can degrade some aspects of overall traffic operations on a highway in addition to the improvements they provide, traffic signal warrants are used to determine when installation may be justified by identifying conditions where the benefits may outweigh the costs. The MUTCD provides a set of 8 warrants to be used in determining if the installation of a traffic signal should be considered. In addition to these, the ODOT Transportation Planning Analysis Unit has also developed a set of “preliminary” traffic signal warrants, which are based on the MUTCD warrants, but require less data for analysis. The preliminary warrants are generally not accepted as a basis for approving the installation of a traffic signal, but are useful for projecting signalization needs for future years. Full warrants are evaluated later as part of the engineering study required by the MUTCD. Many other considerations go into determining whether a signal should be installed. For example, a signal installation is generally not appropriate in a rural area. The MUTCD and Preliminary Signal Warrant (PSW) methodologies are described below.

When evaluating signal warrants (preliminary or MUTCD), it is important to include only the appropriate lane configurations and traffic volumes. Incorrect modeling of intersections is a very common mistake and can make a significant difference to the outcome of the analysis. There may be times when minor streets need to be modeled as major streets because of high side-street volumes (e.g., rural interchange) or left turns behave as right turns when dealing with one-way streets. In such cases, sound engineering judgment is critical to obtaining accurate analysis. Direction for proper modeling of intersections when analyzing signal warrants is included in the next section.

Traffic signal warrants must be met and the State Traffic Engineer’s approval obtained before a traffic signal can be installed on a state highway. However, approval of a signal depends on more than just a warrant analysis. Meeting a warrant is necessary to install a signal, but it does not mean a signal should be recommended or guarantee its installation. Considerations to be evaluated include safety concerns, alternatives to signalization, signal systems, delay, queuing, bike and pedestrian needs, railroads, access, consistency with local plans, local agency support and others. The engineering investigation, conducted or reviewed by the Region Traffic Engineer, must demonstrate a reduction in delay, improvements in safety, improved connectivity or some other "benefit" and why a signal is the best solution as compared to other alternatives, such as listed in MUTCD Section 4B.04a. During the consideration, the Region Traffic Engineer, input from TRS must be obtained prior to reaching any conclusions. Coordination with TRS should occur early in the project process to allow sufficient time to develop and evaluate alternatives to signalization if deemed necessary. Once the investigation and recommendation is reviewed, TRS will act on the request.

If preliminary signal warrants are met, project analysts need to forward a copy of the PSW form and analysis to TRS and coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual.

7.4.1 Preliminary Signal Warrants

Introduction
The single most important criterion for preliminary signal warrant analysis is engineering judgment. In the following procedures only the fundamental parameters of volumes and approach lanes are provided.

Background
There are 8 traffic signal warrants found in the MUTCD, Page 4C-1. The signal warrants are:

- Warrant 1, Eight-Hour Vehicular Volume
  - Case A – Minimum Vehicular Volume
  - Case B – Interruption of Continuous Traffic
- Warrant 2, Four-Hour Vehicular Volume
- Warrant 3, Peak Hour
- Warrant 4, Pedestrian Volume
- Warrant 5, School Crossing
- Warrant 6, Coordinated Signal System
- Warrant 7, Crash Experience
- Warrant 8, Roadway Network

OAR 734-020-0460 (1) stipulates that only MUTCD Warrant 1 Case A and Case B may be used to project future needs for traffic signals beyond three years from the present time (Corrected to reflect numbering used in the Millennium Edition of the MUTCD). Case A deals primarily with high volumes on the intersecting minor street. Case B addresses high volumes on the major street and the delays and hazards to vehicles on the minor street trying to either access or cross the major street. The preliminary warrant is considered satisfied if either Case A or Case B is met.

Information for Narrative
The following statement should be included in the Analysis Methodology section of the Narrative:

> TPAU uses Signal Warrants 1, Case A and Case B (MUTCD), which deal primarily with high volumes on the intersecting minor street and high volumes on the major-street. Meeting preliminary signal warrants does not guarantee that a signal shall be installed. Before a signal can be installed a field warrant analysis is conducted by the Region. If warrants are met, the State Traffic Engineer will make the final decision on the installation of a signal.

Analysis
In MUTCD Warrant 1 the eighth highest hour of an **average** day is used to determine whether a warrant is met. At the analysis stage in TPAU, ADT is used for preliminary signal warrant analysis. A conversion factor of 5.65% is applied to the ADT to reach the eighth highest hour. The conversion factor of 5.65% was developed based on a study of 1991 to 1994 manual counts and as agreed on by TPAU and TRS. This factor was used to convert MUTCD hourly volumes to

ADT volumes (divided the MUTCD volume by the factor .0565). This equals the target ADT volume to meet MUTCD Warrant 1. As an example, for Case A to be met the MUTCD requires a minimum total of 500 vehicles per hour on both approaches of the major street, where the major and minor streets both have only one lane for moving traffic (at 100%, assuming no reductions). To convert this to ADT volumes, the following calculations are made:

$$ADT = \frac{500}{0.0565} = 8{,}850$$

These calculations have already been completed for the analyst, as can be seen in Exhibit 7-25.[5]

If the 85th percentile speed of major street traffic exceeds 40 mph in either an urban or rural area or when the intersection lies within the built-up area of an isolated community (typically non-MPO) having a population of less that 10,000, reduce the target volume for the warrants to 70 percent of the normal requirements. The warrant volumes, along with the number of lanes, are shown in the preliminary traffic signal warrant analysis sheet in Exhibit 7-25.

[5] Note that the value of 8,850 calculated in the analysis example is the same as the value on the worksheet for this scenario.

Exhibit 7-25 Preliminary Traffic Signal Warrant Analysis Form

Oregon Department of Transportation
Transportation Development Branch
Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	**Minor Street:**
Project:	**City/County:**
Year:	**Alternative:**

Preliminary Signal Warrant Volumes

Number of Approach Lanes		ADT on Major Street Approaching From Both Directions		ADT on Minor Street, Highest Approaching Volume	
Major Street	Minor Street	Percent of Standard Warrants		Percent of Standard Warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8,850	6,200	2,650	1,850
2 or more	1	10,600	7,400	2,650	1,850
2 or more	2 or more	10,600	7,400	3,550	2,500
1	2 or more	8,850	6,200	3,550	2,500
Case B: Interruption of Continuous Traffic					
1	1	13,300	9,300	1,350	950
2 or more	1	15,900	11,100	1,350	950
2 or more	2 or more	15,900	11,100	1,750	1,250
1	2 or more	13,300	9,300	1,750	1,250
5.65% of the above ADT volumes is equal to the MUTCD vehicles per hour (vph)					
	100 percent of standard warrants				
	70 percent of standard warrants[2]				

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major				
	Minor				
Case B	Major				
	Minor				
Analyst and Date:			Reviewer and Date:		

[1] Meeting preliminary signal warrants does not guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

[2] Used due to 85th percentile speed in excess of 40 mph or isolated community with population of less than 10,000.

Determining the number of approach lanes and determining the approach volumes to use in the warrant analysis requires knowledge of the involved intersection.

1. Major Street (Higher Volume Street)

   - Include only the through and through/turn lanes in the number of approach lanes.
   - For the ADT, count total volume approaching from both directions, including all turn movements.

2. Minor Street (Lower Volume Street)

   - Include only the through, through/turn and left turn lanes in the number of approach lanes.
   - For the ADT, count the highest approaching volume (one direction only, do not include the ADT approaching from both directions) including some or none of the right turn volume as discussed in the following scenarios and examples:
     - **Scenario # 1 – Shared Left-Through-Right Lane**: Some of the right turns are included in the minor street approach ADT if the right turn demand is greater than 85% of the capacity of the shared lane. Use unsignalized capacity analysis to calculate the capacity of the shared lane. The right turn discount is 85% of the shared lane capacity (85% of the capacity is used because once the v/c exceeds 0.85, drivers suffer longer delay and begin to take unsafe gaps). Subtract the right-turn discount from the total right turn volume to determine the number of right turns in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

Example 7-9 Right Turn Discount for Shared Left/Through/Right Lane

Example Application: Right Turn Discounts (Only for the minor road.)

The diagram below shows a typical unsignalized intersection, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The 85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound shared lane capacity is 120 vph. The right-turn discount is 85% of the shared lane capacity, 120 x 0.85 = 102 right turns. The number of right turns included in the warrant would be 180 – 102 = 78.
- Determine the minor approach ADT. The minor street approach peak hour volume used in the warrant is 90+50+78 = 218. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (218 / 0.10) = 2,180.

Example Volume Diagram

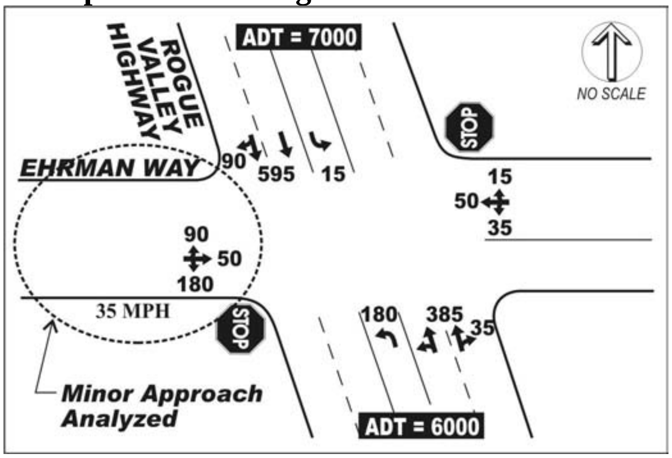


Exhibit 7-26 Signal Warrant Analysis Example

Oregon Department of Transportation
Transportation Development Branch
Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	Single Lane Minor Approach L/T/R

Preliminary Signal Warrant Volumes

Number of Approach Lanes		ADT on Major Street Approaching from Both Directions		ADT on Minor Street, Highest Approaching Volume	
Major Street	Minor Street	Percent of Standard Warrants		Percent of Standard Warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8,850	6,200	2,650	1,850
2 or more	1	10,600	7,400	2,650	1,850
2 or more	2 or more	10,600	7,400	3,550	2,500
1	2 or more	8,850	6,200	3,550	2,500
Case B: Interruption of Continuous Traffic					
1	1	13,300	9,300	1,350	950
2 or more	1	15,900	11,100	1,350	950
2 or more	2 or more	15,900	11,100	1,750	1,250
1	2 or more	13,300	9,300	1,750	1,250

5.65% of the above ADT volumes is equal to the MUTCD vehicles per hour (vph)

x	100 percent of standard warrants
	70 percent of standard warrants[2]

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2+	10,600	13,000	
	Minor	1	**2,650**	**2,180**	N
Case B	Major	2+	**15,900**	**13,000**	
	Minor	1	1,350	2,180	N

Analyst and Date:	Reviewer and Date:

[1] Meeting preliminary signal warrants does not guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

[2] Used due to 85th percentile speed in excess of 40 mph or isolated community with population of less than 10,000.

The figure above shows the Preliminary Signal Warrant Analysis for Example 7-9. The preliminary signal warrant is not met because the Minor Street ADT is less than the warrant volume in Case A and the Major Street ADT is less than the warrant volume in Case B, as shown in the darkened cells.

Scenario # 2 – Exclusive Right-Turn Lane: Some of the right turns are included in the approach ADT if the right turn lane demand is greater than 85% of the capacity of the right turn lane. Use unsignalized capacity analysis to calculate the capacity of the right turn lane. The right turn discount is 85% of the right turn lane capacity. Subtract the right turn discount from the total right turning volume to determine the number of right turns that will be included in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

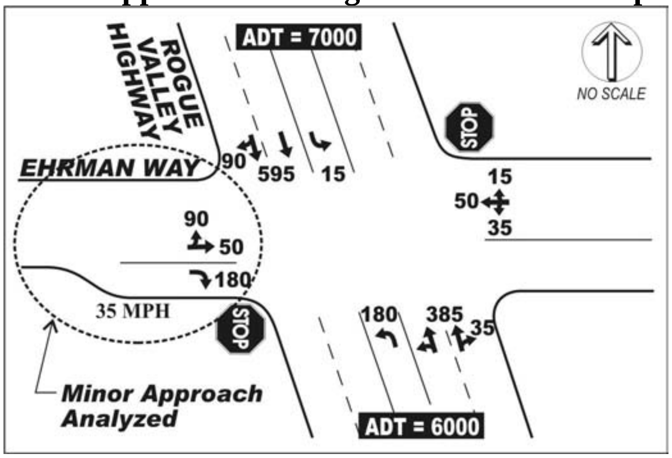
Example 7-10 Right Turn Discount for Exclusive Right Lane Lane

The diagram below shows a typical unsignalized intersection with a separate right turn lane on the eastbound approach, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The 85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound right turn lane capacity is 639 vph. The right turn discount is 85% of the shared lane capacity, 0.85 x 639 = 543 right turns. The number of right turns included in the warrant is 180-543 = -363 = 0. If the number is less than or equal to zero, do not include any right turns in the warrant. The EB right turn lane is not included in the number of approach lanes.
- Determine the minor approach ADT. The minor approach peak hour volume used in the warrant is 90+50+0 = 140. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (140/0.10) = 1,400.

The form below shows the Preliminary Signal Warrant Analysis for Example 7-10. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Case A and the Major Street ADT is less than the warrant volume in Case B, as shown in the darkened cells.

Minor Approach with Right Turn Lane Example



Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation **Transportation Development Branch** **Transportation Planning Analysis Unit**					
Preliminary Traffic Signal Warrant Analysis[1]					
Major Street:	Rogue Valley Highway		**Minor Street:**	Ehrman Way	
Project:	Ehrman Way		**City/County:**	Medford	
Year:	1995		**Alternative:**	2 Lane Minor Approach L/T, R	
Preliminary Signal Warrant Volumes					
Number of Approach Lanes		ADT on Major Street Approaching from Both Directions		ADT on Minor Street, Highest Approaching Volume	
Major Street	Minor Street	Percent of Standard Warrants		Percent of Standard Warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8,850	6,200	2,650	1,850
2 or more	1	10,600	7,400	2,650	1,850
2 or more	2 or more	10,600	7,400	3,550	2,500
1	2 or more	8,850	6,200	3,550	2,500
Case B: Interruption of Continuous Traffic					
1	1	13,300	9,300	1,350	950
2 or more	1	15,900	11,100	1,350	950
2 or more	2 or more	15,900	11,100	1,750	1,250
1	2 or more	13,300	9,300	1,750	1,250
5.65% of the above ADT volumes is equal to the MUTCD vehicles per hour (vph)					
x	100 percent of standard warrants				
	70 percent of standard warrants[2]				
Preliminary Signal Warrant Calculation					
	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2+	10,600	13,000	N
	Minor	1	**2,650**	**1,400**	
Case B	Major	2+	**15,900**	**13,000**	N
	Minor	1	1,350	1,400	
Analyst and Date:			Reviewer and Date:		

[1] Meeting preliminary signal warrants does not guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer’s approval obtained before a traffic signal can be installed on a state highway.

[2] Used due to 85th percentile speed in excess of 40 mph or isolated community with population of less than 10,000.

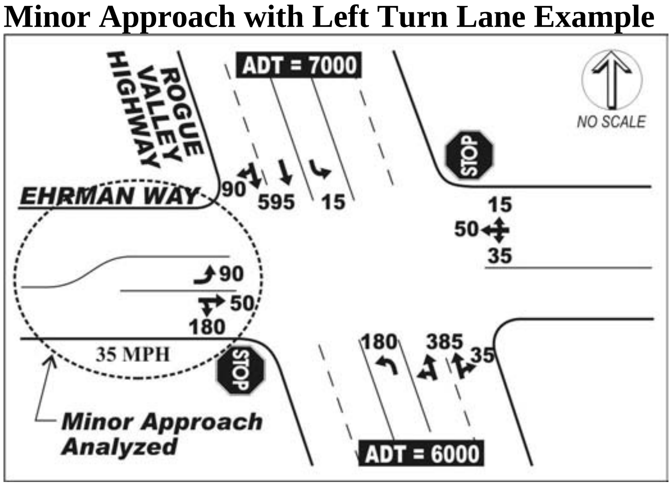
- Scenario # 3 – Shared Through-Right Lane: Some of the right turns are included in the approach ADT if the right turn demand is greater than 85% of the capacity of the shared through-right lane. Use unsignalized capacity analysis to calculate the capacity of the through-right shared lane. The right turn discount is 85 % of the shared lane capacity. Subtract the right turn discount from the total right turn volume to determine the number of right turns in the warrant. If the remainder is less than or equal to zero, do not include any of the right turns in the approach ADT.

Example 7-11 Right Turn Discount for Shared Through/Right Lane

The diagram below shows a typical unsignalized intersection with a shared through-right lane on the eastbound approach, the peak hour volumes, the ADT volumes and lane configurations. The peak hour volumes are 10% of the ADT. The 85th percentile speed is 35 mph and the intersection is located in a city with a population of 60,000.

- Determine the number of right-turns to include in the warrant. Using an unsignalized intersection methodology it was determined that the eastbound shared lane capacity is 277 vph. The right turn discount is 85% of the shared lane capacity, 0.85 x 277 = 235 right turns. The number of right turns included in the warrant is 180 – 235= -55 = 0. If the number is less than or equal to zero, do not include any right turns in the warrant. The EB right turn lane is not included in the number of approach lanes.
- Determine the minor approach ADT. The minor approach peak hour volume used in the warrant is 90+50+0= 140. Since the peak hour volume is 10% of the ADT, the minor approach ADT is (140 / 0.10) = 1,400.
- The form below shows the Preliminary Signal Warrant Analysis for Example 7-6. The preliminary signal warrant is not met since the Minor Street ADT is less than the warrant volume in Case A and the Major/Minor Street ADT's are both less than the warrant volumes in Case B, as shown in the darkened cells.

Minor Approach with Left Turn Lane Example



Warrant Analysis of Minor Approach #1 Example Conditions

Oregon Department of Transportation
Transportation Development Branch
Transportation Planning Analysis Unit

Preliminary Traffic Signal Warrant Analysis[1]

Major Street:	Rogue Valley Highway	**Minor Street:**	Ehrman Way
Project:	Ehrman Way	**City/County:**	Medford
Year:	1995	**Alternative:**	2 Lane Minor Approach L, T/R

Preliminary Signal Warrant Volumes

Number of Approach Lanes		ADT on Major Street Approaching from Both Directions		ADT on Minor Street, Highest Approaching Volume	
Major Street	Minor Street	Percent of Standard Warrants		Percent of Standard Warrants	
		100	70	100	70
Case A: Minimum Vehicular Traffic					
1	1	8,850	6,200	2,650	1,850
2 or more	1	10,600	7,400	2,650	1,850
2 or more	2 or more	10,600	7,400	3,550	2,500
1	2 or more	8,850	6,200	3,550	2,500
Case B: Interruption of Continuous Traffic					
1	1	13,300	9,300	1,350	950
2 or more	1	15,900	11,100	1,350	950
2 or more	2 or more	15,900	11,100	1,750	1,250
1	2 or more	13,300	9,300	1,750	1,250

5.65% of the above ADT volumes is equal to the MUTCD vehicles per hour (vph)

x	100 percent of standard warrants
	70 percent of standard warrants[2]

Preliminary Signal Warrant Calculation

	Street	Number of Lanes	Warrant Volumes	Approach Volumes	Warrant Met
Case A	Major	2+	10,600	13,000	
	Minor	2	**3,550**	**1,400**	N
Case B	Major	2+	**15,900**	**13,000**	
	Minor	2	**1,750**	**1,400**	N

Analyst and Date:	Reviewer and Date:

[1] Meeting preliminary signal warrants does not guarantee that a signal will be installed. When preliminary signal warrants are met, project analysts need to coordinate with Region Traffic to initiate the traffic signal engineering investigation as outlined in the Traffic Manual. Before a signal can be installed, the engineering investigation must be conducted or reviewed by the Region Traffic Manager who will forward signal recommendations to headquarters. Traffic signal warrants must be met and the State Traffic Engineer's approval obtained before a traffic signal can be installed on a state highway.

[2] Used due to 85th percentile speed in excess of 40 mph or isolated community with population of less than 10,000.

- Scenario # 4 – Double Right-Turn Lane: Include all of the right turning volume in the approach ADT if a double right turn lane is required. If such is the case, the number of approach lanes for warrant analysis is 2 or more.

The above information is meant to serve as general guidelines only. Engineering judgment may be required when one or both of the streets are one way, the intersection is not a typical four legged design or the highest volume is associated with a turn movement. Engineering judgment must be the deciding factor in preliminary warrant analysis.

7.4.2 Manual of Uniform Traffic Control Devices Signal Warrants

As previously noted, the MUTCD provides 8 warrants to be used in determining whether the installation of a traffic signal is justified for a given location. It should be noted that while the MUTCD states that a traffic signal should not be installed unless one or more of the warrants are met, it also emphasizes that meeting one or more warrant shall not in itself require the installation of a traffic signal and that the analysis of the warrants should be included as part of a comprehensive engineering study. The MUTCD warrants, if evaluated, should be evaluated along with all the other components of a full traffic signal engineering investigation as described in the ODOT Traffic Manual. MUTCD signal warrants should only be evaluated for existing and future short-term (up to 3 years in the future) conditions. Evaluating the need for a traffic signal over 3 years is not recommended as land uses and travel patterns can change within that time period, therefore, traffic conditions are not as predictable. A brief description of each warrant is included below. For a complete description of the warrants and their appropriate application, see the MUTCD.

- **Warrant 1, Eight-Hour Vehicular Volume**: Either can qualify. This warrant has two conditions.
  - Minimum Vehicular Volume, Condition A, is where a large volume of intersecting traffic is the principal reason to consider installing a traffic signal.
  - The Interruption of Continuous Traffic, Condition B, is for where the major street volume is so heavy that minor street traffic suffers excessive delay or conflicts with the major street.
- **Warrant 2, Four-Hour Vehicular Volume:** Applied where the volume of intersecting traffic is the principal reason to consider signal installation
- **Warrant 3, Peak Hour**: Used at locations where traffic conditions are such that for a minimum of one hour of an average day, the minor street traffic suffers undue delay when entering or crossing the major street.
- **Warrant 4, Pedestrian Volume**: Where the major street volume is so heavy that a large number of pedestrians experience excessive delay in crossing the major street.
- **Warrant 5, School Crossing**: For use where school children crossing the major street is the principal reason to consider installing a traffic signal.
- **Warrant 6, Coordinated Signal System**: For use where progressive movement in a coordinated signal system necessitates installing traffic signals at intersections where they

would not otherwise be needed to maintain proper vehicle platoons.

- **Warrant 7, Crash Experience**: Intended where the severity and frequency of crashes are the reason to consider installing a traffic signal. This should include the three most recent calendar years for which data is available and only those crash types susceptible to correction by traffic signal control should be considered. Generally requires a minimum of 5 such crashes in a 12-month period.
- **Warrant 8, Roadway Network**: Is intended for use where installing traffic signals at some intersections might be justified to encourage concentration and organization of traffic flow on a roadway network.

The MUTCD and ODOT Traffic Signal Policy and Guidelines provide for the installation of traffic signals that meet criteria for special applications. These applications include providing access to fire and other emergency vehicles, regulating the flow of traffic at a freeway ramp, controlling traffic at a drawbridge or at a one-lane facility and temporary installations for construction projects.

7.5 Estimating Vehicle Queue Lengths

Vehicle queues can have a significant effect on highway safety and operation. Queues that exceed the provided storage at turn lanes can block the adjacent through lanes creating a temporary reduction in capacity as well as an unexpected obstruction in the travel lane that could result in a crash. In through lanes long queues can block access to turn lanes, driveways and minor street approaches, in addition to spilling back into upstream intersections. Under these conditions there are significant losses in capacity that can quickly spread to other upstream intersections and adjacent streets. There can also be a higher potential for crashes as drivers turning onto or off of the highway are required to pass through gaps in the queue that provide limited visibility and other drivers incurring long delays become more aggressive. Therefore, the estimation of vehicle queue lengths is an important traffic analysis procedure that should be included in most operational and safety projects.

Estimates of queue lengths should be based on the anticipated arrival patterns, duration of interruptions and the ability of the intersection to recover from momentary heavy arrival rates. The average queue length and the 95th percentile queue length should be shown in the report. The 95th percentile queue length shall be used for design purposes. A queue blockage or spillback condition is considered a problem when the duration exceeds 5 percent of the peak hour. The average vehicle length, including buffer space between vehicles, to be used in analysis shall be 25-feet, unless a local study indicates otherwise, with all queue length calculations rounded up to the next 25-foot increment. Queue lengths subject to over-capacity conditions can only be adequately assessed through the use of simulation software. The 25-foot average does not apply to microsimulation, where vehicle lengths differ by vehicle type. Refer to Chapter 8.

7.5.1 Methodologies for Signalized Movements

For signalized movements queue length estimates are most often recommended to be calculated using traffic analysis software. However, manual methods are also available that can offer acceptable estimates without requiring access to a computer. In either case, engineering judgment should be used to discern whether the results obtained are reasonable.

Manual Methods

Manual methods offer a practical means of estimating queue lengths with little equipment or data required. While they can produce reasonable results, unless otherwise noted, they are generally recommended for planning-level analysis, with the use of specialized software preferred for design purposes.

Left Turn Movement Queue Estimation Techniques

Three common methods of manually estimating vehicle queue lengths for single-lane left turn movements include the use of a nomograph[6] and two "rule of thumb" procedures. The nomograph (Exhibit 7-27) assumes a random rate of arrivals and uses the turning volumes, signal cycle length and a weighted average vehicle length based on the percentage of trucks in the turning volume to estimate vehicle queues at 90th and 95th percentile probabilities of storing all

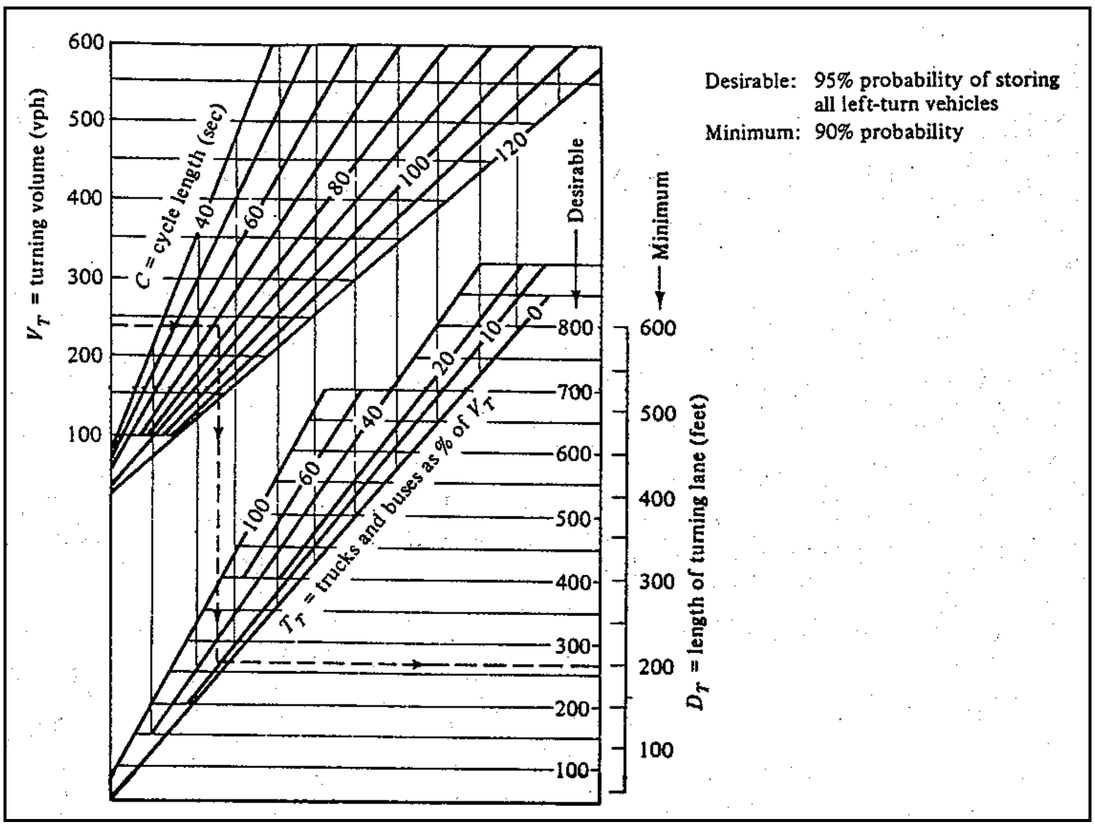
[6] J. E. Leish, *At-Grade Intersections,* A Design Reference Book and Text, Jack E. Leish & Associates, undated.

vehicles.

A "rule of thumb" equation[7] uses similar input while providing a simple procedure that can be applied without need to reference a manual. Using this method, single-lane left turn vehicle queue lengths are estimated as shown below.

> Storage Length = (Volume/Number of Cycles Per Hour) x (t) x (25-feet)
>
> Where "t" is a variable, the value of which is selected based on the minimum acceptable likelihood that the storage length will be adequate to store the longest expected queue. Suggested values are listed in Exhibit 7-28. Typically, transportation analysis uses the 95th percentile queue.

Exhibit 7-27 Nomograph for Estimating Single Lane Left Turn Vehicle Queue Lengths at Signalized Intersections



[7] *Discussion Paper No. 10: Left-Turn Bays*, Transportation Research Institute, Oregon State University, 1996, p. 17.

Exhibit 7-28 Selection of "t" Values

Minimum "t" Value	Percentile
2.0	98 %
1.85	95 %
1.75	90 %
1.0	50 %

It should also be noted that the value of 25-feet used in the equation represents the average storage length required for a passenger car. If a significant number of trucks are present in the turning volumes, the average storage length per vehicle should be increased, as shown in Exhibit 7-29. This adjustment is only for the manual methods; software packages may require a different adjustment.

Exhibit 7-29 Storage Length Adjustments for Trucks

Percent Trucks in Turning Volume	Average Vehicle Storage Length
< 2%	25 ft
5%	27 ft
10%	29 ft

While both the nomograph and the rule of thumb equation are intended for use in estimating vehicle queue lengths for single-lane left turn movements, the vehicle queue lengths for double left turn lanes can be estimated by dividing the results of these methods by 1.8. This value represents the assumption that queued vehicles will not be evenly distributed between the turn lanes.

Right Turn Movement Queue Estimation Techniques

A similar rule of thumb equation, sometimes referred to as the “red time” formula[8], is also available for signalized single-lane right turn queue estimates. It is represented by the following equation.

Storage Length = (1-G/C) (V) (K) (25-feet) / (Number of Cycles Per Hour) (N_L)

where:

G = Green time provided for the right turn movement
C = cycle length
V = right turning volume
K = random arrival factor

[8]Koepke, F. J., Levinson, H. S., *Access Management Guidelines for Activity Centers*, NCHRP Report 348, TRB, Washington, D.C., 1992, p. 99.

N_L = number of right turn lanes

A value of 2 should be used for the random arrival factor (K) where right-turn-on-red is prohibited. Where right-turn-on-red is allowed, a value of 1.5 should be used.

As with the equation for left turn queue estimates, the value of 25-feet used in the equation represents the average storage length required for a passenger car. If a significant number of trucks are present in the turning volumes, the average storage length per vehicle should be increased in the same manner recommended for the left turn queue estimate using Exhibit 7-29.

Another, less accurate, method for manually estimating vehicle queue lengths is using the assumption that “V” vehicles per hour per lane entering a signalized lane with a cycle length of 90 seconds will produce a “V”-foot-long queue per lane. For example, if the volume turning left from a dual left turn lane is 400 vehicles per hour, a ballpark queue length estimate would be 400/2 = 200 feet per lane.

Computer Software

The use of software in estimating vehicle queue lengths can often be conducted simultaneously with capacity analysis, which can make it a very convenient method. There are many different software programs available that provide queue length estimates. However, caution should be used in selecting one as results may vary significantly between programs. As an example, the HCS has been found to produce consistently poor queue length estimates as compared to field measurements and should not be used for this purpose.

For the estimation of queues at intersections belonging to a coordinated signal system, over-capacity conditions and areas where queue spill-back may be a problem, it is recommended that the SimTraffic simulation software be used to report the 95th percentile queues. Refer to Chapter 8 for further information on SimTraffic.

Whether queue lengths have been calculated through manual methods or computer software, as a general rule-of-thumb the installation of signalized turn lanes with more than 350-feet of storage should be reconsidered through discussions with Region Traffic. In some cases, it may be preferable to install dual turn lanes with shorter storage bays.

7.5.2 Methodologies for Unsignalized Movements

At unsignalized intersections, the movements of interest are often the major street left turns and all minor street movements. The most common methodologies used for estimating queue lengths for these movements include the Highway Capacity Software (HCS)[9], the Two-Minute Rule, the Harmelink Curves[10] and a method published by John T. Gard[11].

---

[9] *Highway Capacity Software,* McTrans, University of Florida, Gainesville, Floriday.
[10] M.D.Harmelink, *Volume Warrants for Left-Turn Storage Lanes at Unsignalized Grade Intersections*, Highway Research Record 211, 1967.
[11] *Estimation of Maximum Queue Lengths at Unsignalized Intersection.* John T. Gard, ITE Journal/November 2001.

TPAU has conducted a study to evaluate the first three of these methodologies for estimating queue lengths. This study, Storage Estimates for Unsignalized Intersections, concluded that while the Two-Minute Rule provided conservative estimates for major street left turns and minor street right turns, it appeared to underestimate queue lengths for minor street left turns and shared left/right and left/through/right lanes. Despite this, the Two-Minute Rule was still found to produce more reliable results than the HCS or Harmelink methods. In particular, the HCS method was found to consistently underestimate queue lengths. This is confirmed in the previously mentioned study by John T. Gard. Therefore the HCS and Harmelink methods shall not be used. Either Simulation or the Two-Minute Rule may be used, until the John T. Gard method has been satisfactorily validated with local data.

Simulation

If simulation is being performed as part of the analysis, queue lengths should be taken from the simulation results. If simulation is not being done, it should be considered. If the effort to do a simulation analysis is not desired, the two-minute rule should be used. If the results of the two-minute rule analysis are deemed unacceptable, the option is to do a simulation analysis.

Two-Minute Rule

The Two-Minute Rule is a rule of thumb methodology that estimates queue lengths for major street left turns and minor street movements by using the queue that would result from a two-minute stoppage of the turning demand volume. This method does not consider the magnitudes and impacts of the conflicting flows on the size of the queue. The calculation of the 95th percentile queue using the two-minute rule methodology shall use the following equation:

$S = (v)\ (t)\ (L)$

where:

> S = the 95th percentile queue storage length (feet)
> v = the average left-turn volume arriving in a 2-minute interval
> t = a variable representing the ability to store all vehicles; usually 1.75 to 2.0 (See Exhibit 7-28.)
> L = average length of the vehicles being stored and the gap between vehicles; 25 ft. for cars. This value can be increased where a significant number of trucks are present in the turning volume using the same relationship between average vehicle storage length and percent trucks in turning volumes shown for the signalized movement rule of thumb method discussed earlier in this chapter.

8 TRAFFIC SIMULATION MODELS

8.1 Purpose

Traffic simulation models are complex tools that can provide valuable information on the performance and potential improvement of transportation systems. Traffic simulation models are in a constant state of improvement and accordingly this chapter attempts to be adaptive with the changes in the industry. This chapter currently presents instruction on calibration of microsimulation models created in Trafficware's SimTraffic and a brief overview of the other simulation models and parameters used in ODOT projects. Topics covered include:

- Traffic Simulation Modeling – General Calibration Instructions
- SimTraffic – Overview and Calibration Instructions
- VISSIM – Overview
- Paramics - Overview
- CORSIM – Overview

8.2 Traffic Simulation Modeling – General Calibration Instructions

Traffic simulation models are computer programs that simulate traffic movements over a user-defined transportation network and present the results via animation and reports. The degree of user control over the simulation and the types of facilities that can be modeled will vary depending on the program being used. These should not be confused with urban travel demand models (Section 4.6), which use current and projected land use and transportation network data to estimate current and future travel demand and traffic patterns.

Traffic simulation models (meso or microscopic) are complex tools that generally require more labor than programs that perform capacity analysis at a macro level. Because of this, they are generally only used when the use of other types of analysis tools will not be adequate for a given project. Simulation models offer a greater degree of flexibility than most programs designed specifically for capacity analysis and can be used for a wide range of analysis needs such as examining the interactions between different modes of transportation, modeling the operations of HOV lanes or bus priority systems and evaluating operations through measures of effectiveness not offered by most other types of analysis programs. Simulation models are also very useful for presentations, especially for those given to audiences lacking technical knowledge of traffic analysis, because it provides a visual basis for evaluating operations that most people can easily relate to and understand.

Simulation models are commonly used by ODOT to analyze corridors or networks under congested conditions, where upstream or downstream operations have a significant influence on actual intersection operations (e.g., intersection blockage from queue spillback). It should be noted that simulation models use different methodologies for estimating queue lengths than other procedures described in this manual. These methodologies are typically based on observations of queues experienced during simulation, which are influenced by parameters such as driver characteristics, lane changing behavior and various traffic flow interactions. Capturing the impact of up and downstream operations on vehicle queues can make these models very effective at estimating queue lengths, but underscores the importance of good model calibration. General guidelines for the application of simulation models have been published by the Federal Highway Administration, which can be found at the FHWA website under traffic analysis tools.

Depending on the specific program used, there may be numerous parameters that can be manipulated by the user to create a system that most accurately represents the one being analyzed. Before any simulation model is used to represent existing or future conditions, the existing conditions model created must be calibrated by adjusting operational parameters until the model provides a reasonable representation of existing conditions measured in the field. Existing conditions need to be replicated; otherwise future conditions will not be correct. Existing conditions should include only data, operations and measures known to currently exist in the project study area. Vehicle counts should be kept as close as possible to the original volumes obtained from the field. If all counts are available from the same day, vehicle counts used during calibration should be un-factored and unbalanced counts (this day should be as close to the 30^{th} highest hour as possible). If counts cannot all be collected on the same day (or year), every effort should be made to collect counts at primary locations on a day that is on or closely represents, the 30^{th} highest hour. The remaining counts can then be factored and balanced to this primary count day. If all counts occur on scattered days and none of the counts occur on the 30^{th}

highest hour or on a representative day then short sample count should be conducted to factor the off- peak counts to the day the study area was visited. Use the seasonal factor methodology described in Section 4.4 to determine if the count is close enough to the 30th highest hour. If the primary counts for the study area occurred during a time that is less than 90% of the 30th highest hour for that area seasonal trend type, then a re-visit with a sample count is required for the calibration of the “existing” model.

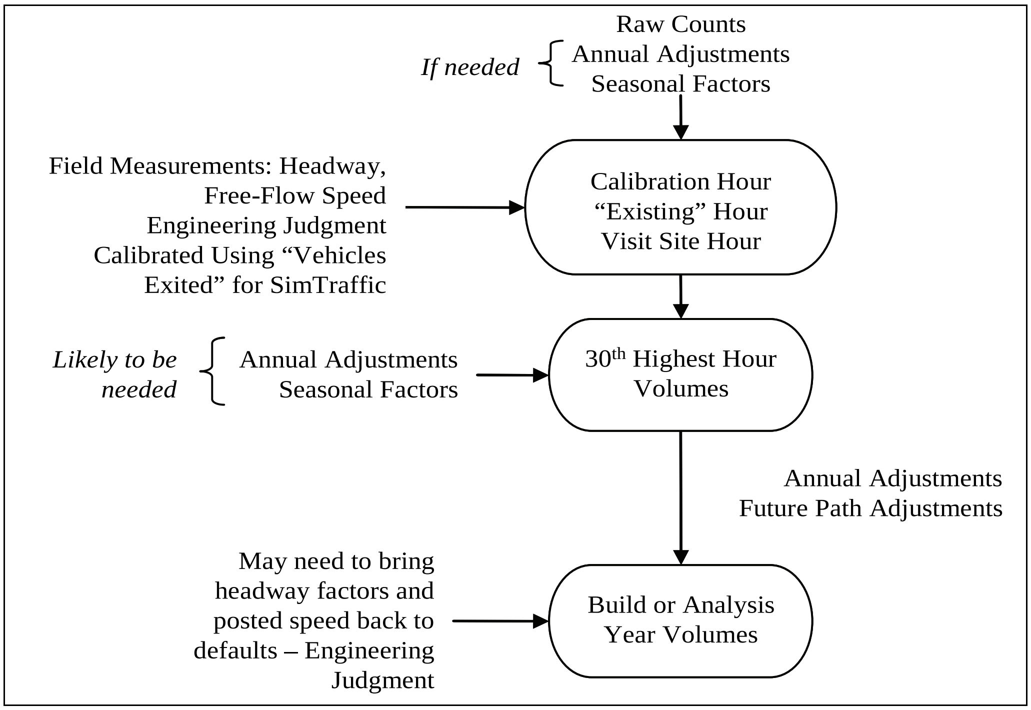
These rules are established to help ensure that calibration volumes 1) are near the 30th highest hour and 2) represent conditions that have been witnessed in the field. The emphasis is placed on witnessed, as the analyst needs to visit the study area on or near the count day (30th highest hour) so that the visual check of the simulation (the first step in calibration) is based on conditions that occurred in the field during the count. The Field Inventory Worksheet, Appendix H, shows all the measures from the field that should be input into the simulation and visually checked in the animation to help analysts in the data collection process. In Chapter 3, Transportation System Inventory, Exhibit 3-2 shows an example completed worksheet for a simulation project. Note that the worksheet is intended to be printed multiple times for a given project area. The collection of worksheets can be placed in a three-ring binder providing a hard writing surface. Each copy of the worksheet can be used for each intersection or area of interest in the study and all copies can be neatly organized in a single project binder (see Exhibit 3-1).

The site visit should occur as close as possible to the 30th highest hour. After the site a calibration scenario can be constructed. For the purpose of calibration, the peak hour volumes from the counts should be seasonally adjusted to the time period of the site visit. The calibration network should include all measurements taken and all operational behavior witnessed. Many of the behavioral issues should be collected on the worksheet provided above. For Synchro and SimTraffic inputs refer to Sections 7.3.9 and 8.3. These sections refer specifically to Synchro/SimTraffic, but the list provided should include most of the measures that would have to be checked or adjusted in any software platform. Note that most microsimulations go into greater detail than SimTraffic, so there will likely be more measures to check and adjust. Also note that illegal behavior such as speeding, improperly using medians or shoulders as turn bays and improper lane changing distances should be accounted for in during calibration, but should not be continued to be assumed in the future build scenarios. All non-calibration alternative analysis should assume that all drivers follow the rules of the road.

Once the “existing” inputs and behavior is coded into the simulation software, the analyst should run an animation to visually check the reasonability of the microsimulation. Any gross error like queues or blockages being much greater or much less than the field observations should be addressed by re-checking inputs. Further refinement may include measuring and adjusting saturation flow rates, driver reaction time and travel speed. A good place to start is by comparing simulated vehicle queues to those visually observed in the field. For some corridors, comparing simulated travel times or average speeds to actual observed conditions may be appropriate.

Good calibration is not only critical for accurate analysis, but will establish credibility during presentations with technical advisory committees or public groups that have prior knowledge of existing problem areas. Exhibit 8-1 illustrates how the calibration process fits into the complete analysis. The calibration, existing and site visit hour refer to the same hour. In other words, the "calibration" data is collected in the study area in the "site visit" hour to represent "existing" conditions. For further information on calibration in general, consult the FHWA Analysis Toolbox. Section 8.3 has the detailed procedures on calibrating a SimTraffic model using SimTraffic for ODOT projects.

Exhibit 8-1 Simulation Construction and Application Flow Chart



8.3 <u>SimTraffic</u>

8.3.1 Overview

SimTraffic performs microsimulation and animation of vehicle traffic, modeling travel through signalized and unsignalized intersections and arterial networks, as well as freeway sections, with cars, trucks, pedestrians and buses. SimTraffic includes the vehicle and driver performance characteristics developed by the Federal Highway Administration for use in traffic modeling. They were developed for CORSIM and Trafficware used them as they were published. Most of the input is entered through the Synchro program, but some parameters, such as the driver and vehicle characteristics, are modified through SimTraffic specifically.

SimTraffic can be used for all ODOT plans, projects and traffic impact studies. SimTraffic is primarily used by ODOT for the analysis of signal systems and vehicle queue estimation, especially in congested areas and locations where queue spillback may be a problem. For the estimation of signalized vehicle queues, SimTraffic is generally preferred in Regions 2 through 5 where v/c ratios exceed 0.70 and in Region 1 where v/c ratios exceed 0.90, but should always be used where v/c ratios exceed 0.90. SimTraffic should typically be used for the analysis of all coordinated signal systems. For isolated intersections, Synchro and SimTraffic should provide similar results. SimTraffic results will differ from Synchro most when the v/c ratio exceeds 0.90, when there are closely spaced intersections and other conditions that are not ideal. Overcapacity queues and metering conditions are identified in Synchro's Timing Window with a "#" or "m" symbol.

8.3.2 Simulation Calibration

As much as possible, operational field data should be obtained for the major facilities in the study area as close as possible to the design hour (see Appendix H). Beyond the field data listed in Section 3.2, additional field measures may be needed to achieve calibration of the microsimulation. If needed, saturation flow studies should be performed at the major intersections. Floating car travel time runs may need to be performed to ensure that observed and simulated travel times (and related speeds) are close. Free-flow link speeds using road-tube counters or speed guns (RADAR, LIDAR, etc) may need to be collected and used in place of posted speed limits during calibration.

At the very least, the existing conditions network needs to be visually calibrated to the field conditions and the "vehicles exited" measure from SimTraffic should be reviewed. If everything is close, then the SimTraffic simulation should duplicate conditions seen in the field. Congested and free-flow areas in the field should be congested and free-flowing in the simulation.

If there is more congestion in the simulation than in the field, then one or more parameters may be off. For example, saturation flows and resulting headway factors may be too low, counts may be balanced too high, peak hour factors may be too low, link and turning speeds may be low, storage bays and taper lengths may be too short and intersection paths and lane change distances may be incorrect. If congestion is too low then the reverse of these may be a cause.

To help determine the cause of inconsistencies with known conditions, any number of measures of effectiveness (MOE) may be reviewed, however as a minimum measure, “ vehicles exited” needs to be checked to ensure that the model is calibrated.

“Vehicles Exited” represents the number of vehicles that make it through an intersection over a given period of time. This should equal the volume coded in the network for the “existing hour”. The calibration target for each intersection in the network is a tolerance of 1% over the analysis period based on the difference between the simulation and the input field-counted exiting (existing) hour volumes. However, at a minimum, the tolerances for any movement over 100 vph should be within 5% of the coded volume. Movements with less than 100 vph should be checked to make sure that the vehicles exiting is reasonable. These limits are required to achieve calibration for the calibration volume set (not required for the 30th highest hour or build year network). Exhibit 8-2 shows an excerpt from the Performance report showing the Vehicle Exited rates and calibration percentages. Note that all movements over 100 vph are under the 5% maximum tolerance and the entire intersection is under the 1% intersection tolerance.

Exhibit 8-2 Example Vehicles Exited from Performance Report

1001: Route 20 & Spring Hill Drive Performance by movement Entire Run

Movement	EBL	EBT	WBT	WBR	SBL	SBR	All
Total Stops	38	353	704	25	298	14	1432
Avg Speed (mph)	12	26	16	23	9	14	19
Vehicles Entered	39	1161	1198	476	317	19	3210
Vehicles Exited	39	1164	1200	476	320	19	3218
Hourly Exit Rate	39	1164	1200	476	320	19	3218
Input Volume	38	1169	1187	489	311	17	3211
% of Volume	103	100	101	97	103	112	100

Although calibration (fine-tuning) may take some time, it is necessary because if the existing conditions is not duplicating observed conditions, then the future conditions or build alternative performance will not be predicted very well. This is critical if any animated output is to be shown at public meetings. In achieving accurate calibration it is important that the SimTraffic parameter file is setup properly.

8.3.3 Simulation Preparation

In addition to setting up the SimTraffic 7 parameter file, there are a number of Synchro settings that must be updated for simulations to work properly in SimTraffic 7. More signal timing detail must be added in the Phasing Window. These phasing details, settings and defaults are shown in Section 7.3.7. Project data needs to be entered into the Simulation Settings Window and the Detector Window. The Detector Window is covered under the Synchro sections in Section 7.3.9 because detector data is necessary if actuated signal functions are to be used in Synchro. The important Simulation Settings Window and the SimTraffic parameter data are included in

Section 8.3.3 and 8.3.4, respectively. Earlier versions of SimTraffic only need to create the SimTraffic parameter file.

8.3.4 Simulation Settings Window

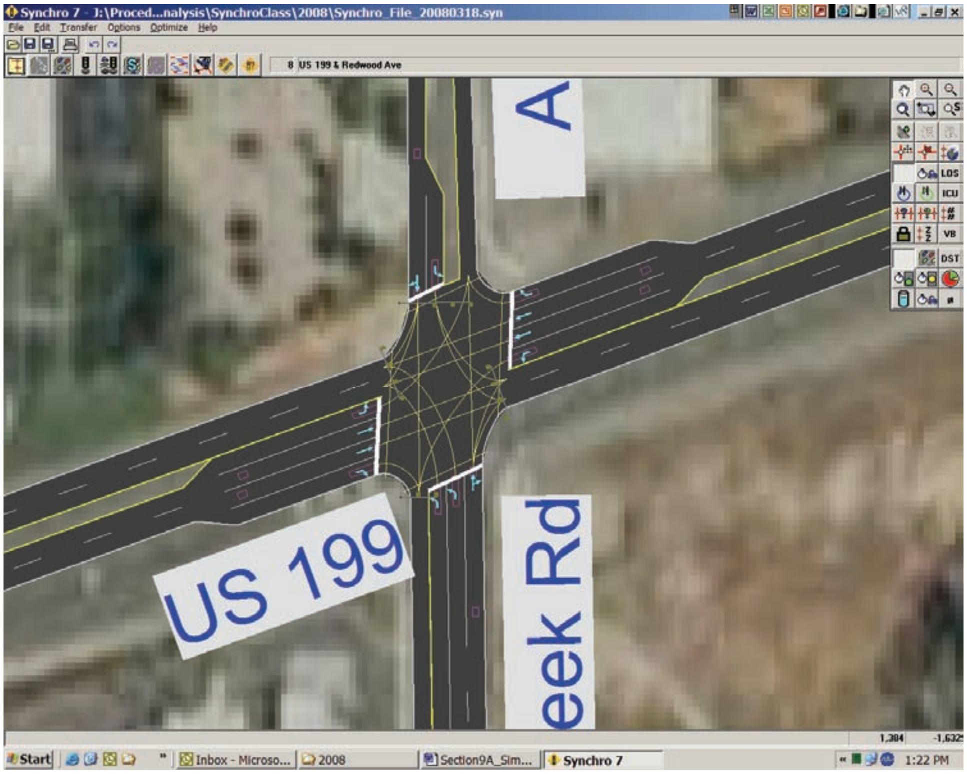
The following data is only used by SimTraffic and needs to be included for a proper simulation. This data allows for geometric refinement and operational behavior of the simulation. The data required by SimTraffic should be a part of the field collection/observation process and is included the “Field Inventory Worksheet” in Appendix H.

- **Storage Length (ft) –** The Storage Length is the length of a turning bay from the stop bar to the beginning of the taper. Storage Length is the area that can store vehicles and does not include tapers. If the Left or Right Turn lane goes all the way back to the previous intersection, enter "0". Storage Length data is used for analyzing potential blocking problems. Storage length is typically field measured or estimated from aerial photographs. If measurements are unknown or if the facility is new, the initial storage lengths of 100’ for urban and 150’ for rural can be used. SimTraffic outputs will be used to refine these lengths for build alternatives.
- **Taper Length (ft) –** The Taper Length is the remaining length of the turning bay from the end of the storage length to where the outer edge of the turning bay meets the outer edge of the adjacent lane. This value is field-measured or estimated from aerial photographs. For state highways, the taper lengths can be obtained from the Highway Design Manual Figures 9-6 for right turn lanes and 9-7 for left turn lanes. This allows turning bays to store several more vehicles and allows a truer and a more consistent (with design) representation.
- **Lane Alignment –** The Lane alignment controls the vehicle paths in SimTraffic. When links are constructed, Synchro shows either a “Left” or “Right” alignment as default. This may not be correct especially if multilane approaches, skewed intersections, short links, free-flow ramp connections and merge/diverge/weaving sections make up a particular intersection.

  Other choices are “L-NA” and “R-NA” which will force the vehicle path either left or right. To check the lane alignment, the Intersection Paths box must be checked under the Map Settings window. The default color or zoom level will likely need to be changed to clearly see the paths.

  Exhibit 8-3 shows that Synchro defaults to single-lane turn lanes turning into a multilane leg with paths going to either departing lane. Unless lines are marked on the pavement guiding vehicles into different lanes Oregon vehicular code states that vehicles need to turn into the nearest lane. In most of these cases the Lane Alignment needs to be changed to “L-NA” or “R-NA” depending on the turn type.

Exhibit 8-3 Default Lane Alignment



For the existing calibrated network, the legal setting may not need to be followed if the majority of field-observed vehicles turn into both lanes (although itself an improper lane choice). Design alternatives should be always be coded legally.

Note that the northbound dual left turn lane shown in Exhibit 8-3 has the correct paths. The southbound left still needs to be changed to limit traffic to the inside through lane. In cases of acceleration lanes, merging traffic should be forced right using “R-NA” and through traffic forced left using “L-NA.” This will keep through vehicles out of the acceleration lane.

- **Enter Blocked Intersection –** This setting controls whether mainline or side-street traffic can enter a blocked intersection. In earlier versions of SimTraffic, vehicles did not block intersections. Default is “No” for intersections and “Yes” for bend nodes and ramp junctions. This factor is best obtained through field observation.

  Along many busy roadways, minor intersections and driveways are frequently blocked by through traffic, so in this case the setting should be “Yes” for the through traffic. If “Do Not Block Intersection” signs exist, then the setting should remain “No” unless the signs are generally ignored. If there are intersections or accesses that are frequently blocked and through vehicles let side street vehicles out, then the side street movements can be set to “1 veh” which will allow one vehicle to enter. Use of the “2 veh” setting has a tendency to cause the simulation to clog up.
- **Link Offset (ft) –** The Link Offset is used to set the roadway left or right of the natural

centerline. This is typically used in creating "dogleg" or offset intersections without creating a second node.

- **Crosswalk Width (ft)** - this is the width of the crosswalk on an approach. This setting controls the placement of the stop bar which controls detector placement and link length. ODOT default crosswalk width is 12 feet (outside edge to outside edge) unless the adjoining sidewalk is wider. Local intersections should be measured.
- **Headway Factor** - The saturation flow rate in SimTraffic for intersection approaches is adjusted through the Headway Factor. The saturated flow rate calculated in Synchro is not used in SimTraffic; however, the corresponding headway factor is automatically calculated. In simulation calibration, the headway factor can be adjusted to help fine-tune (calibrate) the SimTraffic simulation. Exhibit 8-4 shows the equivalent headway factor for a given saturated flow rate. Earlier versions of Synchro/SimTraffic need to have the headway factor manually calculated in the Lane Window.

Exhibit 8-4 Headway Factors

Headway Factor	Saturated Flow Rate
1.2	1650 vphpl
1.1	1750 vphpl
1.0	1850 vphpl
0.9	2050 vphpl
0.8	2250 vphpl

- **Turning Speed (mph) –** This is the turning speed used by SimTraffic by movement. Higher speeds will increase the capacity of the SimTraffic simulation. Synchro default is 15 mph for left turns and 9 mph for right turns. The 9 mph right turn speed is too slow unless used for turning onto residential local streets or in a downtown central business district location.

  ODOT default is 15 mph for left and right turns. Non-standard turns at skewed intersections, channelized turns and interchanges should have different values and can be estimated by recording speeds while driving through the subject intersections or using a speed gun to capture turning vehicle speeds. Turning speeds are also needed for merge/diverge sections at interchanges or bend nodes.
- **Lane Change Distances** - Changes to these calculated values can help calibrate the vehicle lane-changing operation. Changes may be necessary if vehicles are having difficulty completing lane changes ahead of intersections or off-ramps or if vehicles are artificially clogging up at lane drops after an intersection or a two-lane ramp merging into a single lane. High heavy vehicle percentages combined with a higher amount of long vehicles and/or a congested network increases the chances that modifications will be required. Closely spaced intersections will have short lane change distances while interchanges will have longer lane change distances as many drivers move into the desired lane considerably ahead of an off-ramp. The analyst will need to experiment with these values, either longer or shorter until the traffic is flowing consistent to the observed conditions or flowing smoothly for future conditions. Modifying ramp geometry so that

the ramps enter the mainline as turns rather than as a straight-through movement makes for smoother operation and less need to modify these distances.

There are two different types of lane change distances: mandatory and positioning. The Mandatory Distance is the distance measured from the stop bar at which a lane change must occur. The Positioning distance is the distance measured back from the Mandatory Distance where a vehicle first attempts a lane change. The Mandatory and Position Distance 2's are extra distance added if a second lane change is necessary. All of these distances can extend around corners. Adding to the challenge of changing these variables, is that the driver types in SimTraffic have a range of a 50% (aggressive) to a 200% (passive) multiplier to the set distances.

8.3.5 SimTraffic Parameter File

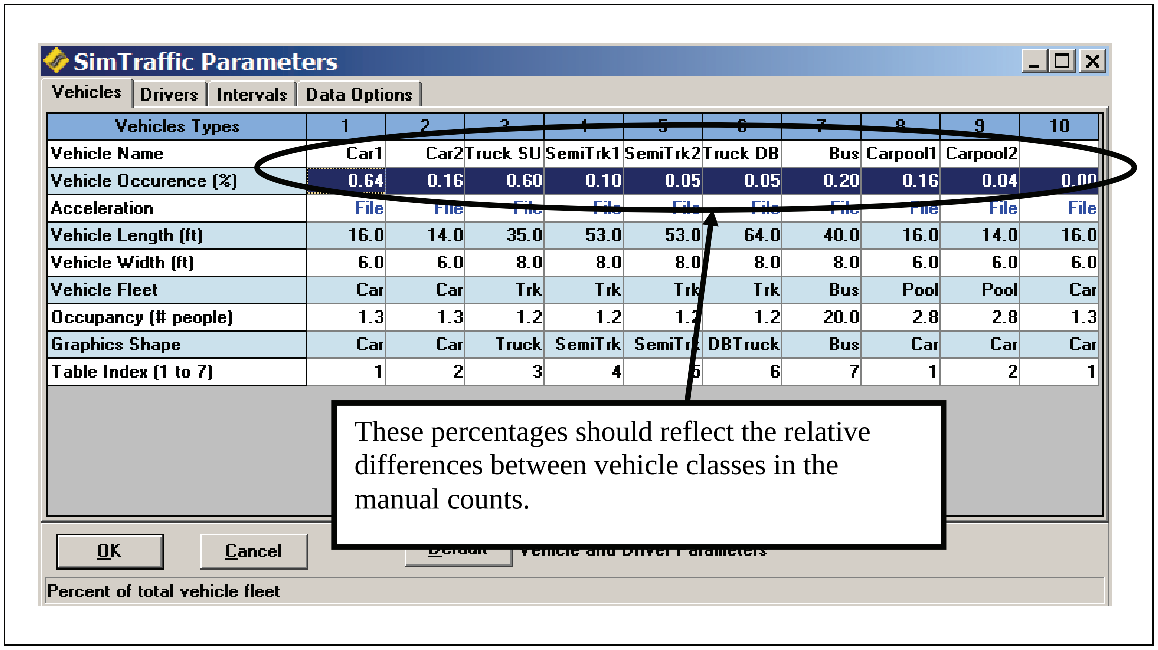
The SimTraffic parameter file controls the simulation operation and the defaults must be changed to reflect the proper impacts of queuing, travel time, etc. The parameter file has three major sections: Vehicles, Drivers and Intervals. The TPAU Analysis Tools webpage has a default SimTraffic template file with all of the basic parameters set up. The following shows the variables that need to be changed. All other settings are left unchanged.

The Vehicles tab controls the type and physical vehicle characteristics.

- **Vehicle Occurrence (%)** - SimTraffic uses the Synchro heavy vehicle percentage to simulate the total number of heavy vehicles relative to all vehicles. When the simulation calls for a heavy vehicle, the vehicle type is represented by this factor which represents the percentage breakout of the global truck fleet. Likewise, when a car is called for, this factor will split the car types among the global car fleet percentages.
  - Earlier versions of SimTraffic defaulted to having the total vehicle percentages sum up to 100%.
  - SimTraffic 7 defaults total up to 100% for the car fleet and 100% for the truck (includes buses) fleet as shown in Exhibit 8-5.
  - Change the Vehicle Occurrence (%) for the different vehicle classes to match the composite average of your classification counts. If classification counts are unavailable, state highway vehicle classification segment data (available at http://highway.odot.state.or.us/cf/highwayreports/traffic_parms.cfm) can be used substituted. Average between multiple counts at the project boundaries and on different significant facilities both state and local. Note that while the heavy vehicle percentages per approach may vary largely, the heavy vehicle mix does not vary as much. The total truck fleet should total up to 100% and the total car fleet should total up to 100%.
    - Car1 represents the larger passenger vehicles in the fleet (i.e. SUV's, large pickups);
    - Car2 represents smaller passenger vehicles in the fleet;
    - TruckSU represents single unit trucks (i.e. delivery vans, dump trucks);
    - SemiTrk1 represents single tractor-trailer combinations;
    - SemiTrk2 represents shorter single tractor-trailer combinations;

- Truck DB represents trucks with two trailers; Note: SemiTrk2 and Truck DB can be customized to fit other truck types like triple trailers.
- Bus represents buses in the fleet;
- Carpool1 & Carpool2 represents vehicles with the same characteristics as Car1 and 2 but with higher occupancies. Zero out the default Carpool1 and Carpool2 vehicles. These will have no effect on the simulation unless vehicle occupancy is used as an evaluation measure.

Exhibit 8-5 SimTraffic Default Vehicle Parameters

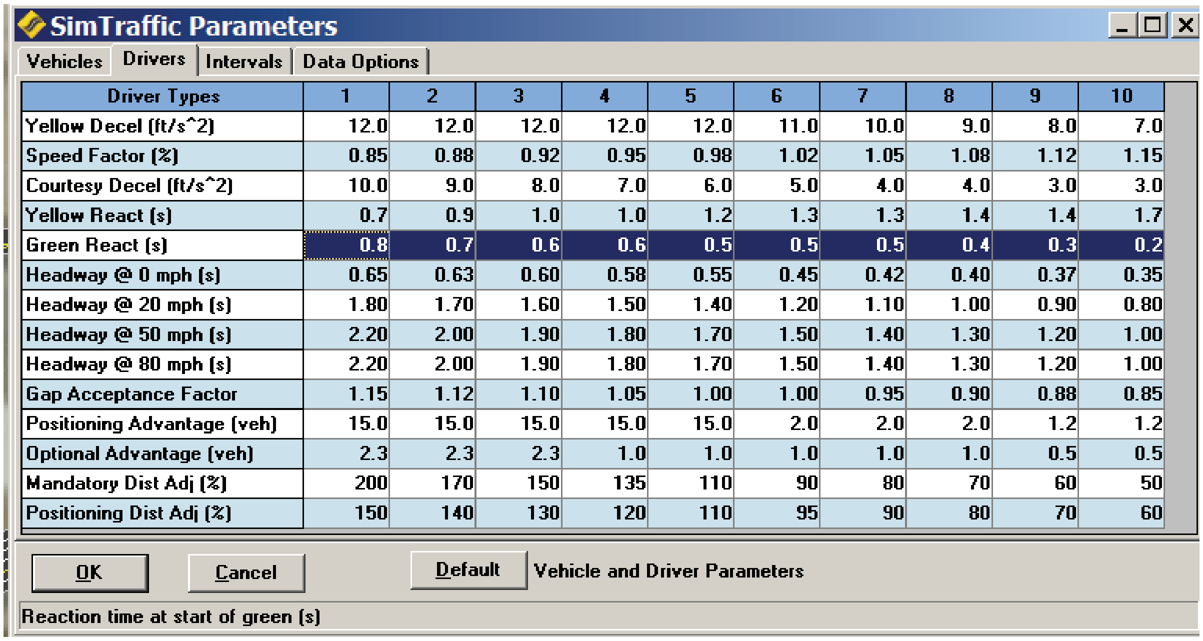


Vehicles Types	1	2	3	4	5	6	7	8	9	10
Vehicle Name	Car1	Car2	Truck SU	SemiTrk1	SemiTrk2	Truck DB	Bus	Carpool1	Carpool2	
Vehicle Occurence (%)	0.64	0.16	0.60	0.10	0.05	0.05	0.20	0.16	0.04	0.00
Acceleration	File	File	File	File	File	File	File	File	File	File
Vehicle Length (ft)	16.0	14.0	35.0	53.0	53.0	64.0	40.0	16.0	14.0	16.0
Vehicle Width (ft)	6.0	6.0	8.0	8.0	8.0	8.0	8.0	6.0	6.0	6.0
Vehicle Fleet	Car	Car	Trk	Trk	Trk	Trk	Bus	Pool	Pool	Car
Occupancy (# people)	1.3	1.3	1.2	1.2	1.2	1.2	20.0	2.8	2.8	1.3
Graphics Shape	Car	Car	Truck	SemiTrk	SemiTrk	DBTruck	Bus	Car	Car	Car
Table Index (1 to 7)	1	2	3	4	5	6	7	1	2	1

- **Vehicle Length (ft) –** This parameter directly affects queuing distances. Leaving the length unchanged will result in the queues being underestimated. Change the vehicle length in the following vehicle types:
  - Car1 = 20 ft;
  - Car2 = 16 ft;
  - TruckSU = 30 ft;
  - SemiTrk1 = 75 ft.

The Drivers tab (Exhibit 8-6) controls the behavior characteristics for the 10 different driver types that make up the simulation from the passive to the aggressive. For example, Driver Type 1 has 15% lower link speeds and will take 200% more distance when making a lane change while Driver Type 10 will travel 15% faster than the link speed and have lane change distances 50% of the coded values. All of the factors in the Drivers tab remain the same with exception of the Green React (s) setting. This setting reflects the time from when the signal turns green to the time that the vehicle begins to move. This value can be captured in the field and used as a calibration parameter. TPAU research indicates that Oregon values are substantially different than the defaults in SimTraffic. Change the Green React times to match Exhibit 8-7.

Exhibit 8-6 SimTraffic Default Driver Parameters



Driver Types	1	2	3	4	5	6	7	8	9	10
Yellow Decel (ft/s^2)	12.0	12.0	12.0	12.0	12.0	11.0	10.0	9.0	8.0	7.0
Speed Factor (%)	0.85	0.88	0.92	0.95	0.98	1.02	1.05	1.08	1.12	1.15
Courtesy Decel (ft/s^2)	10.0	9.0	8.0	7.0	6.0	5.0	4.0	4.0	3.0	3.0
Yellow React (s)	0.7	0.9	1.0	1.0	1.2	1.3	1.3	1.4	1.4	1.7
Green React (s)	0.8	0.7	0.6	0.6	0.5	0.5	0.5	0.4	0.3	0.2
Headway @ 0 mph (s)	0.65	0.63	0.60	0.58	0.55	0.45	0.42	0.40	0.37	0.35
Headway @ 20 mph (s)	1.80	1.70	1.60	1.50	1.40	1.20	1.10	1.00	0.90	0.80
Headway @ 50 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Headway @ 80 mph (s)	2.20	2.00	1.90	1.80	1.70	1.50	1.40	1.30	1.20	1.00
Gap Acceptance Factor	1.15	1.12	1.10	1.05	1.00	1.00	0.95	0.90	0.88	0.85
Positioning Advantage (veh)	15.0	15.0	15.0	15.0	15.0	2.0	2.0	2.0	1.2	1.2
Optional Advantage (veh)	2.3	2.3	2.3	1.0	1.0	1.0	1.0	1.0	0.5	0.5
Mandatory Dist Adj (%)	200	170	150	135	110	90	80	70	60	50
Positioning Dist Adj (%)	150	140	130	120	110	95	90	80	70	60

Exhibit 8-7 ODOT Green React Times

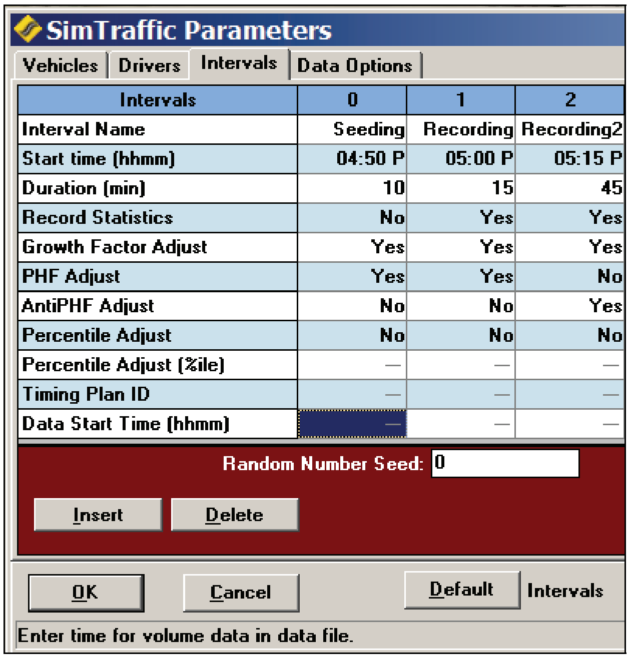
Driver Type	1	2	3	4	5	6	7	8	9	10
Green React (s)	2.0	1.6	1.3	1.1	1.0	0.9	0.9	0.8	0.7	0.5

The Intervals tab controls the actual operation and data recording of the simulation. Exhibit 8-8 shows the ODOT interval defaults.

- **Seeding "0" Interval** – The Seeding Interval fills the network before any statistics are recorded. This value must be long enough for vehicles to travel the length of the network. ODOT default is 10 minutes or the time to travel the longest trip on the network, whichever is longer.
- **Recording Intervals** – Simulation statistics are recorded in these intervals. The ODOT default uses at least two intervals, one 15-minute in length to represent the peak 15-minute period and one 45-minute interval to fill out the hour simulation period. However, you can have more intervals if you would like. For future analysis networks, the 15-minute interval is preferably placed as the first recording interval because it most represents the peaking in the output reports, regardless of where it occurs in the actual peak hour. However, for the calibration network, the 15-minute peak period should be coded to represent the actual peak 15-minute period as it occurred during the counts. The names of the recording intervals can be anything as they have no impact on the results.

- **Duration (min) –** Change to 10 minutes (time to cross the network if longer) for the seeding interval, 15 minutes for the first recording interval and 45 minutes for the second recording interval (or, if this is being applied to the calibration work, a distribution representing the peak as it occurred in the counts).
- **Start Time (hhmm) –** After Duration is specified, change the start time to reflect the hour being simulated.
- **Record Statistics –** Set to "Yes" for all recording intervals.
- **Growth Factor Adjust –** Set to "Yes" for all intervals.
- **PHF Adjust & AntiPHF Adjust –** The combination of these two settings creates a spike in the simulated hour. The PHF Adjust should be set to "Yes" during the seeding and the peak 15-minute intervals and the AntiPHF Adjust set to "No.". The AntiPHF Adjust should be set to "Yes" and the PHF Adjust set to "No" for all other recording intervals.
- **Percentile Adjust** - Set to "No" for all intervals. Use of this setting will overestimate the queuing in the simulation.
- **Random Number Seed –** SimTraffic uses nine different simulation scenarios (1 through 9). If it is desired to produce duplicate results, select a non-zero setting. ODOT default is to set it to '0' which will produce random arrival rates with each run.

Exhibit 8-8 ODOT Intervals Defaults


SimTraffic Parameters

Vehicles | Drivers | Intervals | Data Options

Intervals	0	1	2
Interval Name	Seeding	Recording	Recording2
Start time (hhmm)	04:50 P	05:00 P	05:15 P
Duration (min)	10	15	45
Record Statistics	No	Yes	Yes
Growth Factor Adjust	Yes	Yes	Yes
PHF Adjust	Yes	Yes	No
AntiPHF Adjust	No	No	Yes
Percentile Adjust	No	No	No
Percentile Adjust (%ile)	—	—	—
Timing Plan ID	—	—	—
Data Start Time (hhmm)	—	—	—

Random Number Seed: 0

Insert | Delete

OK | Cancel | Default Intervals

Enter time for volume data in data file.

8.3.6 Simulation Execution

Once all Synchro and SimTraffic settings are completed, the simulation is ready to be executed. Upon starting the simulation, the "Errors and Warnings" window will appear. This shows anything that is outside of the value ranges what SimTraffic expects to find. Errors are split into fatal and non-fatal errors. Fatal errors will not allow the simulation to run and must be corrected. Fatal errors usually are related to lanes and lane groups where no lanes exist on a link.

Non-fatal errors still allow a simulation to be run, but these need to be reviewed and corrected if possible for best results. Some examples of non-fatal errors that need to be corrected are:

- “Detector too close to stop bar” ;
- Minimum green /total split/pedestrian timing errors;
- Reference phase not in use errors;
- Storage lane and length errors.

Some examples of non-fatal errors that can be left alone as these are “how it is” are:

- “Angle between approaches less than 25 degrees.” Small angles will lengthen out an intersection area and may cause unpredictable operation.
- Any error referencing vehicle extensions or minimum gaps exceeding 111% of travel time between detectors. Errors such as these indicate that actuated signal operation will be not as efficient.
- “Volume-delay operation not recommended with long detection zone.” SimTraffic has issues generally with ODOT’s default phasing variables.

ODOT standard is to average together at least five (5) random acceptably working (no system gridlock) runs. If you have a congested or a large network, it is advisable to have 7-10 runs to allow for “blown” runs which are caused by system gridlock so there are at least five good runs averaged together at the end. The system gridlock is typically caused by the improper actions of simulated vehicles that end up getting stuck. If every run or a majority of runs have gridlock, then the analyst should further refine the simulation settings, especially the headway factors, blocked intersection and lane change distance parameters.

It can take 20-40 minutes a run (depending on network size, congestion level and computer speed). Make sure you have adequate available storage. Each simulation file can be in excess of 1 GB. If you run out of space during a multiple recording session, SimTraffic will continue to run, but the simulations will stop being recorded.

Once the runs are completed, check each simulation run by selecting each number in the drop-down run number box to make sure it is free of any system gridlock errors and that the simulation reflects what is expected. If there are bad runs, make note of the run number, so it may be skipped in the report process.

8.3.7 Simulation Outputs

SimTraffic outputs are used for queue analysis, determination of storage lane lengths, travel times and other evaluation criteria. Many times in the evaluation of alternatives the typical v/c and LOS measures may have very small differences. It is common practice today to use additional MOE’s to describe an operation of an alternative. These MOE’s can include travel time, stopped delay, average speed and queue blocking. These are very useful in alternative comparisons because lower travel times, delays and stops coupled with higher average speeds, will indicate a more operationally efficient alternative.

Make sure before selecting any report to print or preview that a number is showing in the run number box. Otherwise, a message appears from SimTraffic saying that it needs to record the simulation again.

To preview a report, select the desired report(s) and make sure that the Multiple Runs box is checked. Select the desired .hst files (skipping any bad runs). Reports are generally broken down into sections by intersection and interval. The Summary of All Intervals section of the report is where information is pulled from for analysis. Check to make sure that content, headers and footers are correct before printing.

- **Simulation Summary Report –** Used to check whether that the runs look to have similar characteristics. Entering and exiting vehicles, total delay and total stops should be relatively consistent between runs. This is a second check of the run adequacy (the first is visual inspection). This report also gives system total MOE's which can be used in alternative comparisons.
- **Queuing and Blocking Report** - The Queuing and Blocking report generates the 95th percentile queues which are used to design turn bay storage as well as document operation of the study area. Exhibit 8-9 shows a typical report.

  This report shows three different queues: maximum, average and 95th. The reported maximum queue is the highest queue calculated every two minutes. The average queue (50th percentile) is the average of the calculated two-minute queues. The 95th Queue is the 95th percentile of the reported maximum queue over the simulated period. With the Random Number Seed set to zero, the queues in this report will be different from those in another set of simulation runs. When reporting out the estimated queue lengths, round up to the next 25 feet.

Exhibit 8-9 Sample Queuing and Blocking Report

Queuing and Blocking Report
Baseline 2/7/2008

Intersection: 2: Redwood Ave & US 199, Interval #1

Movement	WB	WB	NE	NE
Directions Served	L	LT	R	R
Maximum Queue (ft)	49	99	139	132
Average Queue (ft)	11	18	43	48
95th Queue (ft)	61	91	142	167
Link Distance (ft)	592	592	382	382
Upstream Blk Time (%)				
Queuing Penalty (veh)				
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

Intersection: 2: Redwood Ave & US 199, Interval #2

Movement	WB	WB	NE	NE
Directions Served	L	LT	R	R
Maximum Queue (ft)	321	341	120	144
Average Queue (ft)	64	68	16	16
95th Queue (ft)	344	343	75	77
Link Distance (ft)	592	592	382	382
Upstream Blk Time (%)	3	5		
Queuing Penalty (veh)	25	49		
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

95th Percentile Queues

Intersection: 2: Redwood Ave & US 199, All Intervals

Movement	WB	WB	NE	NE
Directions Served	L	LT	R	R
Maximum Queue (ft)	341	374	155	157
Average Queue (ft)	51	56	22	24
95th Queue (ft)	299	300	96	106
Link Distance (ft)	592	592	382	382
Upstream Blk Time (%)	2	4		
Queuing Penalty (veh)	19	37		
Storage Bay Dist (ft)				
Storage Blk Time (%)				
Queuing Penalty (veh)				

The Upstream Block Time and the Storage Block Time are of particular interest in helping describe the overall impact of queuing. While the 95th percentile queues may show how long a queue is, the block time shows for how long of the simulated hour the queue will block intersections or storage bays.

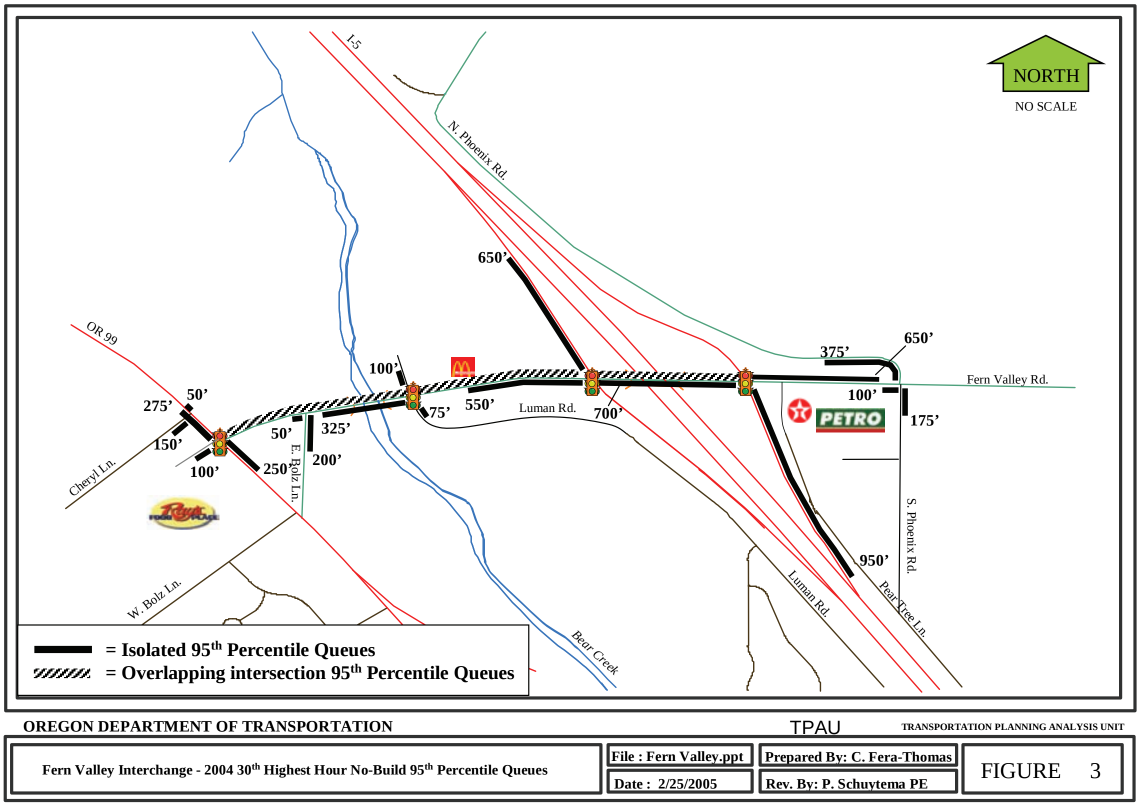
Even if queue spillback into adjacent intersections is not occurring, storage bays may be overflowing, causing local problems such as the blockage of adjacent lanes. A queue blockage or spillback condition is considered a problem when the duration exceeds five (5) percent of the peak hour. Spillback may also be a sign of cycle failure as there may not have been enough green time available to serve all waiting vehicles. Signals do not recover instantly, so one spillback cycle could affect the operation of the next two or three cycles which can be a significant portion of hourly cycles.

- **Upstream Blk (Block) Time (%)** - This is an estimated percentage of the peak hour in which the queue from the subject node blocks an upstream node. This is especially useful when analyzing a complex Synchro network, to determine the extent of queuing on a system when reporting out results. It can also be used to determine if an alternative or option will provide the best progression.
- **Storage Blk (Block) Time (%)** - This reports an estimated percentage of the peak hour in which the length of the through or turning queues exceeds the storage length. For build analysis, if your storage block time is significant (>5%), then it is recommended to enter a longer bay length, rerun the simulation and continue this until you get a percentage less than 5%. Keep in mind that storage bays should adhere to the practical limit of 300 – 350 feet (most storage bays are 100 to 150 feet), so some alternatives and their simulations will still have significant storage block time.

Queues can be reported directly from the subject approach if the queue length is less than the link length. If a queue is longer than the link length, then the total actual queue length will be the link length(s) that are completely filled up plus the last queue length that does not exceed the link length. The analyst will need to trace the queue back from the intersection in question, so you will likely pass though multiple intersections and bend nodes to obtain the actual queue length. However, this queue is made up of contributions from other intersections that the subject queue spills back into which can make it hard to tell and report where exactly the queue originates. Queues are best reported graphically by identifying the queues under spillback conditions separately from the ones that do not exceed the link length. Exhibit 8-10 shows a sample 95th percentile queuing diagram. To minimize reporting issues, link curvature should be used where possible to eliminate any unnecessary bend nodes.

The combination of the upstream and storage block times can also be used to report out the impacts of queuing at a higher level instead of reporting out the 95th percentile queues for intersection approaches.

Exhibit 8-10 Sample Queuing Diagram



- **Performance Report** - The Performance report (Exhibit 8-11) gives the MOE comparisons for each intersection by approach, movement or run; for each approach by run; or a total for the entire network. MOE's are summed over the entire hour (i.e., hours of delay). During calibration, "vehicles exited" needs to be used to ensure calibration, see Section 8.3.1 for more instruction.

Exhibit 8-11 Sample Performance Report

SimTraffic Performance Report
Baseline 2/7/2008

2: Redwood Ave & US 199 Performance by movement Entire Run

Movement	WBL	WBT	NET	NER	All
Total Delay (hr)	2.8	2.6	0.0	1.6	7.0
Delay / Veh (s)	8.6	15.9	1.9	4.1	8.0
Total Stops	69	120	0	58	247
Travel Dist (mi)	147.4	72.6	0.4	112.2	332.5
Travel Time (hr)	6.2	4.3	0.0	4.3	14.8
Avg Speed (mph)	24	17	28	26	23
Fuel Used (gal)	6.4	3.3	0.0	3.8	13.5
HC Emissions (g)	259	121	0	155	534
CO Emissions (g)	8425	3399	11	4823	16658
NOx Emissions (g)	797	385	1	463	1645
Vehicles Entered	1176	597	9	1351	3133
Vehicles Exited	1177	598	9	1353	3137
Hourly Exit Rate	1177	598	9	1353	3137
Input Volume	1420	678	12	1475	3585
% of Volume	83	88	75	92	88
Denied Entry Before	0	0	0	0	0
Denied Entry After	0	0	0	0	0

- **Arterial Report –** The Arterial Report (Exhibit 8-12) is another version of the Performance report but reports out travel time, delay and speed along a roadway section on a per vehicle basis. This roadway must have at least two nodes for this report to be available for it and the roadway has to have the same road name without any special characters (i.e. dashes) along all of the reported sections. The presence of a mixture of one-way and two-way sections along an arterial corridor may require segmenting and the individual results summed.

Exhibit 8-12 Sample Arterial report

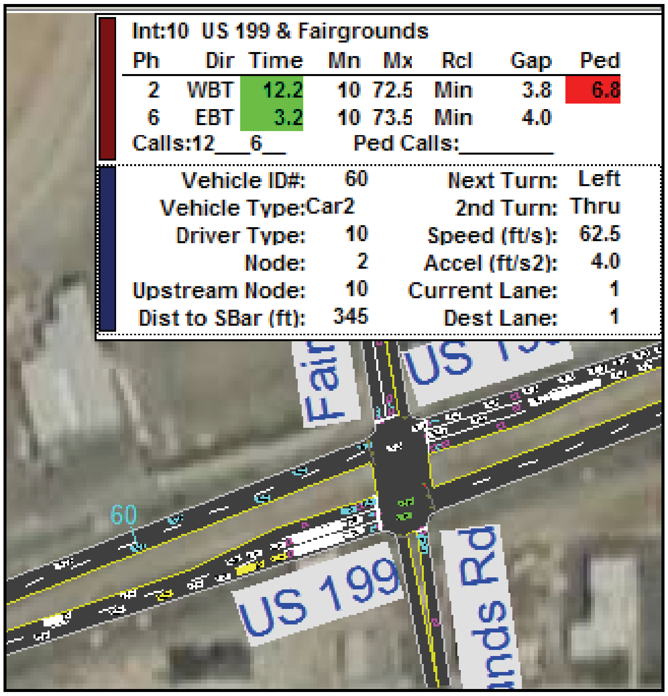
Arterial Level of Service: EB US 199, Entire Run

Cross Street	Node	Delay (s/veh)	Travel time (s)	Dist (mi)	Arterial Speed
Allen Creek Rd	5	18.5	26.2	0.1	12
Redwood Ave	8	8.6	22.5	0.2	27
US 199	2	4.4	11.0	0.1	27
Fairgrounds Rd	10	18.0	28.0	0.1	16
Total		49.5	87.8	0.5	19

Animated Tracking

In the SimTraffic simulation, clicking on a vehicle will bring up a box (Exhibit 8-13) showing speed, acceleration, distance to next turn, etc. this will allow the analyst to track vehicles as they travel through the network. Clicking on the vehicle again will remove the tracking box. In addition, signalized intersections can be clicked on showing the signalized operation in action as it goes through the phases. Both of these can be useful in debugging a simulation. It is recommended that the simulation speed be set to real time or slower for best viewing.

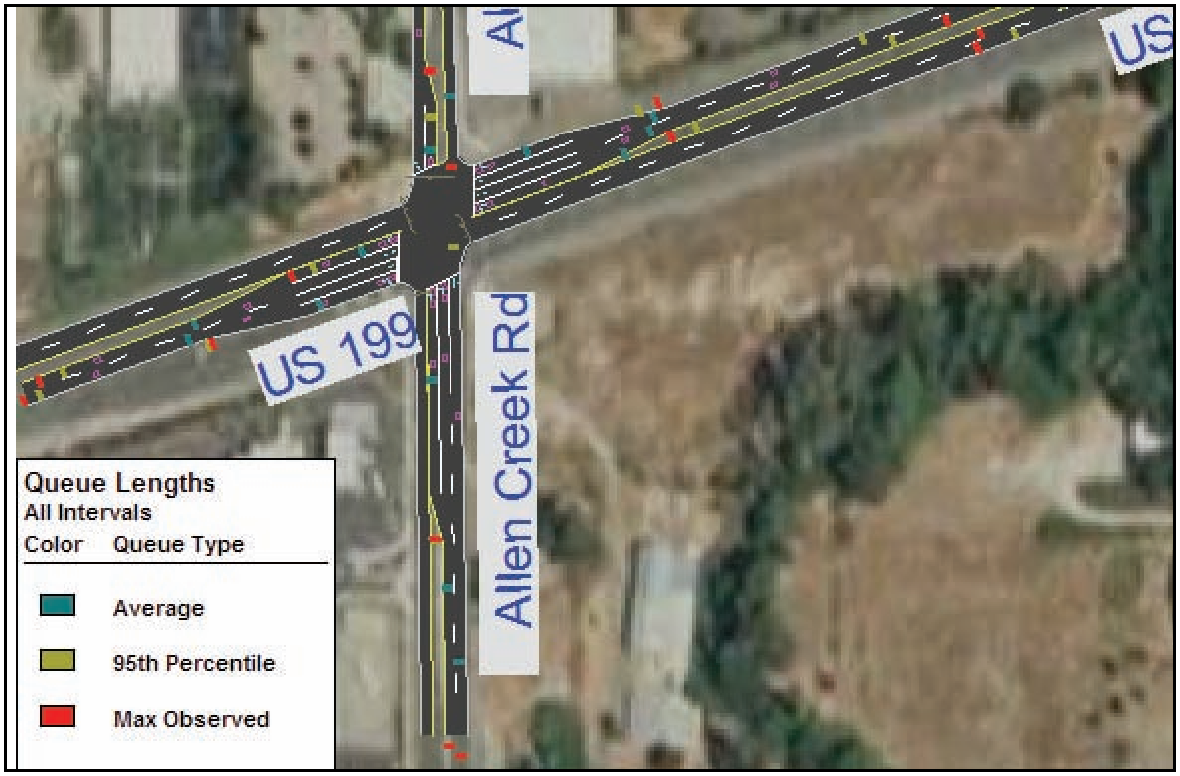
Exhibit 8-13 Animated Vehicle and Signal Tracking



Static Graphics

Other reports include the "Static Graphics" reports (Exhibit 8-14). Select the Graphics tab and you will get a box showing reports such as total delay, percent time blocked queues, etc. These reports are based on the same information that the previous comprehensive reports use, but display the information in graphical form, rather than a table of numbers. These report out just the run number selected rather than an average of runs of the regular reports. These can be use to quickly visualize the issues for the analyst or for others.

Exhibit 8-14 Example Queue Length Static Report



8.4 VISSIM - Overview

VISSIM is a simulation program that can model multi-modal traffic flows including cars, trucks, buses, heavy rail and light rail transit (LRT) as well as model traffic management systems (ramp meters, toll roads and special lanes) and transit priority systems. VISSIM can also model trip assignment, over fixed routes or dynamically, where vehicles change routes in response to specified events and can animate traffic movements in 3-D. VISSIM is a program that can stand alone, but is data intensive to create files for use on its own. Alternatively, the files can be created in VISUM (a travel demand program) that can then import the files into VISSIM for analysis. Because most regions do not perform travel demand modeling, it is important to note issues both with and without VISUM.

Other advantages of VISSIM include the rail-roadway interface, which requires VISSIM Level 3 or 4 in order to model the effect of rail crossing blockages on queues and roadway operations. Another advantage is that VISSIM has the capability of "dynamic traffic assignment" (DTA), which will reroute a vehicle on the network in case of a crossing blockage or an overcapacity situation. Note that this strength of the software comes at the price of larger study areas to allow for correct dynamic assignment and to address effects occurring potentially outside of the focus of the study area. DTA will likely require more data, measures and resources to properly calibrate.

Other VISSIM disadvantages are that it does not produce signal coordination timing and can be data intensive and time consuming to construct and calibrate a scenario, especially from scratch.

When importing a Synchro network to VISSIM through the Pre-Synchro converter or a network from VISUM to VISSIM, note that turn bay lengths cannot extend greater than halfway between adjacent intersections in VISUM. The analyst will need to shorten the turn bays before importing into VISUM and later extend them to the proper length in VISSIM. Impacts (in the travel demand model) from long queues that would be outside a turn lane are not easily discernible, since the turn lanes are modified outside of the demand model program. Some issues to consider when using VISSIM for analysis are (based on ODOT's assessment of version 4.1):

- The flexibility of the program means the analyst is required to write their own reports. This would require exporting the queuing data to Excel; using Excel to calculate the maximum queues for each time period, calculating averages and standard deviations and then calculating the 95th percentile queue on each approach for each run.
- When creating an intersection, VISSIM does not assume the legs are connected together and will assume no turning movements or traffic control until the analyst draws in: all the turn movements, where vehicles slow down when turning the corners, when they need to yield to pedestrians, where the signal heads and detectors are, what the relationships between the signal heads and the detectors are, what the design of the ring/barrier signal controllers are and where queues, speeds or other data is to be measured, etc.
- VISSIM does not do signal coordination/progression, so the network must be exported to Synchro to develop the timing and progression and then imported back into VISSIM. Turn bay limitations with the import process as mentioned above will still apply.
- VISSIM should not be used in a TIS process as there are too many parameters to change

and is likely out of the capable review range of most Region analysts.

- VISSIM has the capability of performing analysis directly on VISUM traffic volume assignments and includes a post-processing function. The results of this type of analysis may be acceptable for certain applications, such as sketch planning and alternative screening. However, for most types of analysis, DHVs are required. The function in VISSIM does not create DHVs, therefore the post-processing procedures outlined in Chapter 4 are still necessary.
- Most ODOT offices do not own the VISSIM software. VISSIM submittals by consultants should include the VISSIM model translated into Synchro files to enable effective ODOT review. The ODOT Synchro defaults should be implemented in the VISSIM model to the extent possible.
- Most region offices are unlikely to have the knowledge base to use VISUM.

Currently, VISSIM is not practical enough for most ODOT applications. Model development is data intensive, requires detailed knowledge on many input parameters and has limited standardized output reports. Use of Synchro or VISUM can speed up some of the work but requires an import/export process which can be very time consuming.

8.5 Paramics - Overview

Paramics and VISSIM share a lot of the same benefits in functionality and issues with complexity and time to achieve calibration. Paramics, like VISSIM, is a simulation program that can model multi-modal traffic flows including cars, trucks, buses, heavy rail and light rail transit (LRT) as well as model traffic management systems (ramp meters, toll roads and special lanes) and transit priority systems. Paramics can also model trip assignment, over fixed routes or dynamically, where vehicles change routes in response to specified events and can animate traffic movements in 3-D. Paramics is a program that can stand alone, but is data intensive to create files for use on its own. Paramics does offer some importing functionality to bring networks in from other software, but it does not have a direct link to VISUM. However, all of Paramics' inputs are text files, making it easy to customize automations (macros, scripts, etc.) to take networks from other platforms and format the data into the text files Paramics requires. This creates many opportunities to bring networks from any software quickly into Paramics.

Arguably the biggest strength of any dynamic assignment software (like Paramics and VISSIM) is the "dynamic traffic assignment" (DTA) option, which will reroute a vehicle on the network in case of a rail crossing blockage or an overcapacity situation. Note that this strength of the software comes at the price of larger study areas to allow for correct dynamic assignment and to address effects occurring potentially outside of the focus of the study area. DTA will likely require more data, measures and resources to properly calibrate.

Paramics has disadvantages similar to VISSIM since it does not produce signal coordination timing and can be very data intensive and time consuming to construct and calibrate a scenario, especially from scratch.

Some issues to consider when using Paramics for analysis are (based on ODOT's assessment of version 5.2):

- The flexibility of Paramics means the analyst is required, in most cases, to write their own reports. Paramics does have a set of limited standardized reports. This would require exporting the queuing data to Excel or comparable software; creating functions to calculate the maximum queues for each time period, calculating averages and standard deviations and then calculating the 95th percentile queue on each approach for each run.
- Paramics does not do signal coordination/progression, so the network must be constructed in Synchro (or similar software) to develop the timing and progression, which can then be incorporated into Paramics.
- Paramics should not be used in a TIS process as there are too many parameters to change and is likely out of the capable review range of most Region analysts.
- To date, none of the ODOT offices own the Paramics software. Paramics submittals by consultants should include the Paramics model translated into Synchro files to enable effective ODOT review. The ODOT Synchro defaults should be implemented in the Paramics model to the extent possible.

Currently, Paramics is not practical enough for most ODOT applications. Model development is data intensive, requires detailed knowledge on many input parameters and has limited standardized output reports. The use of Paramics input text file format can speed up some of the work but requires a custom import/export process which can be very time consuming to develop.

8.6 CORSIM - Overview

CORSIM is a microscopic traffic simulation program, applicable to surface streets, freeways and integrated networks with a complete selection of control devices, i.e., stop/yield sign, traffic signals and ramp metering. CORSIM simulates traffic and traffic control systems using commonly accepted vehicle and driver behavior models and combines two traffic simulation models: NETSIM for surface streets and FRESIM for freeways. CORSIM allows for user control of trip assignment through the ability to set vehicle-type specific turn percentages and set predefined vehicles routes.

9 DETERMINING NEEDS

9.1 Purpose

The primary purpose for conducting the analysis presented in previous chapters is to determine how a given facility performs relative to the selected performance measures of the study. This chapter presents an overview of the process for comparing the results of the Existing and No-Build analysis with adopted OHP standards, in order to identify deficiencies in the performance of the facility. Solutions are addressed in Chapter 10. Topics covered include:

- Standards for Determining Needs
- Applicable Oregon Highway Standards
- Analysis of Transportation Systems

9.2 Standards for Determining Needs

The term 'need' as used by transportation professionals is defined as:

> "A 'need' has generally been defined by transportation analysts as any case where the current or planned facility conditions falls below an established standard."

The above perspective assumes that the adopted standard is the minimum acceptable condition for a facility and any case where conditions fall below that level is a deficiency that should be corrected. The relevant standards presented in the latest version of the Oregon Highway Plan should be considered, as discussed in Chapter 2. Standards provide a critical element of the decision-making framework for assessing deficiencies and improvement alternatives since they are developed to maximize overall system performance while limiting liability to the agency responsible for construction, operations and maintenance.

Selection of Performance Measures

Performance measures, sometimes referred to as measures of effectiveness, are quantitative criteria that indicate how well a function or activity is being performed. Some common performance measures used in traffic engineering include v/c ratio, LOS, vehicle delay, travel time, emissions, vehicle speed, mode shift and capacity.

Most road authorities (state, county or city) maintain adopted standards for operational efficiency that identify specific performance measures. It is important to identify all applicable standards and corresponding performance measures for study roadways to provide a basis for evaluating the results of transportation analysis and to determine if project goals and objectives are being achieved. The use of performance measures to identify needs and evaluate alternatives is discussed further in Chapters 9 and 10. ODOT measures highway mobility performance through volume to capacity (v/c) ratios and has adopted separate standards for identifying current and future needs and project design.

Operational standards for identification of current and future needs are documented in the 1999 OHP in Policy 1F. Tables 6 and 7 within Policy 1F list maximum allowable v/c ratios for various combinations of highway classifications and surrounding land uses, with Table 7 applying to the Metro Area and Table 6 applying to the remainder of the state. However, it should be noted that the text within Policy 1F contains exceptions to the standards listed in these tables and, therefore, must be consulted as well. Furthermore, the OHP Registry of Amendments webpage should be checked for amendments that may affect this policy.

As an example, Amendment 00-04, which was adopted on December 13, 2000, created alternate mobility standards for the South Medford Interchange and the Metro Area. These alternate standards can be found in the document "Amendment to 1999 Oregon Highway Plan Alternate Highway Mobility Standards Metro Area." When using these standards, it should be noted that there is an error in the Table 7 footnote. The existing first bullet under OHP Table 7 was a leftover from the original Table 7 and is proposed to be stricken from the OHP with the next revision. Each of the hours needs to be analyzed separately, using an appropriate PHF, with the results compared to the respective v/c ratios provided in Table 7.

These standards are applicable to existing, future no-build and future build conditions for TISs (typically associated with comprehensive plan amendments, zone changes, development reviews and approach applications) and all no-build alternative work for existing and future conditions analyzed in other types of projects including transportation facility projects, transportation system plans, corridor plans, refinement plans, interchange area management plans and access management plans. In situations where an interchange or interstate freeway needs to be modified in association with proposed development impacts, it is necessary to coordinate with Federal Highway Administration (FHWA) and the developer to work out any issues relative to the OHP versus ODOT's HDM guidelines.

Operational standards for project design are documented in Exhibit 10-1 of ODOT's HDM. These standards (the functional equivalents of the LOS standards in the American Association of State Highway and Transportation Officials [AASHTO] Green Book) represent the level of operation for which state facilities are expected to be designed and are intended to be applied to an analysis year occurring 20 years beyond the year of completion. These standards are applicable to future build alternatives associated with all project types except Traffic Impact Studies associated with development, unless an interchange or interstate freeway is involved. It should be noted that for ramp terminals, the HDM mainline maximum v/c ratio is the standard that applies. There is no equivalent ramp terminal v/c ratio in the OHP as there is in the HDM.

Exhibit 9-1 illustrates the appropriate sources of performance measures for different project types.

Exhibit 9-1 Sources of Performance Measures by Project Type

	TIS	Projects	Studies
Existing Conditions	OHP	OHP	OHP
Future No-Build	OHP	OHP	OHP
Future Build(s)	OHP	HDM	HDM

9.3 <u>Applicable Oregon Highway Standards</u>

9.3.1 Mobility

The OHP establishes the mobility standards for all state facilities. ODOT measures highway mobility performance through v/c ratios and has adopted separate standards for identifying current and future needs and project design.

Most of the analysis procedures summarized in Exhibit 2-1 have direct (or equivalent) v/c ratio results for performance assessment. The compliance with the appropriate standard (maximum v/c ratio thresholds defined in the OHP) is the first tier of the evaluation. These procedures are noted in Exhibit 9-2.

Exhibit 9-2 Types of Performance Measures Applications

Type of Analysis	Volume to Capacity Ratio	Meets / Does Not Meet	Speed	Queue Length
Signalized Capacity	X			
Unsignalized Capacity	X			
Preliminary Signal Warrants		X		
Segment Analysis	X			
Signal Warrants		X		
Turn Lane Criteria Analysis		X		
Queuing Analysis				X
Segment Analysis	X		X	
Progression Analysis			X	
Weaving Analysis	X		X	
Merge/Diverge Analysis	X		X	
Passing/Climbing Lanes	X		X	
Simulation Modeling	X		X	
Arterial Analysis			X	

However, several procedures do not yield v/c ratio outcomes. For example, traffic signal warrants are one guide to assess the readiness of an intersection or junction to be controlled by signals, but it is not, by itself, a performance indicator. However, these analyses are useful to flag potential modifications in traffic controls or facility designs that should be incorporated into Build scenario evaluations.

The other category of performance measures focuses on travel speeds, including progression analysis, arterial analysis and selected outputs of many simulation models. The vehicle speed outcomes can be compared to target or design speeds to assess relative benefit, but there is no direct comparison with v/c ratio in these analyses. It is recommended that these types of measures should be used in conjunction with either intersection or segment analysis that do have v/c ratio related outcomes to determine the compliance with mobility standards.

9.3.2 Safety

The safety evaluations parameters are less discrete compared to mobility standards and generally rely on a comparative evaluation to other state facilities as a basis for acceptability. Section 5.2 of the Oregon State Highway Crash Rate Table states:

> *"Table II presents a five-year comparison of crash rates for the state highway system, for urban and rural areas by functional classification."*

For the crash analysis, use this table to compare the historical segment crash rate for a studied section to the statewide average rate in the table for a comparable type. The analyst must determine if the studied segment is within an urban or rural area, the roadway classification and whether it is a state primary or secondary highway. A listing of primary and secondary highways is included after Table IV in the Crash Rate Table. Note that the category "State Highway System" provided alongside the primary and secondary system categories is a combination that should NOT be used for most crash rate comparisons.

Exhibit 9-3 2008 Crash Rates by Jurisdiction and Functional Classification

JURISDICTION AND FUNCTIONAL CLASSIFICATION	MILES	ANNUAL VEHICLE MILES	CRASHES	FATALI-TIES	CRASH RATE*	FATALITY RATE*
TOTAL STATE HWY SYSTEM	**7,453.23**	**19,523,091,729**	**16,142**	**221**	**0.83**	**1.13**
Interstate Freeways	730.52	8,526,366,378	3,169	38	0.37	0.45
Other Fwys/Expressways	54.27	1,290,552,234	858	8	0.66	0.62
Non-Freeways (combined)	6,668.44	9,706,173,117	12,115	175	1.25	1.80
Other Principal Arterials	3,280.79	7,509,225,541	9,631	115	1.28	1.53
Minor Arterials	1,959.83	1,811,486,662	2,031	44	1.12	2.43
Urban Collectors	8.69	10,172,238	11	0	1.08	0.00
Rural Major Collectors	1,381.52	371,721,968	439	16	1.18	4.30
Rural Minor Collectors	34.72	3,432,935	3	0	0.87	0.00
Rural Local	2.89	133,773	0	0	0.00	0.00
URBAN HWY SYSTEM	**819.67**	**9,207,412,773**	**10,054**	**61**	**1.09**	**0.66**
Interstate Freeways	176.15	4,445,167,356	2,066	16	0.46	0.36
Other Fwys/Expressways	54.27	1,290,552,234	858	8	0.66	0.62
Non-Freeways (combined)	589.25	3,471,693,183	7,130	37	2.05	1.07
Other Principal Arterials	512.59	3,163,978,720	6,584	34	2.08	1.07
Minor Arterials	67.97	297,542,225	535	3	1.80	1.01
Urban Collectors	8.69	10,172,238	11	0	1.08	0.00
Urban Cities	**568.62**	**6,973,941,364**	**8,497**	**48**	**1.22**	**0.69**
Interstate Freeways	111.61	3,256,667,634	1,733	13	0.53	0.40
Other Fwys/Expressways	47.73	1,184,858,022	794	6	0.67	0.51
Non-Freeways (combined)	409.28	2,532,415,708	5,970	29	2.36	1.15
Other Principal Arterials	366.57	2,337,353,812	5,527	26	2.36	1.11
Minor Arterials	41.06	192,668,622	440	3	2.28	1.56
Urban Collectors	1.65	2,393,274	3	0	1.25	0.00
Suburban Areas	**251.05**	**2,233,471,409**	**1,557**	**13**	**0.70**	**0.58**
Interstate Freeways	64.54	1,188,499,722	333	3	0.28	0.25
Other Fwys/Expressways	6.54	105,694,212	64	2	0.61	1.89
Non-Freeways (combined)	179.97	939,277,475	1,160	8	1.23	0.85
Other Principal Arterials	146.02	826,624,908	1,057	8	1.28	0.97
Minor Arterials	26.91	104,873,603	95	0	0.91	0.00
Urban Collectors	7.04	7,778,964	8	0	1.03	0.00
RURAL HWY SYSTEM	**6,633.56**	**10,315,678,956**	**6,088**	**160**	**0.59**	**1.55**
Interstate Freeways	554.37	4,081,199,022	1,103	22	0.27	0.54
Non-Freeways (combined)	6,079.19	6,234,479,934	4,985	138	0.80	2.21
Other Principal Arterials	2,768.20	4,345,246,821	3,047	81	0.70	1.86
Minor Arterials	1,891.86	1,513,944,437	1,496	41	0.99	2.71
Rural Major Collectors	1,381.52	371,721,968	439	16	1.18	4.30
Rural Minor Collectors	34.72	3,432,935	3	0	0.87	0.00
Rural Local	2.89	133,773	0	0	0.00	0.00
Rural Cities	**218.91**	**491,825,707**	**536**	**0**	**1.09**	**0.00**
Interstate Freeways	14.05	89,782,362	26	0	0.29	0.00
Non-Freeways (combined)	204.86	402,043,345	510	0	1.27	0.00
Other Principal Arterials	109.77	271,934,779	321	0	1.18	0.00
Minor Arterials	53.24	92,629,842	148	0	1.60	0.00
Rural Major Collectors	41.60	37,222,524	41	0	1.10	0.00
Rural Minor Collectors	0.25	256,200	0	0	0.00	0.00
Rural Areas	**6,414.65**	**9,823,853,249**	**5,552**	**160**	**0.57**	**1.63**
Interstate Freeways	540.32	3,991,416,660	1,077	22	0.27	0.55
Non-Freeways (combined)	5,874.33	5,832,436,589	4,475	138	0.77	2.37
Other Principal Arterials	2,658.43	4,073,312,042	2,726	81	0.67	1.99
Minor Arterials	1,838.62	1,421,314,595	1,348	41	0.95	2.88
Rural Major Collectors	1,339.92	334,499,444	398	16	1.19	4.78
Rural Minor Collectors	34.47	3,176,735	3	0	0.94	0.00
Rural Local	2.89	133,773	0	0	0.00	0.00

* Crash Rate Formula: ((crashes*1,000,000)/VMT); Fatality Rate Formula: ((deaths*100,000,000)/VMT)

When comparing a statewide average rate to a segment crash rate for a study highway, simply exceeding the statewide average rate should not be interpreted as proof that a section is hazardous. Much like an intersection crash rate of 1.0 or greater, a segment crash rate that exceeds the statewide average crash rate should merely be considered as an indication that further investigation is necessary. The analyst should also examine the collision type and collision information such as time of day, milepost, roadway conditions and other factors to more accurately understand the crash history.

9.4 Analysis of Transportation System

9.4.1 Existing System

The analysis scoping, selecting performance measures and procedures for evaluating the existing transportation system are described in Chapters 2, 3, 5, 6, 7 and 8 of this manual. Refer to those sections for appropriate methods and techniques.

Elements of the existing transportation system that do not fall below current adopted performance standards should be flagged for consideration in developing facility alternatives. See Chapter 10.

Similarly the crash analysis procedures are described in Chapter 5. Locations that fall above the statewide average for a similar facility type and setting should be flagged for possible countermeasures or other improvements to be incorporated into the build plan alternatives. See Chapter 10.

9.4.2 Future No-Build System

The Future No-Build System typically includes the same street and intersection network, traffic controls and operational assumptions that were applied for the Existing System analysis without any improvement. In some cases, the Future No-Build System may include improvement projects that are assumed to be funded and constructed within the project planning horizon. The analyst should coordinate with Region Planning staff to identify these projects. Typically such projects would be listed in the STIP, city or county TSPs or MPO Regional Transportation Plans (RTPs). In these cases, it may be more useful to refer to this situation as the Future Base scenario, to reduce confusion with suggestion that no-build implies no improvement projects.

The same measures and analysis techniques applied for the Existing Transportation System will be applied on the Future No-Build System. However, the forecasted future volumes will be used in this analysis to assess how the future No-Build System operates. The future volumes should be developed according to the guidelines described in Chapter 4.

Elements of the transportation system that fall below current adopted performance standards should be flagged for consideration in developing facility alternatives. See Chapter 10.

There is no widely accepted method for assessing future traffic safety conditions. The detailed type of analysis used in Chapter 5 is not applied to future year traffic volumes. However, some project alternatives may help to resolve existing safety issues or deficiencies by upgrading substandard designs (modernization) or eliminating the primary conflicts (e.g., constructing a grade-separated crossing).

9.4.3 Travel Demand Management Options

The future analysis may also include elements that modify the initial travel demand that are expected in the future no-build forecasts. There are many techniques and programs that effectively manage future traffic demands, both on a temporal and modal basis, to work towards reducing the overall travel demands within the project area. Common demand management techniques could include:

- Proposed changes to the current land use zoning.
- Restrictions to the intensity of development within an existing zone (e.g., trip caps).
- Increase or enhanced transit services.
- Comprehensive Travel Demand Management (TDM) programs applied to larger employment centers that increase auto occupancy, bus ridership and help to spread out the peak demand levels for a given site.

It is recommended that the alternatives development process give consideration to TDM components that can augment physical or operational improvements within the study area. Refer to Chapter 10 for more details about TDM options.

10 ANALYZING ALTERNATIVES

10.1 Purpose

The project alternatives should be developed and their effectiveness analyzed consistent with the goals and evaluation criteria selected for the project and to specifically address deficiencies identified through the Existing and No-Build System analysis. This chapter presents the process for conducting the transportation analysis of Build Alternatives. Topics covered include:

- Highway Design Manual Guidelines
- Screening Preliminary Alternatives
- Identifying Limitations to Design Concepts
- Documentation of Screening Process
- Evaluating Build Alternatives

10.2 Highway Design Manual Guidelines

The performance measures applied to flag deficiencies in the Existing or No-Build system, as described in Chapter 2, provide a basis for requiring improvements. However, when defining the scope and nature of improvements, these indicators are not sufficient. The project design guidelines identified in the Highway Design Manual should also be applied to measure acceptability of performance for the horizon forecast year. Refer to Chapter 2 for more detail.

The HDM has different design guidelines for different roadways and the expectation is that the guidelines will be followed. In some cases, however, the costs and impacts associated with a preferred improvement project are too great to fully comply with HDM guidelines and an exception to the design must be submitted and approved. Design exceptions are not intended as a commonplace occurrence, are not necessarily a quick process and should not be relied on prior to approval. Design exceptions may be needed for planning studies. Corridor studies are usually not developed at a level of detail that involves design exceptions. Transportation Growth Management (TGM) funded projects and refinement plans may have enough detail and information that would support design exception requests. As with normal project development projects, complete background information and sufficient justification as to why the guideline was unable to be met must be provided or be available to initiate the design exception process.

For a project that may be constructed within five years, the planner or project leader in charge of the planning project should contact the Region Technical Services Resource Manager (TSRM) to assist in putting together the design exception request. The design exception request should be processed in the same manner as a project development design exception, which is listed in Section 13.3 of the HDM. For projects that may be constructed within five to ten years, the design exceptions should be identified and the TSRM or the Roadway Engineering Manager should give an indication that a design exception is warranted and would probably be approved.

10.3 Screening Preliminary Alternatives

Alternatives for facilities should be developed, assessed and evaluated relative to the matrix of performance measures selected for this study. Depending on the scope and complexity of the study, it may be appropriate to have a tiered screening process. This process would begin with a screening process that allows for a large range of potential alternatives to be defined (typically through a workshop or open house process). This enables many stakeholders to express any outstanding concerns and potential solutions at a sketch or concept level format. These initial sketch alternatives are then filtered to just a few alternatives through the first screening process. These alternatives would then be advanced to the next level in order to select the best candidates for the purposes of alternative performance evaluations. Alternatives that are screened out should be documented as to why and tracked in the project files. This helps document the entire project selection process as well as reference to answer questions about alternative development.

Projects that have an up-to-date travel demand model representation of the study area could use this tool to rapidly perform initial assessments of system performance without the need for detailed analytical calculations required for the full performance measures evaluation. These initial assessments typically focus on more general performance indicators, such as v/c ratios on arterials and highways, v/c ratios across screenlines or approach volumes at major intersections and junctions. These findings can be useful for quickly assessing the general feasibility of a preliminary improvement concept and provide a basis for eliminating or further refining an initial concept.

10.3.1 Coordination with Stakeholders

The development of potential improvement alternatives should be done in cooperation with any groups within ODOT or other agencies that will be involved in the design, implementation, construction, maintenance or operations of the facilities. The district and regional units within ODOT that may be contacted during this process are listed in Chapter 2.

ODOT Engineers

Typically, the highway design and traffic operations engineers within ODOT have a key role in assisting the review and confirmation of the selected alternatives. The district or regional staff that would be responsible for the design and implementation of the selected alternative should be included in the concept development, performance assessment and suggested for further refinements.

Local Agencies

The local authorities for affected roadways, other than the state, should be included in the selection and review of alternatives. Typically this includes local cities, counties or regional metropolitan planning organizations.

ODOT Rail Division

The Rail Division, which is based in Salem, has jurisdiction over railroad crossings and traffic control devices used within crossing areas. They also have exclusive legal authority over public grade crossings and provide coordination with the railroads for affected private rail crossings.

The Rail Division should be contacted any time a project will have an impact directly to or within 500 feet of a railroad or rail crossing.

10.3.2 Potential Facility Solutions

Potential solutions to address existing or future deficiencies can range the following categories:

- Travel Demand Management TDM
- Potential Land Use or Regulatory Changes
- Access Control and Local Circulation Improvements
- Transportation System Management (TSM)
- Capacity Increases
- Intersection Control Improvements
- Interchanges

In general, the analyst should first consider the least impact to existing development, natural systems and cost, then progress towards improvements that have potentially larger investments and associated impacts until the identified need is resolved.

Travel Demand Management (TDM)
The initial assessment for the project area should consider solutions that do not require physical improvements to the transportation system. Travel demand management generally includes the following types of programs and services that can marginally reduce the estimated travel demand where these types of programs are not in place. In general, these types of programs are most suitable for urban areas where commute traffic represents a significant component of the study period flows. In general, they include:

- Carpooling/Ridesharing
- Shuttle Service/Transit Service Expansion
- Transit Fare Subsidies
- Flextime/Compressed Work Week
- Bike Parking/On-Site Lockers and Showers
- Telecommuting

The effectiveness of these types of programs can be estimated based on surveys conducted for the Employee Commute Options Rule compliance. Typically, these measures can reduce commute travel demand for a given activity center by 1 to 10 percent or more, if the management takes aggressive measures. For more details, refer to the 1996 study[12] that assessed the marginal reduction in traffic generation associated with various TDM options.

Potential Land Use or Regulatory Changes
In addition, other planning actions taken by the local jurisdiction may have substantial effects on

[12] Guidance for Estimating Trip Reductions From Commute Options, Oregon Department of Environmental Quality, August 1996.

the initial horizon year forecasts that would reduce the future demand and partially (or fully) mitigate the identified need. These actions could include:

- Re-zoning land to allow less intense transportation uses.
- Restricting the intensity allowed within the current zoning by imposing trip caps that are regulated by local ordinance.
- Supporting mixed use development that minimizes trips onto the roadway system.

These actions require coordination with local agencies that are responsible for land use review and approval and it may require a separate review and approval process to be implemented.

Access Control and Local Circulation Improvements
State facilities should be reviewed to compare background access provisions on state highways against the adopted standards as presented in OAR 734-051. Consolidating (or eliminating) existing vehicular access can substantially improve travel speeds and reduce vehicle conflicts along the highway. Typically, this would require coordination with affected property owners and implementation of necessary permits and easements to effect an alternative local circulation plan. This approach is most effective on a site that is making development application and has substandard existing access spacing provisions.

In addition, the local agency could implement alternative local circulation plans that reduce the volume of traffic using the highway and shifts a portion of the local vehicle trips onto local roadway facilities. This can be accomplished through connecting circulation routes within adjoining uses across parking lots or via alleys, frontage roads and backage roads.

Transportation System Management (TSM)
Substandard performance at highway intersections can be addressed by adding capacity to critical movements or upgrading the traffic control schemes to serve higher demand levels. These types of improvements are also discussed further in Chapter 7. The progression of potential solutions includes:

- **Reconfiguring Lanes:** This involves revising existing lane designations. An example would be revising a two lane approach, where you have a shared left/through lane and an exclusive right turn lane into an exclusive left turn lane and a shared through/right lane. This may or may not involve phasing changes at a signalized intersection.
- **Signal Phasing:** This involves signal phasing changes such as adding a right turn overlap or adding a u-turn.
- **Added Turn Lane Without Widening:** An example would be converting available shoulder or parking space for use as a turn lane.

Capacity Increases

<u>Added Turn Lane</u>
Review right and left-turn lane warrants to serve higher peak period demands. A good planning-level threshold is when turning volumes exceed roughly 150 to 200 vehicles per hour, a turn lane should be considered as an option. If the volumes satisfy warrants, review the intersection

geometry to determine if improvements are required on the receiving side of the intersection to adequately serve the extra approach lane.

For example, a second left turn lane on one approach will require two lanes exiting the intersection for receiving the turning volumes. Another example that can be less intuitive is when a left turn lane is suggested, the opposite side should also be considered for a turn lane since the cross-section on the receiving side needs to be widened anyway to align the through lanes.

Furthermore, the corridor needs of extra lanes between intersections may necessitate widening of the highway to add travel lanes to reduce merge/diverge and weaving issues between intersections. This is particularly the case in urban areas with closely spaced intersections. The approach and departure lanes at major intersections may dictate the cross-section of the highway between these major junctions.

- **Single Left or Right Turn Lane**: Typically a single left or right turn lane can carry about 300 vehicles per hour when intersecting another major cross-section. Higher volumes typically have major vehicle queue spillback and delay issues.
- **Dual Left or Right Turning Lanes** (at intersections):Typically a dual left or right turning lanes at an intersection can carry up to 500 vehicles per hour. When forecasted volumes exceed this level, analysis of alternative solutions is needed. Alternative solutions may include improved adjacent accesses, better connecting linkages, interchange and signal phasing adjustments.
- **Triple Left Turn Lanes**: When it starts to become apparent that dual left turn lanes are not sufficient to accommodate volumes, a grade separation should be considered as opposed to triple left turn lanes. Triple left turn lanes require a long run-out length of six-lane highway. ODOT presently has no triple left turn lanes.
- **Channelized Right Turn Lanes**: When an exclusive right-turn lane volume approaches or exceeds 1,000 vehicles per hour and is not controlled by a traffic signal, the intersection can be modified to provide an exclusive receiving lane that requires no merging with other movements. This results in a free-flow movement with no conflict points.
- **Excessive Intersection Size**: When the width of an intersection leg starts to exceed approximately 110 feet curb to curb, further widening results in diminishing returns in terms of additional capacity, due to longer pedestrian crossing times and other factors.

<u>Added Through Lane</u>

The addition of travel lanes on a highway facility may be appropriate to serve forecasted travel demands. As noted in the previous section, within urban areas the cross-section requirements of the highway may be influenced by the approach and departure lane requirements at the major intersections. Outside of urban areas, added through lanes may be needed to serve forecasted long-range growth in nearby communities or to reduce delays associated with trucks climbing extended grades. The limits of the recommended widening improvements should consider operational performance, study area intersections and the appropriate transition lengths back to the existing highway cross-section.

Intersection Control Improvements

All-Way Stop Controls

If the side street approach to the highway carries roughly the same volume as the highway, an all-way stop control may be appropriate to reduce delays on the minor streets in cases where the existing controls are stop signs on the minor approaches only. However, this solution should consider freight volume levels and any functional designations for priority freight movement on the highway. An all-way stop control is not recommended when freight movement is a priority, since it adds recurring delays on the highway regardless of volume levels.

Roundabouts

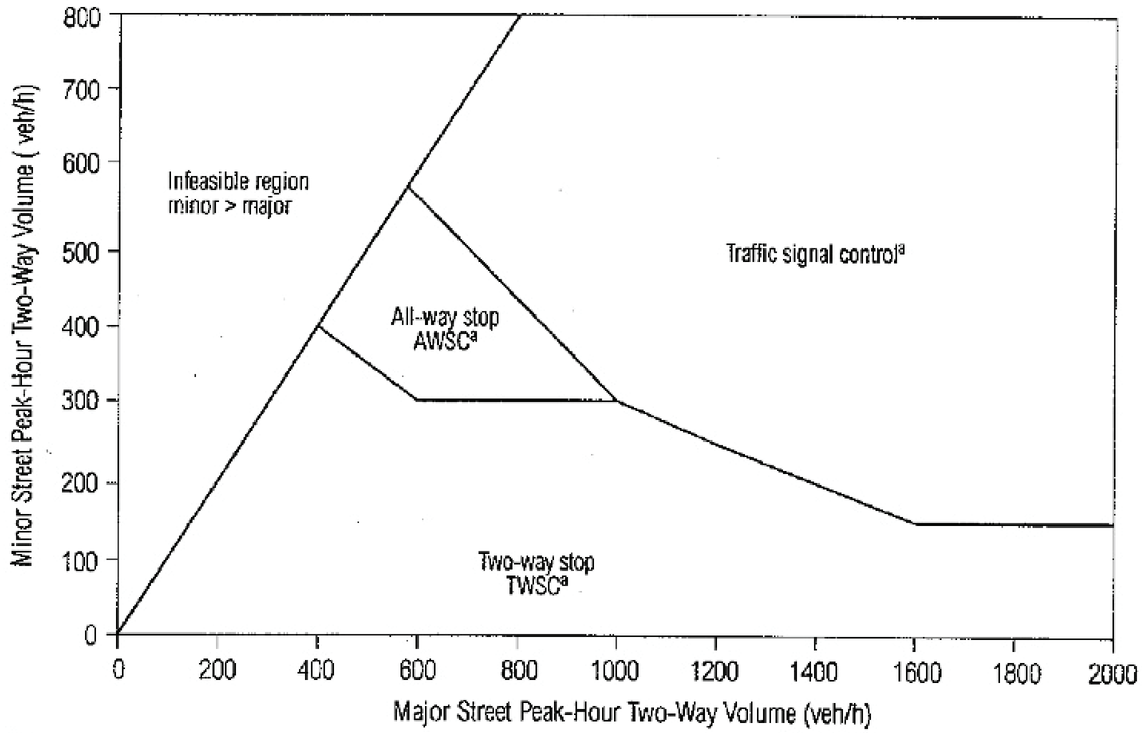
ODOT guidelines for consideration of siting roundabout facilities on state highways are contained in the Traffic Manual and the HDM. Currently the HDM criteria includes the following:

- Should not have more than four approach legs.
- Should meet acceptable v/c ratios for the proposed design life.
- Should have posted speeds 35 mph or less.
- Should have normal circular geometry.
- Should have similar or balanced volumes on all approach legs.
- Should be at an intersection of two highways with roughly the same functional classification or no more than one level of difference (arterial to arterial, to collector).
- Should be mostly commuter and local traffic.
- Should not have high pedestrian volumes.
- Should not have high volumes of large trucks.
- Should not be located within an interconnected signal system.
- Should not be in locations where exiting vehicles would be interrupted by queues from signals, railroads, drawbridges, ramp meters or by operational problems created by left turns, accesses, etc.
- Should not be located where grades or topography limit visibility or greatly complicate construction.
- Should be at an intersection of roughly the same functional classification or no more than one level of difference (arterial to arterial, collector to collector or arterial to collector, e.g., avoid arterial to local street etc.).

Traffic Signal Controls

The ODOT standard signal warrant analysis is required to justify new signal installations. Issues to be considered include traffic volumes, freight volumes, pedestrian volumes, safety history and spacing relative to existing signal and the accepted standards for the highway facility. A general guideline for the appropriate type of intersection controls is presented in the HCM, Exhibit 10-15. A facsimile of that diagram is shown in Figure 10-1. As shown, the two-way vehicle volumes on the minor and major street facilities can be used to quickly determine possible traffic control schemes, ranging from two-way stop controls up to traffic signal controls. It is acknowledged that in some cases a roundabout installation may be an alternative solution to be considered.

Exhibit 10-1 Intersection Traffic Control Options



Notes

a. Roundabouts may be appropriate within portion of these ranges.

Source: Adapted from *Traffic Control Devices Handbook* (8, pp. 4–18) - peak-direction, 8-h warrants converted to two-way peak-hour volumes assuming ADT equals twice the 8-h volume and peak hour is 10 percent of daily. Two-way volumes assumed to be 150 percent of peak-direction volume.

Interchanges

Interchanges on highways are appropriate on all freeway facilities and most expressway facilities to reduce conflicts and to give priority to through movements on the state facility. ODOT and FHWA policies govern the different levels of interchanges which may be considered depending on whether a facility is an interstate, a non-interstate freeway or an expressway. For example, partial directional interchanges could be considered on expressways, but generally not on interstate freeways, although there may be specific locations where a partial directional interchange would be an appropriate treatment that would need to be approved by FHWA. In addition, some arterial locations may have grade-separated solutions when volume demands exceed the levels that can reasonably be served by an at-grade intersection.

When traffic volumes exceed these levels or if the functional integrity of the facility requires it, an interchange or grade-separated junction should be considered. This could take the form of an interchange or it could be a series of overcrossings on parallel routes to reduce the demands on the major arterials to a level that could be served by at-grade facilities.

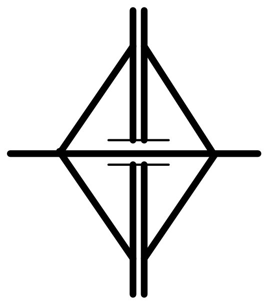
Grade-separated configurations should be developed to serve the forecasted travel demands consistent with the layout and spacing standards recommended in the HDM. Refer to that manual for more specific details that are useful in laying out interchange concepts. The following is a short review of the common elements of an interchange and a discussion of the conventional layout configurations that could be considered during alternative development:

<u>Ramp Types</u>

- **Jughandle Ramps**: These ramps are generally used at low-level interchanges, not for freeway connections and are characterized by low speeds. They may be considered at major private approaches to a state highway. When used for non-interchange at-grade intersections they are termed connections as opposed to ramps.
- **Diagonal Ramps**: The carrying capacity of a ramp is determined by the conflicting movements at the ramp terminals. Typically a single lane straight ramp can carry 1,500 to 1,800 vehicles per hour.
- **Loop Ramps**: Typically a single lane loop ramp can carry 1,200 to 1,500 vehicles per hour. A loop ramp is appropriate to reduce left turning volumes at ramp terminal intersections. As noted above, when left turning volumes exceed 500 vehicles per hour, the typical at-grade intersection cannot generally accommodate it. For example, if a highway approach to a freeway interchange forecasted 700 left turns in the peak hour onto a freeway on-ramp, in most cases, the v/c ratio at this intersection would exceed guidelines. One solution would be to add a loop ramp so that this traffic demand could turn right at the intersection, in advance of the signal and loop onto the freeway rather than making a left turn, which requires a major share of the intersection capacity. On-loops are generally preferred over off-loops, because of concerns regarding the speed differential between the off-loop and the mainline and difficulties encountered on off loops during adverse weather conditions.
- **Directional Ramps**: A directional ramp always bends toward the desired direction of travel. These are free-flow non-looping ramps that generally operate at high speeds. A semi-directional ramp exits a road in a direction opposite from the desired direction of travel, but then turns toward the desired direction of travel. Many “flyover ramps” (as in a stack) are semi-directional.

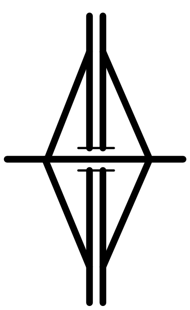
Interchange Types

Exhibit 10-2 Diamond Interchange



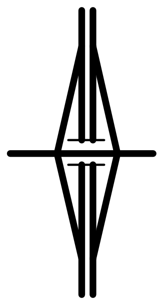
Diamond Interchange: An interchange that has straight ramps in all four quadrants is referred to as a diamond-shaped interchange. The capacity of this facility is typically determined by the operational analysis at the ramp terminals and merge/diverge areas on the mainline. The spacing of the intersections on the crossing street or highway will dictate the available vehicle storage and transition area. A standard diamond interchange has ramp terminal spacing greater than 800 feet. When volume forecasts are high at the terminal intersections and the spacing is limited, these could be factors that influence the need for an alternative layout concept. An operational analysis of the two ramp terminal intersections and any nearby intersections that could influence these locations, will be required. Some variations on the diamond interchange include:

Exhibit 10-3 Compressed Diamond Interchange



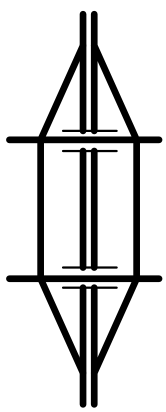
Compressed Diamond Interchange: A typically older interchange design where less than ideal ramp terminal spacing is present, between 400 and 800 feet. Sometimes the two ramp terminals can be operated with a single signal controller. Turn storage is done between the ramp terminals. Queue spillback between the ramp terminals is a common problem.

Exhibit 10-4 Tight Diamond Interchange



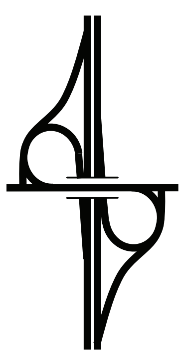
Tight Diamond Interchange: Typically found in urban areas, with ramp terminal spacing less than 400 feet. Usually the two ramp terminals can be operated with a single signal controller. Turn storage is done outside of the ramp terminals.

Exhibit 10-5 Split Diamond Interchange



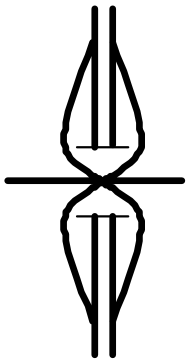
Split Diamond Interchange: Typically found on an urban grid system. Connections between each "half" of the interchange are one-way and are access-controlled.

Exhibit 10-6 Folded Diamond Interchange



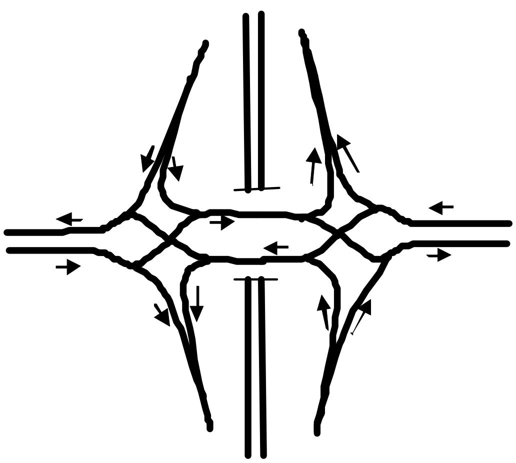
Folded Diamond Interchange: This interchange type "folds" one or two legs of the configuration to minimize impacts in one or two quadrants. Loop ramps can be located where topographical or environmental constraints adjacent to the interchange site do not favor the use of conventional straight ramps, e.g., where a railroad parallels the facility. Loop ramps that are located on the same side of a facility can create weaving sections on the mainline or crossroad that may not be desirable.

Exhibit 10-7 Single Point Urban Interchange



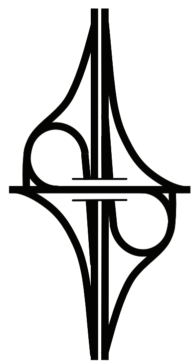
Single Point Urban Interchange (SPUI) also known as Single Point Urban Diamond (SPUD): The SPUI is a relatively recent development that evolved out of the need to limit ROW acquisition in built-up urban areas. SPUIs are a variation of the diamond interchange, which has two ramp terminals with the local arterial. A SPUI combines those two ramp terminal intersections into one larger intersection so that all turning movement to or from the freeway utilize the same intersection. This feature resolves the queue spillback issue that can congest standard diamond intersections, and can be effective in serving high volumes of turning vehicle traffic. SPUI's need cross-street angles close to 90 degrees. High volume right turns may need to be signalized. SPUI's have nearly the same ROW costs as tight diamonds and the structure costs are often high.

Exhibit 10-8 Divergent Diamond



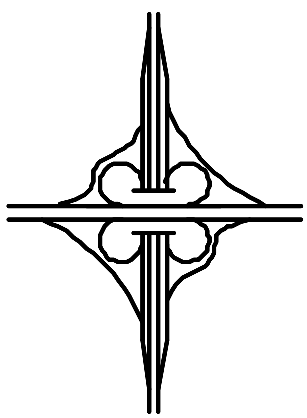
Divergent Diamond Interchange: This is a new type of interchange design that has very few installations in the U.S. This form of diamond interchange has the two directions of minor street traffic cross to the opposite side of the roadway under/over structure. This allows for two-phase signal operations since the left turns occur between the two signals in such a way that they do not cross the opposing through movements.

Exhibit 10-9 Partial Cloverleaf Interchange



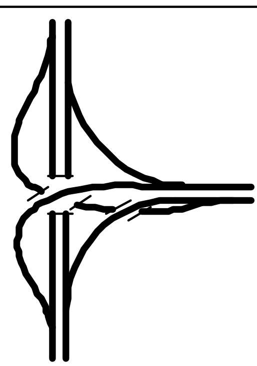
Partial Cloverleaf Interchange: A partial cloverleaf layout combines loop ramps and straight ramps to better serve areas with expected high turning volumes at the ramp terminals. In general, a partial cloverleaf configuration has a higher carrying capacity than a diamond interchange or folded diamond because it has more than four ramps, often providing two on-ramps in each direction as shown in the diagram. The preferred configuration is where loop ramps are located in opposite quadrants of the interchange. Loop ramps can also be recommended where topographical or environmental constraints adjacent to the interchange site do not favor the use of conventional straight ramps, e.g., where a railroad parallels the facility. Loop ramps that are located on the same side of a facility can create weaving sections on the mainline that may not be desirable.

Exhibit 10-10 Full Cloverleaf



Full Cloverleaf: This layout provides loop ramps in all four quadrants of the interchange, requiring a great deal of land area. It is a somewhat outdated design and should typically be used only where loop volumes are low. Loop ramps that are located on the same side of a facility can create weaving sections on the mainline or crossroad that may not be desirable.

Exhibit 10-11 Directional Interchange



Directional Interchange: This type of interchange is more common in urban areas or at junctions of freeways or expressways with other freeways or expressways. An example would be I-5 at I-205. They are high speed high volume connections with all free flow movements. There are configurations with full or partial trumpet or flyover.

10.4 Identifying Limitations to Design Concepts

The facility design concepts are initially selected based on their ability to meet the needs of future travel demands, but each alternative must further balance those project features against the environmental constraints found at that location. A planning study should provide sufficient preliminary information about a range of environmental and physical constraints that could complicate or preclude a particular solution. Environmental criteria should be established as part of the project's evaluation and selection process. Environmental impacts may be allowed only if there are no other feasible alternatives. The analyst should coordinate with the Environmental Program Manager on these issues.

The typical environmental and physical issues to be considered include the following:

- **Exclusive Farm Use (EFU) Lands**: State regulations are very restrictive about the nature of highway improvements that are allowed within these lands. In general, no facility improvements are allowed that add capacity to serve nearby urban areas. Limited safety improvements are acceptable.
- **Environmentally Sensitive Zones**: Proximity of fish bearing streams, open space, riparian zone, etc., requires substantial setbacks from any improvements. In environmental parlance these are known as "4F" zones and may include historic sites, parks and other recreational properties, schools and cemeteries.
- **Built Environment**: Existing buildings and structures generally should not be disturbed, unless the owner is a willing seller and they can be purchased as a part of the improvement project. This requires consideration of historic buildings, schools, hospitals, parks, large developments, low income areas and environmental justice issues.
- **Right of Way**: In general, improvements should be limited to minimize right of way impacts. Acquisition of additional right-of-way adds costs and may not be feasible in some locations.
- **Alternative Modes**: Depending on the functional designation of the highway facility and the adjoining land development there may be need to service pedestrians and bicycles in all solutions under consideration. Alternative concepts that create adverse conditions for non-auto travel, in these cases, will not be acceptable.

Based on the review of the above issues, the alternatives considered for evaluation may be modified or dismissed if any of these areas have substantial issues. An example case would be where the preferred operational solution for a freeway interchange indicated that a partial cloverleaf layout was superior, but because of proximity to EFU land the available configuration space was too constrained. The best solution to meet both the performance objectives and the environmental limitations was a tight diamond configuration.

10.5 <u>Documentation of Screening Process</u>

The alternatives analysis for potential improvement projects should be consistent with the established evaluation criteria.

10.5.1 Evaluation Criteria

The screening criteria should be readily assessable, without detailed evaluations. Examples include:

- Meets project purpose and needs.
- Meets project goals and objectives.
- Compliance with access spacing standards.
- Consistency with agency design guidelines.
- Avoid potential environmental impacts?
- Does the project impact adjacent private properties?

A screening matrix should be developed and applied to all the sketch level concepts and those alternatives that clearly do not meet these basic criteria should be dropped from further consideration. Other alternatives should be advanced to the broader assessment of operational performance analysis, project refinement and preliminary cost estimates, as appropriate.

10.5.2 Alternatives No Longer Considered

As the project advances through alternative development to project design, the process that was applied to develop alternatives should be documented to carry forward into an environmental review document. It is important to describe any initial alternatives that were developed and set aside from further consideration (based on the evaluation criteria) for this purpose. These discarded alternatives should be included in the Alternatives Considered but Dismissed appendix in the narrative report.

10.6 Evaluating Build Alternatives

A Build Alternative refers to any combination of proposed or potential facility improvements to the current transportation system within the study area. The evaluation of Build Alternatives is compared to the No-Build scenario to quantitatively compare relative performance benefits of the various alternatives.

The alternatives selected for evaluation should be reviewed to determine if new model forecasts (or new manual traffic forecasts) are required to reasonably represent the traffic flow conditions with the proposed improvements. For larger study areas, typically a travel demand model is the best tool for evaluating changes in travel patterns associated with potential system improvements and access management plans. However, in smaller studies these changes can be reasonably represented by making manual re-assignments of travel demand, assuming sufficient background volume and travel pattern data is available.

10.6.1 Analysis of Future Conditions

The future conditions analysis should develop quantitative results sufficient to respond to all the selected performance measures for the study. Performance evaluation criteria typically include one or more of the following indicators. Refer to Chapter 5, 6 and 7 for details on how to make these assessments.

- **Volume-to-Capacity Ratio**: This could apply to individual turning movements, average intersection conditions for all movements, roadway or highway segments, weaving movements and highway merge/diverge operations. This is the primary performance evaluation criterion for ODOT facilities.
- **Level of Service**: Many local jurisdictions use Level of Service ratings in their development code as performance criteria. Most facility evaluation methods provide both a v/c ratio result and a Level of Service result.
- **95% Queue Length**: Safety and operational impacts associated with the likelihood of a vehicle queue frequently blocking circulation or access. Use the 95^{th} percentile queue and compare to storage length.
- **Queue Blocking Percentage**: Generally applied to through travel lanes, this is the portion of the study period (percent of time) where standing queues block the advance of vehicles from the adjoining upstream intersections or block the entrance to turn lanes.
- **Other indicators Include**: Travel time, total delay and total number of vehicle stops.

The evaluations for each alternative should assess all of the selected performance criteria. The results can be used to quantitatively compare and contrast the outcomes between alternative and No-Build and each of the respective alternatives to determine the best performing solution.

Analysis Assumptions Relative to No-Build Scenario

Typically, the horizon year travel demand forecast used for the no-build scenario should be applied for each build scenario unless it is determined that the Build scenario would alter the future forecasts for that alternative. For example, where the no-build scenario is heavily capacity constrained, it is likely that diverted traffic will return in the build scenario. If a model is available, both scenarios would be modeled separately. There are two major aspects to consider in making the new travel forecasts: the effects on travel demand and any reasonable changes to the network or operating parameters.

- **Travel Demand Issues**: One outcome of the new travel forecasts may be higher overall volumes on a facility compared to the no-build scenario. This is a common result in a highly congested corridor where a share of existing trips use parallel routes and when sufficient capacity is provided nearby, the trips will be re-assigned to the new facility. Typically travel demand model assignments consider the total travel times between the beginning and end of a trip. When new routes are added with shorter travel times, the model compensates by assigning more trips to the improved facility. For a smaller study area, the total travel demand within the system remains constant, but the locally assigned traffic volumes may be re-distributed. This is a common outcome for most projects.

  In a larger regional system, the latent demand for travel that was constrained by corridors with severe delays during commute hours can experience changes in both travel mode and time-of-day when new facilities are introduced. The net result is a higher total travel demand compared to no-build. For example, if a new interstate bridge were constructed across the Columbia River between Portland and Vancouver, several changes to the no-build demand forecast would occur. First, the number of commute bus trips would likely decrease as more travelers opted to drive to take advantage of faster travel times. Second, because the peak travel times would be shorter, more commuters would leave their home closer to the start of their work shift. The combination of these factors would dampen the effectiveness of the new bridge facility because of higher total vehicle trips and more vehicle trips during the peak hour.

- **Network and Operational Issues**: Care should be taken to consider network or access changes that would substantially change the no-build forecasted volumes on the build network. For example, if the build alternative includes a parallel street extension, major access closure, traffic control change or other action that could re-route traffic flows from one facility to another or one access point to another within the study area, these adjustments should be made before re-evaluating performance. These types of changes indicate the no-build forecast should not be used for the build analysis. If a travel model is being used, then the analyst should review the build assignments to ensure that they reasonably reflect the proposed improvements, including comparing to the no-build assignments. If these forecasts are done by manual methods, a controlling factor in making these adjustments is to maintain the total trip origins and destinations for each land use generator within the study area.

  For example, if the alternative consolidates access to a shopping center, the sum of vehicle trips in and out of the shopping center should be the same before and after the project. The volumes that used the driveways that would be closed by the project must be re-assigned to other driveways that are accessible from the shopping center. This is an example of maintaining the same trip totals around a periphery of an activity center.

  Another example would be where a street extension is proposed to offload local trips from the highway. In this example, the study area includes a one-mile section of a north-south highway that connects to east-west arterials at either end. Before the project there is only one route for all north-south trips. After the project a new parallel north-south collector road is proposed that connects to both of the east-west arterials.

The reasonable check in this case would use a screenline across where the north-south routes connect to the east-west arterials. The total two-way north-south volume should be approximately the same, except for shifts in travel that may have occurred due to the project, for all facilities connecting to the arterials before and after the street extension.

- **Traffic Signal Optimization or Coordination**: The background traffic signal timing parameters should be modified to be consistent with the proposed improvement. Caution should be applied when changing the background signal cycle assumptions for the purposes of future analysis. The analyst should coordinate with the agency responsible for operating the signals to identify upper and lower cycle limits for functional signal operations. Typically the cycle length for the analysis should not exceed 60 seconds for a two-phase traffic signal, 90 seconds for a three-phase traffic signal (e.g., protected highway left turns and permissive side streets left turns) or 120 seconds for a four- or more phased traffic signal.

- **Intersection Approach Lane Changes or Additions**: Any proposed additions or revisions to an intersection approach should be reflected in the capacity analysis and signal phasing, as appropriate. A typical example is adding left turn lanes to serve higher demands during peak hours. New turn lanes may require changes to the background signal phasing to operate safely and the phasing changes should also be reflected in the analysis. In addition, the geometry of the intersection should be reviewed to determine if the added approach lane can be served on the exit leg. For the example above, a second left turn lane on one approach requires a second exit lane on the receiving leg of that intersection for a minimum distance to operate effectively.

Evaluating Severely Congested Facilities

The performance analysis of severely congested roadways and intersections should recognize that many of the conventional (or default) assumptions used in computer software tools are not necessarily appropriate in these cases. For this discussion, severe congestion occurs when the observed demand exceeds facility capacity (v/c is over 1.0). The HCM analysis methods for roadways and intersections are not appropriate in cases where the volume substantially exceeds facility carrying capacity.

When the facility is heavily congested in the base case, the analyst should verify through field studies, additional surveys or other measurements that the observed conditions are reasonably similar to the computer software results. These procedures were covered in Chapter 7, Intersection Analysis. For example, if an intersection analysis indicates v/c ratio near 1.0, it should be noted that intersection evaluations are based on the number of vehicles entering the intersection during the assessment period and may not be the same as the total demand at that location. A field observation may show that heavy vehicle queuing occurs during the peak hour and a substantial share of the actual demand is queued and not served at the intersection during the peak analysis period. In this case, the demand is greater than the actual count of traffic that enters the intersection during the analysis period. When facilities approach capacity levels during the peak hour, one result is for commuters to shift their travel times outside of the busiest hour to reduce their overall travel times. This phenomenon is referred to as peak hour spreading.

For future analysis, a v/c ratio calculation may result in a value higher than 1.0 for an isolated intersection. This condition may result from existing latent demand or excessive future demand of vehicles at an intersection. This should be considered as a d/c rather than an actual v/c ratio and would indicate conditions where mitigation could be considered to improve intersection operations.

Severe forecasted congestion at one location may influence and impact conditions at other intersections within the local transportation system. For example, spillback from one intersection may block traffic from proceeding through a nearby intersection, even when the traffic signal indication permits it. In addition to the intersection v/c ratio analysis, the analyst should review average and maximum (95th percentile) vehicle queues within a congested local system to identify potential cases of secondary congestion impacts, which could reduce the performance otherwise indicated by an isolated intersection analysis for that location. In these types of situations, it is not sufficient to only conduct isolated intersection methods. A more reasonable tool would be a microsimulation, which accounts for interaction between locations, queue spillbacks, blocked intersections and serving excessive demand between signal cycles. See details in Chapter 8.

10.6.2 Progression Analysis

Addendum B

11 AIR AND NOISE TRAFFIC DATA

11.1 Purpose

Federal regulation requires, in some cases, that an air and noise study be completed to determine what impact, if any, will result from a proposed highway improvement and what measures will be taken to lessen these impacts. This chapter presents the general outline for the needs and processing of common data requested for the Air and Noise Analysis section of the Environmental Impact Statement (EIS) or Environmental Assessment (EA). Topics covered include:

- Input for Noise Analysis
- Input for Air Quality Analysis
- EISBase

11.2 Input for Noise Analysis

ODOT is responsible for ensuring that state transportation projects are developed within the Federal Highway Administration's noise policies and procedures. To conduct the noise analysis necessary for measuring compliance, the ODOT Geo-Environmental Section, or noise consultant, requires specific data from the project traffic analyst. This request is typically made through the "Noise, Air and Energy Traffic Requirements Check List" as shown in Appendix H, which is filled out by the noise consultant or Geo-Environmental Section staff and delivered to the project traffic analyst. While this list identifies many different types of possible data needs, the collection and processing of the most common data requested is discussed below. This process should only be done on the No-Build and Selected Alternatives because of the time required to complete the work. Typically it will take a month for the no-build and a bit less for each build alternative.

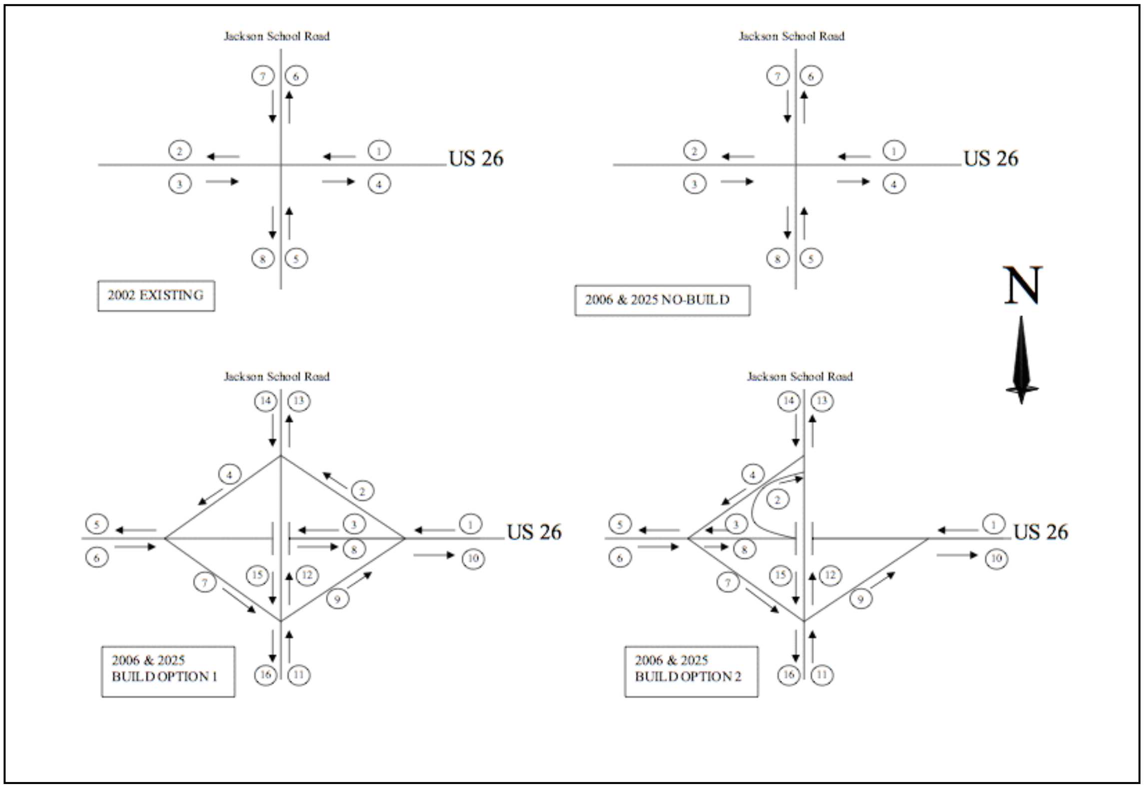
11.2.1 Common Data Needs

The traffic data requested will often be required for a no-build condition under the existing year, the year of opening and the future design year (typically 20 years from opening) as well as for build conditions for each alternative being considered under the year of opening and the future design year.

At the beginning of the project, 16-hour manual full federal (13 vehicle classes) classification turn movement counts need to be ordered at all signalized intersections in addition to all intersections with substantial traffic volumes or heavy vehicle movements. Shorter duration counts can be used at minor intersections between the classification count locations. These counts are also used to develop the traffic volumes for the project. See Chapter 4. The factors that are created from count data are based on the peak hour, the average hour, the average daily traffic, and the peak truck hour, which cannot be calculated from a peak period count. There will need to be enough classification counts so passenger car, medium and heavy vehicle movements can be calculated at the shorter duration count locations. Be sure to request an electronic version of all count data in spreadsheet format to aid in data processing.

The first step in the noise data process should be the creation of link diagrams depicting the study area roadway segments that will be included in the analysis for all no-build and build scenarios considered. These diagrams are not only useful for graphically relating the data provided to its location, but help in identifying links created, modified, or removed with each alternative. Each link should be given a unique number for identification purposes. It should be noted that the link number can be directional. Directional link numbers should be provided for freeways, expressways, interchanges, one-way streets, couplets, divided highways and facilities with separate roadbeds. Where possible, try to keep consistent numbering between the no-build and build link diagrams. This will mean adding extra links that have zero data into the no-build network that will accommodate the build Alternative. The more consistent the diagrams are, the easier it will be for the traffic analyst to troubleshoot and the noise analyst to follow. There is nothing wrong with having links with no data in the scenarios as long as they are labeled as not existing yet or not existing anymore. A set of sample link diagrams is provided in Exhibit 11-1.

Exhibit 11-1 Sample Link Diagram – Jackson School Road Interchange



The second step is recording the specific link characteristics, which include street name, length (in miles), posted speed and LOS C volume. Link length is the center-to-center intersection spacing. Links on the edge of the network should have a length of 0.25 mile. Speeds recorded for this analysis should be either the posted speed limit or the operating speed (highest overall speed at which a driver can travel on a given highway under favorable weather conditions and under prevailing traffic conditions, without at any time exceeding the safe speed as determined by the design speed on a section-by-section basis) where it is determined to be consistently higher than the posted speed limit. Directional interim LOS C volumes for each link can be assumed to be as follows:

- For freeway segments, obtain from the latest version of the HCM. For example, in the HCM 2000 the LOS C volume for a basic freeway segment with a free-flow speed of 60 mph would be 1560 pcphpl. The LOS C volume is calculated using the HCM delay methodologies, via an iterative process based on the project volumes, to identify the volume where the LOS C threshold occurs (at the top end of LOS C, adjacent to LOS D). Use the following defaults only for links at the end of the network.
- For freeway ramps, assume 1000 pcphpl. The analyst should consider effects of ramp metering on freeway ramp LOS C volumes, where applicable.
- For urban arterials, assume 600 pcphpl.
- For suburban arterials, assume 700 pcphpl.
- For rural highways, assume 800 pcphpl.

In noise analysis, the LOS C volume is assumed to represent the maximum volume that can be sustained at free-flow speed. Because vehicle speeds typically affect noise levels more than vehicle volumes, this condition is often the most critical. In areas where peak period congestion is minimal or only occurs for a short time, allowing for continuously high speeds, the peak hour or peak truck hour may be critical. However, in areas where congestion is present for extended periods, lowering vehicle speeds, the LOS C volume may have a greater impact.

11.2.2 Calculations

When the count data ordered becomes available it will be necessary to regroup the 13 vehicle classes into the medium, heavy and all vehicles categories for the noise analysis. Noise sources associated with transportation projects can include passenger vehicles, medium trucks, heavy trucks and buses. Each of these vehicles produces noise, however, the source and magnitude of the noise can vary greatly depending on vehicle type. For example, while the noise from passenger vehicles occurs mainly from the tire-roadway interface and is, therefore, located at ground level, the noise from heavy trucks is produced by a combination of noise from tires, engine and exhaust resulting in a noise source that is approximately 8-feet above the ground. The following list provides information on the types of transportation noise sources that will be part of a typical roadway project, and describes the type of noise each produces[35].

- ***Passenger Vehicles***: Noise emitted from 0 to 2 feet above roadway, primarily from tire-roadway interface. This category includes normal passenger vehicles, small and regular pickup trucks, small to mid-size sport utility vehicles and mini- and full-size passenger vans.
- ***Medium Trucks***: Noise emitted from 2- to 5-feet above roadway, combined noise from tire-roadway interface and engine exhaust noise. This category includes delivery vans (e.g., UPS and Federal Express trucks) large sport utility vehicles with knobby tires, large diesel engine trucks, some tow-trucks, city transit and school buses with under vehicle exhaust, moving vans (U-Haul type trucks), small to medium recreational motor homes, and other larger trucks with the exhaust located under the vehicle. The federal vehicle classifications covered are: 2-axle other with trailer, 2-axle 6-tire single unit and buses. For practical application, include all trucks with 2 axles and 6 tires if insufficient information is available to provide for a more detailed analysis.
- ***Heavy Trucks***: Noise emitted from 6- to 8-feet above the roadway surface, combined noise sources includes tire-roadway interface, engine noise and exhaust stack noise. This category includes all long-haul tractor-trailers (semi-trucks), large tow-trucks, dump trucks, cement mixers, large transit buses, motor homes with exhaust located at top of vehicle, and other vehicles with the exhaust located **above** the vehicle (typical exhaust height of 12- to 15-feet). The federal vehicle classifications covered are: 3-axle and greater single units and all combinations.

---

[35] *Traffic Noise Background Information*, Michael Minor & Associates.

For practical application, include all trucks with 3 or more axles.

NOTE: In reporting information on trucks the following criteria should be used:

- Truck tractor units traveling without a trailer will be considered single-unit trucks.
- A truck tractor unit pulling other such units in a "saddle mount" configuration will be considered one single-unit truck and will be defined only by the axles on the pulling unit.
- Vehicles are defined by the number of axles in contact with the road. Therefore, "floating" axles are counted only when in the down position.
- The term "trailer" includes both semi- and full trailers.

The following are the federal classifications.

- ***Motorcycles (Optional)***: All two- or three-wheeled motorized vehicles. Typical vehicles in this category have saddle type seats and are steered by handlebars rather than steering wheels. This category includes motorcycles, motor scooters, mopeds, motor-powered bicycles and three-wheel motorcycles. This vehicle type may be reported at the option of the State.
- ***Passenger Cars***: All sedans, coupes and station wagons manufactured primarily for the purpose of carrying passengers and including those passenger cars pulling recreational or other light trailers.
- ***Other Two-Axle, Four-Tire Single Unit Vehicles***: All two-axle, four-tire vehicles, other than passenger cars. Included in this classification are pickups, vans and other vehicles such as campers, motor homes, ambulances, hearses, carryalls and minibuses. Other two-axle, four-tire single-unit vehicles pulling recreational or other light trailers are included in this classification. Because automatic vehicle classifiers have difficulty distinguishing class 3 from class 2, these two classes may be combined into class 2.
- ***Buses (Optional)***: All vehicles manufactured as traditional passenger-carrying buses with two axles and six tires or three or more axles. This category includes only traditional buses (including school buses) functioning as passenger-carrying vehicles. Modified buses should be considered to be a truck and should be appropriately classified.
- ***Two-Axle, Six-Tire, Single-Unit Trucks***: All vehicles on a single frame including trucks, camping and recreational vehicles, motor homes, etc., with two axles and dual rear wheels.
- ***Three-Axle Single-Unit Trucks***: All vehicles on a single frame including trucks camping and recreational vehicles, motor homes, etc., with three axles.
- ***Four or More Axle Single-Unit Trucks***: All trucks on a single frame with four or more axles.
- ***Four or Fewer Axle Single-Trailer Trucks***: All vehicles with four or fewer axles consisting of two units, one of which is a tractor or straight truck power unit.
- ***Five-Axle Single-Trailer Trucks***: All five-axle vehicles consisting of two units, one of which is a tractor or straight truck power unit.
- ***Six or More Axle Single-Trailer Trucks***: All vehicles with six or more axles

consisting of two units, one of which is a tractor or straight truck power unit.

- ***Five or Fewer Axle Multi-Trailer Trucks***: All vehicles with five or fewer axles consisting of three or more units, one of which is a tractor or straight truck power unit.
- ***Six-Axle Multi-Trailer Trucks***: All six-axle vehicles consisting of three or more units, one of which is a tractor or straight truck power unit.
- ***Seven or More Axle Multi-Trailer Trucks***: All vehicles with seven or more axles consisting of three or more units, one of which is a tractor or straight truck power unit.

There may be times when data related to buses and motorcycles is requested to be provided separately. Separate motorcycle data is rarely needed in Oregon, but specific data related to bus volumes may be appropriate where the link could be experiencing higher than average bus traffic due to influence by a nearby school, bus barn, or tourist attraction.

The count data can be regrouped by hand or by spreadsheet, however the process can be long and cumbersome. An easy way to process ODOT-counted 12-hour or greater counts is to request the count in electronic form in "TruckSum" format from the Transportation Systems Monitoring Unit. A copy of the spreadsheet is available on the TPAU website. This format organizes the count into the three basic subgroups of medium, heavy and all vehicles for the macro-based TruckSum Excel spreadsheet. See Exhibit 11-2 and Exhibit 11-3, which calculates the initial truck factors. Note: The Exhibit margins were cut off due to a software issue.

The TruckSum spreadsheet calculates for each intersection leg the peak hour volume, the average daily traffic, the average daily truck volume, the average 8-hour volume and the peak truck hour volume. The spreadsheet also calculates the necessary truck factors explained later in this section. The analyst either types the summarized count values into the spreadsheet or copies and pastes the TruckSum-formatted count data into the spreadsheet. Use Paste – Special Values only, to avoid corrupting the output. The analyst also enters the length of the manual count (12 hours minimum), count date and location. Pressing the "Calculate" button on the spreadsheet will generate the factors and the rest of the data (page two of the output). It is best to print out the spreadsheet in landscape format double sided. Note: A side benefit of this spreadsheet, if used early in the project, is that it can be used to help determine the overall peak hour of the study area as will as determining directional factors, K-factors (percent of daily traffic in the peak hour) and truck percentages for each leg.

Exhibit 11-2 TruckSum Input

Summary of Manual Count — 14 hour
Count Date: Oct 12 04
Location: US199 at Dowell

Analyst: Dorothy J Upton

time	type	N - E	N - S	N - W	E - N	E - S	E - W	S - N	S - E	S - W	W - N	W - E	W - S	TOTAL	NORTH LEG	EAST LEG	SOUTH LEG	WEST LEG
6 to 7	Medium Truck	1	0	1	0	1	14	1	2	0	0	6	0	26	3	24	4	21
	Heavy Truck	5	0	0	1	1	15	0	2	0	0	15	0	39	6	39	3	30
	All Vehicles	23	3	6	29	17	212	7	42	2	13	458	1	813	81	781	72	692
7 to 8	Medium Truck	9	4	4	2	1	20	0	4	0	4	13	0	61	23	49	9	41
	Heavy Truck	5	0	1	4	1	15	1	2	0	3	26	0	58	14	53	4	45
	All Vehicles	80	25	35	48	28	565	13	54	11	34	743	5	1641	235	1518	136	1393
8 to 9	Medium Truck	8	1	5	3	2	24	1	2	3	4	18	0	71	22	57	9	54
	Heavy Truck	8	0	1	2	7	25	1	5	0	3	27	0	79	15	74	13	56
	All Vehicles	102	12	45	59	33	512	16	46	11	55	706	6	1603	289	1458	124	1335
9 to 10	Medium Truck	2	0	6	6	0	26	1	2	0	5	15	0	63	20	51	3	52
	Heavy Truck	5	0	0	2	4	23	0	1	1	2	23	0	61	9	58	6	49
	All Vehicles	83	11	46	79	30	595	16	58	6	44	685	6	1659	279	1530	127	1382
10 to 11	Medium Truck	2	2	3	2	0	20	2	0	0	3	14	0	48	14	38	4	40
	Heavy Truck	2	1	5	10	1	27	0	6	0	2	21	0	75	20	67	8	55
	All Vehicles	86	15	47	83	43	562	10	61	4	45	657	3	1618	286	1492	136	1318
11 to 12	Medium Truck	8	2	3	5	2	23	1	1	5	4	12	1	67	23	51	12	48
	Heavy Truck	1	0	0	2	1	17	1	1	0	1	21	0	45	5	43	3	39
	All Vehicles	89	26	43	84	56	551	24	58	14	39	733	7	1724	305	1571	185	1387
12 to 1	Medium Truck	5	0	4	0	6	17	1	1	0	2	23	1	60	12	52	9	47
	Heavy Truck	9	0	1	5	1	25	0	3	0	1	26	0	71	16	69	4	53
	All Vehicles	103	16	46	75	68	737	17	68	9	57	762	6	1964	314	1813	184	1617
1 to 2	Medium Truck	2	1	5	11	4	20	1	4	2	3	23	0	76	23	64	12	53
	Heavy Truck	3	0	2	5	2	25	0	2	0	2	31	0	72	12	68	4	60
	All Vehicles	88	20	49	74	54	694	20	48	9	49	693	3	1801	300	1651	154	1497
2 to 3	Medium Truck	7	3	3	9	0	26	2	3	1	4	25	2	85	28	70	11	61
	Heavy Truck	4	0	3	3	4	27	0	1	0	1	28	0	71	11	67	5	59
	All Vehicles	120	18	53	80	58	793	13	61	8	50	821	9	2084	334	1933	167	1734
3 to 4	Medium Truck	6	2	7	5	2	23	2	2	1	3	15	0	68	25	53	9	49
	Heavy Truck	3	0	1	2	6	24	0	6	0	0	21	0	63	6	62	12	46
	All Vehicles	115	17	56	71	65	862	19	61	7	42	662	6	1983	320	1836	175	1635
4 to 5	Medium Truck	4	2	4	9	7	13	0	6	2	2	24	0	73	21	63	17	45
	Heavy Truck	1	0	1	2	2	19	0	1	0	2	32	0	60	6	57	3	54
	All Vehicles	103	24	69	90	87	932	12	73	12	36	717	8	2163	334	2002	216	1774
5 to 6	Medium Truck	0	1	1	0	2	14	0	0	0	1	10	0	29	3	26	3	26
	Heavy Truck	1	1	0	2	1	23	0	0	0	0	12	0	40	4	39	2	35
	All Vehicles	66	18	48	42	78	968	14	42	8	22	548	1	1855	210	1744	161	1595
6 to 7	Medium Truck	0	0	0	0	0	5	1	0	0	0	6	0	12	1	11	1	11
	Heavy Truck	0	0	0	1	1	9	0	1	0	0	9	0	21	1	21	2	18
	All Vehicles	35	19	22	24	51	713	8	37	7	19	504	3	1442	127	1364	125	1268
7 to 8	Medium Truck	0	0	0	0	0	1	0	0	0	0	8	0	9	0	9	0	9
	Heavy Truck	1	0	0	0	1	13	0	0	0	0	7	0	22	1	22	1	20
	All Vehicles	23	3	8	18	31	504	2	24	10	14	316	3	956	68	916	73	855
8 to 9	Medium Truck	0	0	0	0	0	3	0	0	0	0	3	1	7	0	6	1	7
	Heavy Truck	0	0	0	0	1	10	0	2	0	0	8	0	21	0	21	3	18
	All Vehicles	12	10	10	10	20	337	6	22	5	6	195	2	635	54	596	65	555
9 to 10	Medium Truck	0	0	0	0	0	3	0	0	0	0	6	0	9	0	9	0	9
	Heavy Truck	0	0	0	0	0	1	0	0	0	0	4	0	5	0	5	0	5
	All Vehicles	9	7	12	3	20	236	4	8	0	3	173	1	476	38	449	40	425
TOTAL	Medium Truck	54	18	46	52	27	252	13	27	14	35	221	5	764				
TOTAL	Heavy Truck	48	2	15	41	34	298	3	33	1	17	311	0	803				
TOTAL	All Vehicles	1137	244	595	869	739	9773	201	763	123	528	9373	70	24415				

Exhibit 11-3 TruckSum Output

Summary of Manual Count 14 hour
Count Date: Oct 12 04
Location: US199 at Dowell

Peak Hour is:
4 to 5 PM

HIGHEST 8 HOUR VOLUME FROM :	10 to 6	PM			
type	TOTAL	NORTH LEG	EAST LEG	SOUTH LEG	WEST LEG
Medium	506	149	417	77	369
Heavy	497	80	472	41	401
All Veh	15190	2403	14042	1378	12557

All vehicle ADT Factor 1.18

NORTH LEG		EAST LEG		SOUTH LEG		WEST LEG	
	volume		volume		volume		volume
NB	1886	EB	13302	SB	1243	WB	12379
SB	2332	WB	13430	NB	1283	EB	11766
TOTAL	4217	TOTAL	26732	TOTAL	2525	TOTAL	24145

All Vehicle PHV

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG		
	volume	factor		volume	factor		volume	factor		volume	factor
NB	138	0.073	EB	893	0.067	SB	119	0.096	WB	1013	0.082
SB	196	0.084	WB	1109	0.083	NB	97	0.076	EB	761	0.065
TOTAL	334	0.079	TOTAL	2002	0.075	TOTAL	216	0.086	TOTAL	1774	0.073

Truck ADT by leg

* All factors apply to ADT volumes.

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG			
	volume	factor *		volume	factor *		volume	factor *		volume	factor *	
Medium	218	0.052	Medium	633	0.024	Medium	104	0.041	Medium	573	0.024	
Heavy	126	0.030	Heavy	765	0.029	Heavy	73	0.029	Heavy	642	0.027	
Total	344	0.082	Total	1398	0.052	Total	177	0.070	Total	1215	0.050	TRUCK FACTOR

PHV Trucks ONLY

*All factors apply to PHV for Trucks.

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG			
	volume	factor *		volume	factor *		volume	factor *		volume	factor *	
Medium	21	0.063	Medium	63	0.031	Medium	17	0.079	Medium	45	0.026	Peak Hour Factor MEDIUM TRUCK
Heavy	6	0.018	Heavy	57	0.028	Heavy	3	0.014	Heavy	54	0.030	Peak Hour Factor HEAVY TRUCK

Peak 8 hour Avg.

If the All Vehicle factors are greater than 1.000, then use the corrected values shown. Apply the values as noted .

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG			
	volume	factor *		volume	factor *		volume	factor *		volume	factor *	
All Veh	300	0.899	All Veh	1755	0.877	All Veh	172	0.797	All Veh	1570	0.885	Average Hour Factor ALL VEHICLES
All Trucks	29	0.095	All Trucks	111	0.063	All Trucks	15	0.086	All Trucks	96	0.061	Average Hour Factor TRUCKS
All Veh	300	0.899	All Veh	1755	0.877	All Veh	172	0.797	All Veh	1570	0.885	
All Trucks	29	0.095	All Trucks	111	0.063	All Trucks	15	0.086	All Trucks	96	0.061	

Peak Truck Hours By Leg

* Factors apply to Peak Hour Volumes
** Factors apply to Peak Truck Hour Volumes

If the All Vehicle factors are greater than 1.000, then use the corrected values shown. Apply the values as noted .

NORTH LEG			EAST LEG			SOUTH LEG			WEST LEG			
	volume	factor		volume	factor		volume	factor		volume	factor	
All Veh	334	1.000	All Veh	1933	0.966	All Veh	124	0.574	All Veh	1734	0.977	** Peak Truck Hour Factor ALL VEHICLES
Medium	28	0.084	Medium	70	0.036	Medium	9	0.073	Medium	61	0.035	* Peak Truck Hour Factor MEDIUM
Heavy	11	0.033	Heavy	67	0.035	Heavy	13	0.105	Heavy	59	0.034	* Peak Truck Hour Factor HEAVY
All Veh	334	[illegible]	All Veh	1933	0.966	All Veh	124	0.574	All Veh	1734	0.977	
Medium	28	[illegible]	Medium	70	0.036	Medium	9	0.073	Medium	61	0.035	
Heavy	11	[illegible]	Heavy	67	0.035	Heavy	13	0.105	Heavy	59	0.034	

Analyst: Dorothy J Upton

Average daily traffic (ADT) volumes and peak hour volumes (PHV) maintain the same meaning in noise analysis as they do in other types of analysis described in this manual. They are simply the total number of all vehicle types experienced on a link over a 24-hour period and the highest hourly total of all vehicle types experienced during that 24-hour period, respectively. Both of these must be documented for each link studied during all years and analysis scenarios requested.

The "Truck Factor" represents the percent trucks in the average daily traffic volume, calculated as shown below.

$$\textit{Truck Factor} = \frac{(\textit{24-hour volume of all trucks})}{(\textit{24-hour volume of all vehicles})}$$

The "Peak Hour Factor" for noise analysis, refers to the percent trucks in the peak hour of traffic for all vehicle types. This factor is usually needed for both medium and heavy vehicles separately and is calculated as shown below.

$$\text{Peak Hour Factor,}_{\text{Medium Trucks}} = \frac{(\text{medium trucks in peak hour of all vehicles})}{(\text{all vehicles in peak hour of all vehicles})}$$

$$\text{Peak Hour Factor,}_{\text{Heavy Trucks}} = \frac{(\text{heavy trucks in peak hour of all vehicles})}{(\text{all vehicles in peak hour of all vehicles})}$$

Peak Truck Hour Factors refer to the percent of a specified vehicle type in the peak hour of all truck traffic. Most times the peak truck hour is different from the peak hour. These factors are typically calculated for all vehicle, medium truck and heavy truck classes, but the calculations for all vehicles is different than those for the truck classes. As an example, the peak truck hour factors for various classes are shown below.

$$\text{Peak Truck Hour Factor,}_{\text{All Vehicles}} = \frac{(\text{all vehicles in all truck peak hour})}{(\text{all vehicles in peak hour of all vehicles})}$$

$$\text{Peak Truck Hour Factor,}_{\text{Medium Trucks}} = \frac{(\text{medium trucks in all truck peak hour})}{(\text{all vehicles in all truck peak hour})}$$

$$\text{Peak Truck Hour Factor,}_{\text{Heavy Trucks}} = \frac{(\text{heavy trucks in all truck peak hour})}{(\text{all vehicles in all truck peak hour})}$$

Another type of factor that may be requested, though not often used, is the "Average Hour Factor." This represents the percent of a specified vehicle type in an average 8-hour period and is commonly requested for the all vehicle types and all truck classes.

When bus or motorcycle traffic is requested separately, provide percentages of these vehicle types in the all vehicle peak hour and truck peak hour. For example, if providing bus data, the following calculations would be used:

$$\% \text{ Buses in All Vehicle Peak Hour} = \frac{\text{(buses in peak hour of all vehicles)}}{\text{(all vehicles in peak hour of all vehicles)}}$$

$$\% \text{ Buses in Truck Peak Hour} = \frac{\text{(buses in all truck peak hour)}}{\text{(all vehicles in all truck peak hour)}}$$

Note that while updated ADT and PHV values for each analysis year must be provided, only one set of TruckSum factors must be calculated for each alternative. It is generally assumed that while traffic volumes will increase over time, the proportion of vehicle types in the total volume will remain approximately the same.

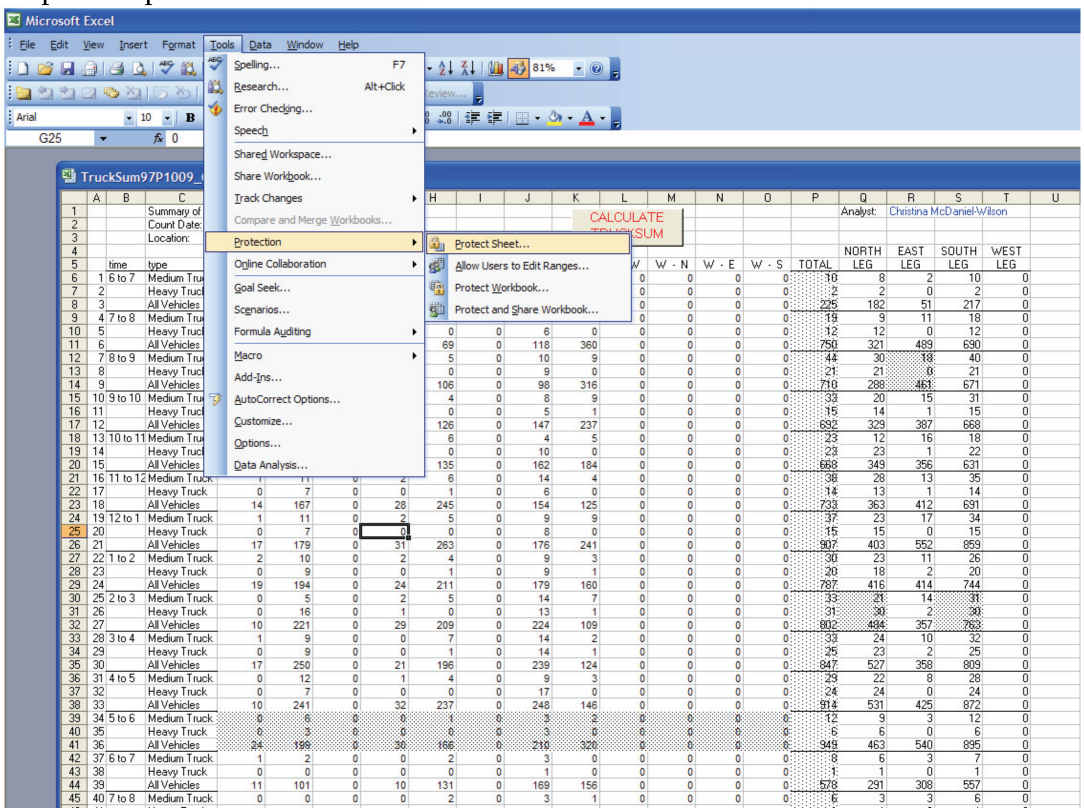
The factors in the TruckSum spreadsheet are for the indicated peak hour. Choose the peak hour that covers the most of the intersections. If the peak hour is different than the chosen hour, then the factors will need to be recalculated using the volumes for that hour. If buses are to be split out separately, then they will have to be done manually by splitting them from the medium trucks.
The factors shown in boxes on the right side of the spreadsheet are described below with the related equations.

Project wide peak hour adjustment using TruckSum

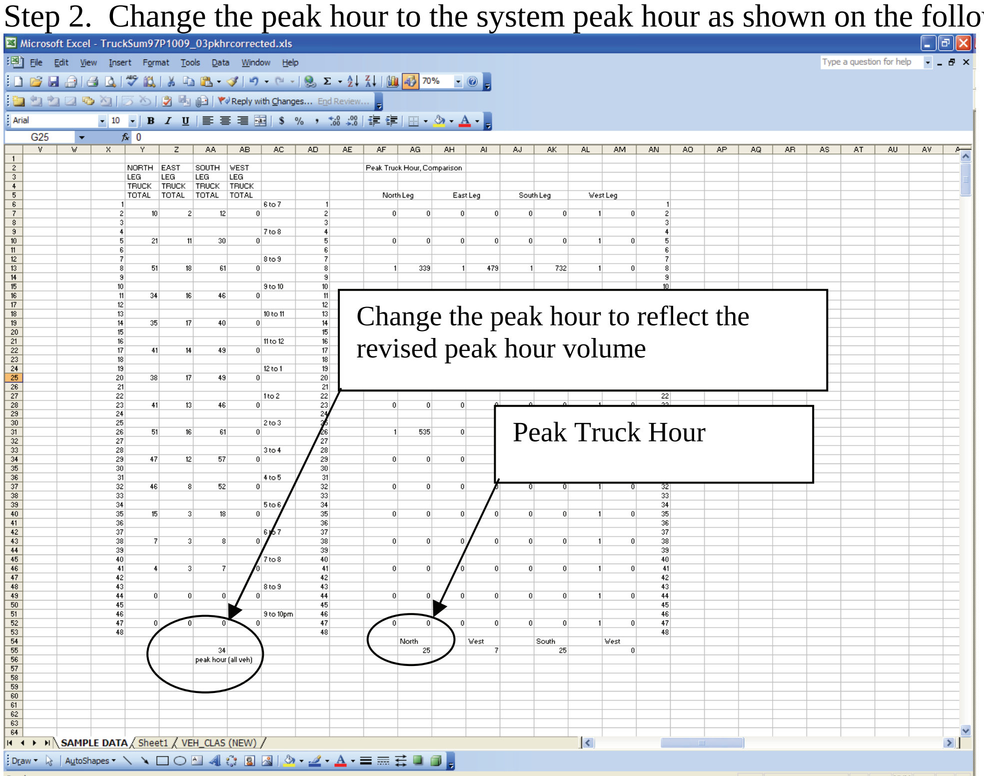
Often, different intersections on a project will have different peak hours. The peak hour calculated by TruckSum is the peak hour for each individual intersection. For those intersections where the peak hour is different from the system peak hour, the analyst can modify TruckSum to report the system peak hour, using the following procedure.

TruckSum factors are calculated using the VLOOKUP function in the EXCEL spreadsheet. To calculate factors for a system peak instead of the intersection peak, the peak hour needs to be changed in the spreadsheet. Over-ride the VLOOKUP function by manually typing in the system peak hour in the peak hour cell. The spreadsheet will no longer choose the peak hour from the table and instead will use the selected hour.

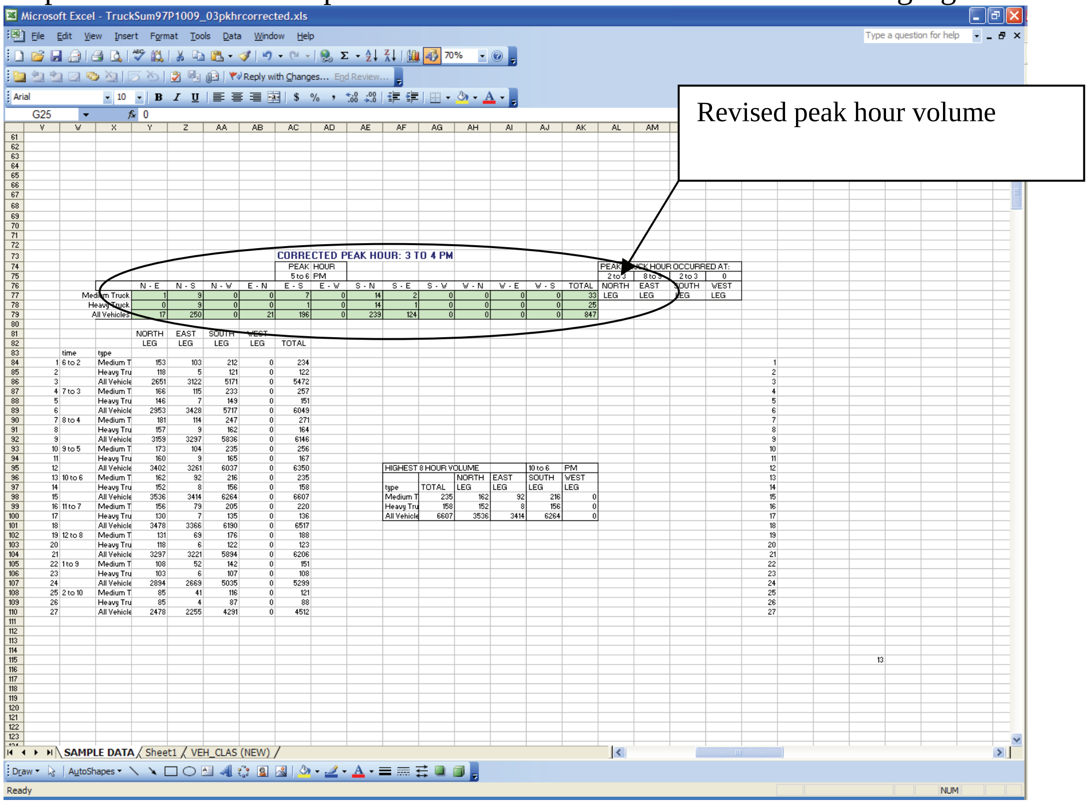
Step 1. Unprotect the worksheet as shown below.

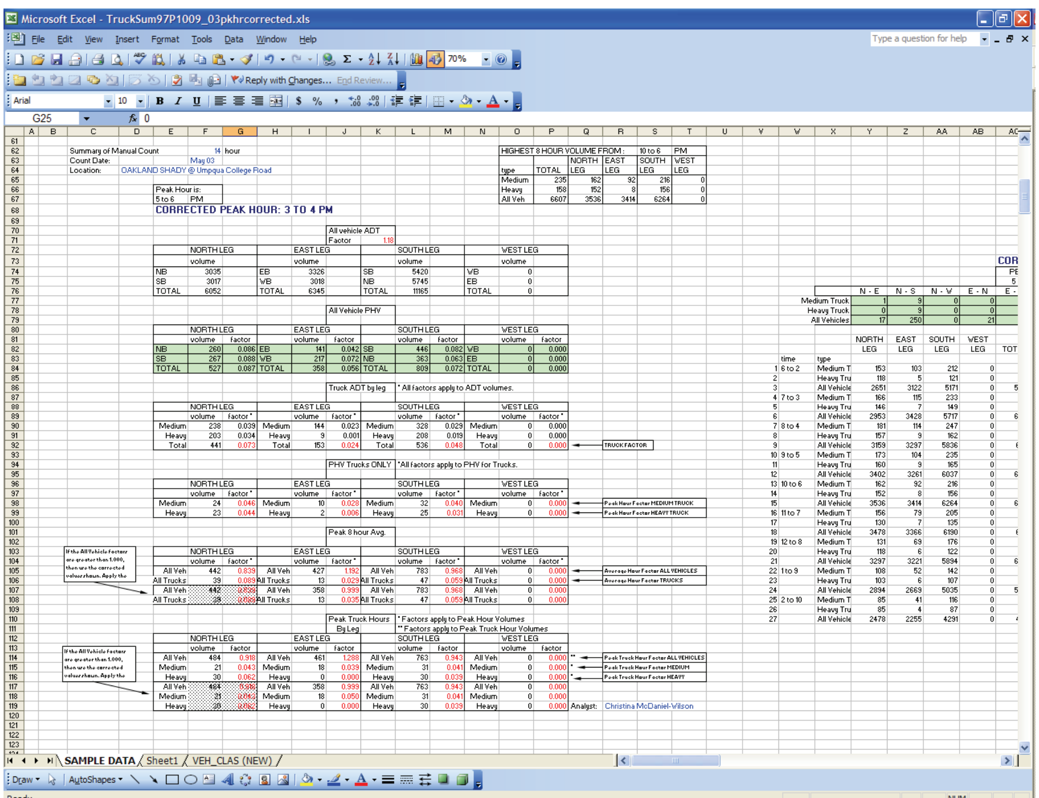


Step 2. Change the peak hour to the system peak hour as shown on the following figure.



Step 3. Observe revised peak hour volumes as shown on the following figures.





11.2.3 Process

The following procedure is suggested. Draw the no-build link diagram on a large blank piece of paper (11”x17”) and put left, through and right arrows at each intersection approach. Take this master sheet and make about 10 copies. Starting with the “peak hour factor medium truck,” write the factor value on each link at the midpoint. Average the factors between adjacent intersections. Multiply this factor times the peak hour volumes for each intersection approach (including the turning movements) for the entire study network. This will generate an initial set of medium truck peak hour volumes. Check and balance the medium trucks over the network. Once the network is balanced, re-compute the final medium truck factor for each link. These steps are necessary to help avoid errors (such as the average hour volume is greater than the peak hour volume) that may come up when the EISBase database program is used to enter all the data. These errors are very hard to track down unless there is this documentation on the factor development. Repeat for the heavy truck peak hour factor.

The average hour for all vehicles factor and the peak truck hour all vehicles factor modify the peak hour volumes. Create the average hour and peak truck hour volumes and the related factors using the above procedure. Use the average hour or peak truck hour volumes to distribute, balance and compute the factors for the average hour truck factor and the medium and heavy peak truck hour factors.

The link ADT should be computed by dividing the directional peak hour volumes by the directional K-factor (these are the factors listed in the “All Vehicle PHV” section of the TruckSum spreadsheet) for each intersection leg. Average the K-factors between adjacent intersections. Sum the directional average daily traffic to get the link average daily traffic. If the directional ADT is desired, multiply the total ADT by the directional split (calculated from the “All Vehicle ADT Factor” section). Average the directional factors between adjacent intersections as well. The average daily traffic should be consistent across the links.

11.3 Input for Air Quality Analysis

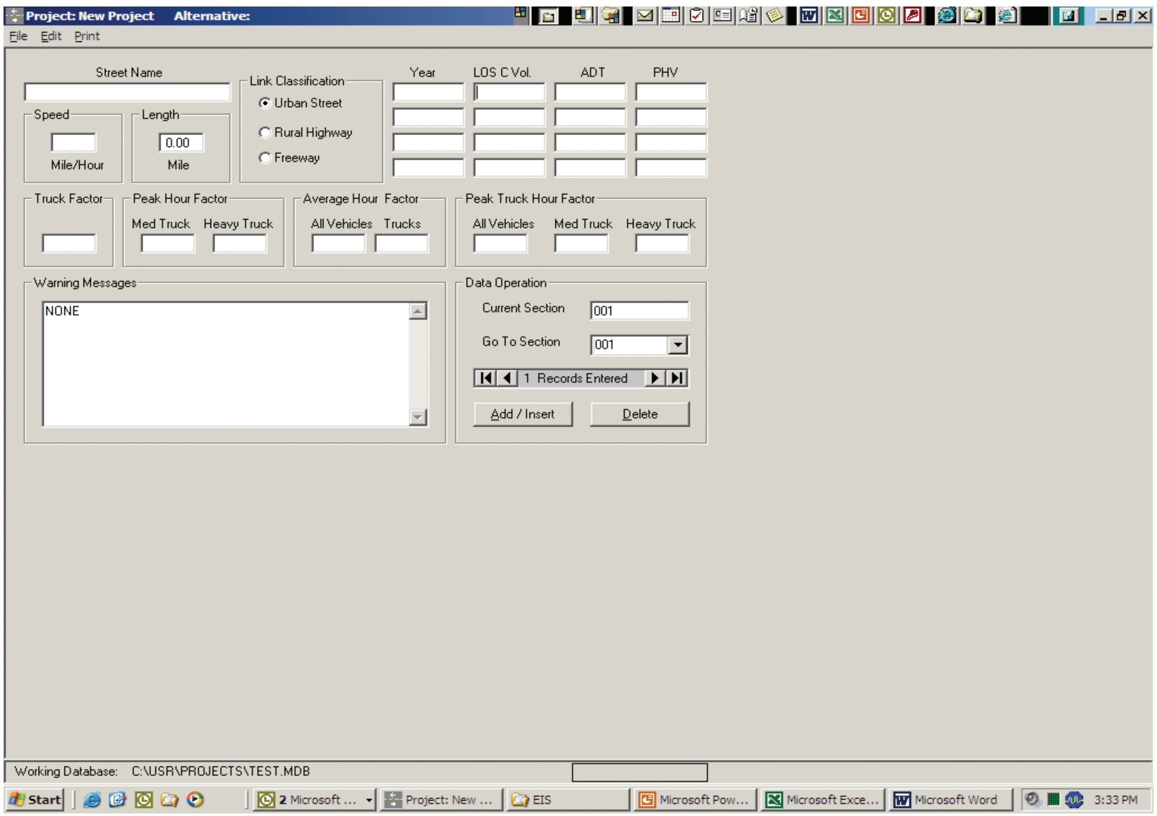
Note: This is an interim section, to be expanded.

Similar to noise analysis, ODOT is responsible for ensuring that state transportation projects are developed within the Federal Highway Administration's air quality policies and procedures. To conduct the air quality analysis necessary for measuring compliance, the ODOT Geo-Environmental Section, or air quality consultant, requires specific data from the project traffic analyst. This request is typically made through the "Noise, Air and Energy Traffic Requirements Check List" shown in Appendix H, which is filled out by the air quality consultant or Geo-Environmental Section staff and delivered to the project traffic analyst.

11.4 <u>EISBase</u>

The EISBase software program is used by ODOT to produce final link volumes and speeds for all analysis scenarios, given the previously described data and factors as input. This represents the finished data needed by the Geo-Environmental staff or noise consultant. Exhibit 11-4 provides an image of an EISBase input screen prior to data entry. As shown in the figure, all of the data required for entry into the input windows has been described in this chapter. No decimal points are required allowing data entry to be sped up, however, data related to buses and motorcycles must be processed separately.

Exhibit 11-4 EISBase Input Screen (Replacement pending.)



While EISBase will calculate future link speeds, there may be times when manual adjustment is required. These instances will typically occur where a future plan shows the classification or use of a link to change in such a manner that the projected speed would no longer apply. Engineering judgment should be used to determine the appropriate link speed when altered manually.

It should be noted when using EISBase, any “All Vehicle Factors” under the “Peak Truck Hour Factor” category that are equal or greater to 1.000 (indicating that the peak truck hour volume and all vehicle peak hour volume are the same) must be input as 0.999 before saving the file. This adjustment is necessary due to a rounding error in the program.

In some cases special categories of vehicles are needed (additional refinement of vehicle types is desired). This is common for buses (such as where a transit mall is present) and passenger cars. For example, if buses are to be split out separately from the all vehicle category, compute all the factors as normal (i.e., heavy, medium and all vehicles) and generate a set of finalized and error-checked values before splitting the buses from the medium trucks. The analyst will have to return to the original manual count sheets to compute the number of buses. Split out the buses from the medium trucks on the factor sheets. Buses plus the remaining medium trucks should equal exactly (this is to avoid creating more errors) to the original medium trucks. The bus data should be inserted into the exported spreadsheet by adding buses peak hour and peak truck hour columns. Adjust the medium trucks downward to accommodate the buses. If passenger cars or motorcycles are desired, then the process is similar, but with the "All Vehicles" category.

After all data has been entered for the given scenarios, check the generated error report (click on "Print => Errors"). The error report checks for consistency between the different link factors and between the different scenarios. For example, the number of trucks in the peak truck hour, for a given link should be greater than the number of trucks in the peak or average hour or the future year ADT should be greater than the existing year ADT. One line is generated for each error. All errors, except for the "LOS C Volumes Exceeded by X%," need to be fixed. Many times the error is caused by rounding either within the program or by the analyst and may only be one or two vehicles different. In all the fixable errors, the analyst will have to go back to the individual volume sheets and do any adjustments to the volumes and the resulting factors.

The TruckSum error check only checks for errors on the link itself for a given year. The analyst should check for errors on the system and between years. It is the reviewer's responsibility to check for reasonability, e.g., the future year should be higher than intermediate years on all links. Also volumes should balance across links.

Example 11-1

Example pending.

11.4.1 Output and Final Product

When all of the data for the applicable analysis scenarios has been entered for each link, all errors have been fixed and the file has been saved, export the results from EISBase into a format that can be converted into a spreadsheet. Providing the analysis results in spreadsheet format facilitates the use of this information by the Geo-Environmental staff or noise consultant. This can be done by clicking the briefcase icon to export the data and selecting the "tab-separated text" as the desired format (.ttx extension). This file can then be opened in Microsoft Excel and adjustments to rows and columns, in addition to any manual data adjustments as described above, can be made, as needed. An example of the finished output that is ready for submittal is shown in Exhibit 11-5. It may be beneficial to ask the requesting noise analyst for preferences in data arrangement to facilitate their intended use (e.g., volumes in rows, vehicle class in columns).

The final submittal from the traffic analyst should include:

- Cover letter explaining contents of enclosures.
- Link diagrams.
- Spreadsheets containing traffic analysis output.
- The error report should be printed for the file only.

Exhibit 11-5 Traffic Analysis Output for Noise Analysis (Replacement pending.)

REGION 1 TRAFFIC ANALYSIS UNIT
EIS TRAFFIC DATA

PROJECT: Jackson School Road Interchange
LOCATION: Southeast of North Plains
ALTERNATIVE: Build Option 1 (with 80-acre UGB Expansion Fully Developed)

PAGE: 1 of 1
PRINTING DATE: November 27, 2002
UNIT: English

			PEAK HOUR					PEAK TRUCK HOUR				
SECT	DIST	YEAR	VOL	AUTO	MTR	HTR	SP	VOL	AUTO	MTR	HTR	SP
US 26 WB east of JSR 1	0.25	2006	1170	1069	39	62	55	1000	828	28	144	55
WB Off-Ramp 2	0.46	2006	370	338	12	20	45	320	265	9	46	45
US 26 WB between ramps 3	0.87	2006	800	717	30	53	55	730	589	22	119	55
WB On-Ramp 4	0.45	2006	40	36	1	3	45	40	32	1	7	45
US 26 WB west of JSR 5	0.25	2006	840	754	31	55	55	770	621	23	126	55
US 26 EB west of JSR 6	0.25	2006	1760	1670	39	51	55	850	748	25	77	55
EB Off-Ramp 7	0.45	2006	90	85	2	3	45	40	35	1	4	45
US 26 EB between ramps 8	0.87	2006	1670	1585	37	48	55	810	713	23	74	55
EB On-Ramp 9	0.46	2006	510	486	11	13	45	240	213	7	20	45
US 26 EB east of JSR 10	0.25	2006	2190	2087	46	57	55	1040	921	31	88	55
JSR NB south of US 26 11	0.5	2006	420	415	3	2	55	180	172	5	3	55
JSR NB overpass 12	0.21	2006	60	60	0	0	55	30	28	1	1	55
JSR NB north of US 26 13	0.38	2006	150	148	1	1	55	150	132	6	12	55
JSR SB north of US 26 14	0.38	2006	210	197	12	1	55	210	176	1	33	55
JSR SB overpass 15	0.21	2006	450	442	6	2	55	270	262	2	6	55
JSR SB south of US 26 16	0.5	2006	390	383	5	2	55	230	223	2	5	55

ABBREVIATION: SECT = SECTION NUMBER; VOL = TOTAL VOLUME; MTR = MEDIUM TRUCK VOLUME; SP = SPEED OF VEHICLE; AUTO = AUTOMOBILE VOLUME; HTR = HEAVY TRUCK VOLUME

ANALYST: ANN L. LIST
CHECKED BY: E. N. GINEER

12 TRAFFIC ANALYSIS REPORTS

12.1 Purpose

Traffic analysis reports are a comprehensive explanation of the final recommendations and the decision making process for a project. This chapter presents an overview of the elements that document the assumptions, methods, findings and recommendations of a traffic analyses. Topics covered include:

- Background
- Technical Memorandum
- Traffic Narrative Report

12.2 Background

This chapter presents an overview of the elements that document the assumptions, methods, findings and recommendations of a traffic analyses. In many cases the narrative and associated diagrams are developed incrementally during the study process in the form of Technical Memorandums, and then circulated for review and discussion at key milestone points during the project review. Any revisions to the Technical Memorandums or new directions in the study analysis are carried forward and then compiled into a full Traffic Narrative at the end stages of the study. The Final Traffic Narrative serves as the legacy document for the study, and should be comprehensive enough to explain and support the final recommendations and decision-making process that led up to it.

12.2.1 Technical Writing Tips

Presentation of technical information in a clear, concise and readily understandable way can be challenging in many regards. This section is not intended to fully answer those challenges, but to highlight several important tips that help to make a technical document achieve these goals. The narrative author is encouraged to avail themselves of training materials or mentors that could help them become proficient technical writers. A few basic tips to suggest in preparing any report include the following:

- **Target Audience**: The intended audience for the report will help to determine the appropriate level of assumed technical knowledge about the subject at hand, and their assumed understanding of the review, adoption and implementation processes for a particular project. In general, the majority of traffic reports will be developed for the review and implementation by staff within, or contracted by, ODOT. In general, these team members have minimal background in the technical traffic issues, but significant experience with the overall process involved. To this end, the technical aspects and outcomes of the project should be clearly explained with a minimum of technical detail necessary to support and explain the narrative. This is very important because writing at the wrong level can generate unintended questions. More extensive technical calculations, findings and other reference materials should be attached to the document as appendices.

  In most cases a document could be circulated to the general public, the press, or other outside agency. In these cases, many of these more fundamental assumptions and process steps should be clearly detailed in the narrative. Presentations to the CAC groups generally handled like any general public group, with the focus on overall process, criteria, outcomes, recommendations and next steps, with a bare minimum of technical content.

- **Tone and Style**: It is recommended that the narrative, in all cases, remain objective, impartial and impersonal so that the findings and recommendations are untainted by any biases. It should be recognized that any internal ODOT document might be released for public review outside of the designated committee groups. This could occur by informal sharing in the interest of coordination or, more formally, through a legal search warrant. All report narrative documents should be treated as if the general public and press will review them, even though many only circulate to the immediate committee members. No matter the purpose or scope of the report, it is vital to maintain a clear and objective style without introducing biases into a traffic report. To be clear that any recommendations are those of the author, not necessarily of ODOT, it is preferred to use the phrase, "It is recommended that . . ."

- **Readability and Document Structure**: The following sections of this chapter have suggestions about the narrative general layout of the document, but these should be tailored, as appropriate, to address individual study scopes and objectives. One of the keys for rapidly understanding materials is to divide the document into a logical, easy-to-follow flow of narratives, summary tables and illustrations that are grouped according to key topics. In a report, for example, they would be grouped by chapter, or by sub-topic in a lengthier chapter. This basic structure provides a convenient framework for presenting and referencing a wide range of materials.

- **A Word About Acronyms**: A comprehensive list of acronyms used in transportation evaluations are assembled in Appendix B of this manual for reference purposes. However, care should be taken when developing the report narrative to limit the use of acronyms, except for the most common ones, that appear repeatedly throughout a particular document. The most common examples might include: ODOT, v/c ratio, OHP and HDM. Excessive use of acronyms can quickly degrade the readability of the narrative, even when the reader understands their meaning. It is standard practice to introduce any acronym in the narrative when it is first used by defining it. In longer reports, it is also useful to attach a short list of all the acronyms used in the report as a quick reference guide.

12.2.2 Diagrams and Illustrations

Technical diagrams can be a powerful resource for quickly explaining report assumptions, findings and recommendations. One measure of a high quality report would allow a reader to scan through the study tables and figures, and then be able to glean the general conclusions without reading any of the narrative. For the purposes of traffic study reports, the technical diagrams include the following list of typical illustrations:

- Study area map.
- Local street and highway system.
- Traffic volumes on links or turning movements at intersections or junctions.
- Trip patterns or trip distribution routes.
- Lane diagrams of existing or proposed intersection approaches.
- Existing or proposed circulation routes within the study area.
- Existing and proposed street or ramp centerline alignments.
- Alternative street improvement scenarios.
- Preferred street improvement scenario.

- Land use and zoning maps.

The best report graphics clearly label key reference streets, maintain a reasonable minimum 8-point font size, and avoid trying to illustrate many layers of new information at one time. A good rule-of-thumb is to limit the number of new layers to three or less for any diagram. Examples of different information layers would be streets, peak hour volumes and functional street class. Complex diagrams can be developed in stages, explaining each new set of layers. In general, street project alternatives should be illustrated on separate diagrams.

All documents need to be legible and usable in black and white.

12.2.3 Tables

Summary tables should be included for ease in making comparisons among scenarios and alternatives. Failing conditions should be denoted with white text on a black background. The preferred software to build tables is MS Word as opposed to MS Excel, due to formatting issues, although MS Excel may be acceptable for appendices.

12.3 <u>Technical Memorandum</u>

12.3.1 Purpose

A technical memorandum (TM) typically addresses one major stage of the project evaluation process, and presents the analysis, findings and any potential next steps for that stage. Subsequent technical study stages build on the information presented in the previous memorandums, and allow for an incremental process to assess, refine and build consensus on the preferred project. These technical memorandums are also described in Chapter 2.

12.3.2 Products

The focus of a technical memorandum can vary widely, but, in general, they include the following technical materials, in a typical 3-stage study development process.

TM #1 - Existing/No-Build System Analysis: This memo presents the key system inventory features and performance deficiencies that will shape development of study alternatives. The memo should include statements on the project purpose and need, study area background, and existing and future volume development. Discussed results should include the crash analysis and possible countermeasures, preliminary signal warrants, access or spacing issues, the volume-to-capacity ratios and LOS, if appropriate, and the 95th percentile queues.

TM #2 - Preliminary Alternatives Screening: This memo presents the screening criteria, the initial roster of project alternatives and the scoring of how well the preliminary alternative matched up with the screening criteria. Screening criteria are more general indicators of performance. This could include performance analyses, or these could be deferred until the next stage. Screening performance results typically include Level of Service results, volume-to-capacity ratio results and model-based results (travel times, speeds, v/c ratios, relative comparisons). Remember to keep track of the reasons why alternatives were dropped (will be needed for the narrative).

TM #3 - Future Alternatives Analysis: This memo presents the detailed evaluations of all alternatives that progressed through the screening process. These alternatives have full performance assessments and any other related evaluations (preliminary environmental, compliance with standards, etc.) as defined in the study criteria. Detailed performance results typically include Level of Service results, volume-to-capacity ratio results, 95th percentile queues, storage lengths required and simulation results.

For consultants doing ODOT analysis work, all input and output sheets shall be included with all technical memos and narratives.

12.3.3 Distribution

The technical memorandums should be distributed to the PT and the CAC for review and comment. The Region Traffic Manager should be added to the distribution list where he/she is not a member of the PT.

12.4 <u>Traffic Narrative Report</u>

12.4.1 Purpose

The majority of the traffic study analysis will be completed by the point that the Draft Traffic Narrative Report is developed. The purpose of this report is to present the final solution selected from the study alternatives.

12.4.2 Product

The Draft Traffic Narrative Report should present the full study process and outcomes, include the interim Technical Memorandums and any feedback from work team committees or other ODOT units that reviewed and commented on this effort. The major step to be completed with the Draft Traffic Narrative Report is to provide conclusions on the function of alternatives from a traffic analysis standpoint.

The project team selection process for a preferred alternative overall uses the analytical evaluation outcomes, relative scoring evaluations to isolate one alternative, or a hybrid of several alternatives that best meet the study objectives. This is necessarily a collaborative process with established Project Management Team members and affected ODOT technical units.

The report itself should be developed consistent with the following standard outline. A sample narrative report has been provided in Appendix F.

Sample Outline

- **Cover Sheet**
    - Agency/Company Title, Division, Unit, City, State (in header, footer or along bound edge)
    - “Project Title Traffic Analysis” (to clarify that this is just the traffic analysis)
    - City (if applicable) and County
    - Highway Name, Number and Route Number
    - Milepoint Range
    - Month and Year report published
- **Title Page**
    - “Project Title Traffic Analysis” (to clarify that this is just the traffic analysis)
    - Highway Name, Number and Route Number
    - Milepoint Range
    - Full Mailing Address
    - Prepared by and reviewed by (including stamp by preparing PE or reviewing PE if preparer is not registered; requires signature of non-registered preparer)
- **Table of Contents, List of Figures, List of Tables, List of Appendices**

- **Executive Summary**: Summary of report including purpose, need, scope of alternatives, re-statement of conclusions and alternative recommendation.
- **Background Information**: Overview of study area including vicinity and study area maps, affected facilities and jurisdictions, past project or planning decisions that could influence outcomes, general problem statement and objectives for the study.
- **Existing Conditions**: Inventory and analysis of base year facility and operating conditions.
- **Future Year Forecasts and Needs (No-Build)**: Horizon year traffic forecasts and performance assessment on the existing street system with no project improvements. Agreed upon baseline projects should be included. See Chapter 9 for more details.
- **Preliminary Alternatives Screening**: Screening criteria, concept alternatives to address outstanding needs and preliminary screening of alternatives with highlight of those set aside from further evaluation.
- **Alternative Results**: Discussion of performance results for each analyzed alternative for the build, interim and design years.
- **Alternative Summary**: The alternatives are compared and contrasted against each other, including a summary table, according to appropriate performance measures.
- **Conclusions**: The analyst should be careful to make conclusions based on the traffic analysis, rather than recommendations on a preferred alternative, as the best alternative from a pure traffic standpoint may not be the best overall. The analyst should coordinate with the PT Leader if it is desired to also report the recommendation by the project team as to the overall preferred alternative.
- **Further Areas of Study**: Optional
- **Appendices**
    - Crash History: Detailed crash analysis and listing of crashes in study area.
    - Record of Calibration: The calibration record will vary in detail level and length by project, but the record should address the following items;
        - A table or list citing all changes that were made to the inputs or model modules to achieve calibration, beyond the standard changes that would occur after collecting field inventory (standard list found in Section 3.2). This list or table should include
            - the issue that was occurring before the change was made,
            - the goal of the change, and
            - some record how the change improved the calibration.
        - For each Measure of Effectiveness (MOE) of the calibration, include a table that shows the before and after results for each MOE. Before results should be with all standard inputs, but no changes beyond the standard adjustments. After results should recorded after all changes to achieve calibration were included in the model. Minimally, the APM requires that the MOE – "Vehicles Exited" be used to assess the calibration of microsimulations (for SimTraffic, for other software use a comparable measure that sums vehicles making individual

movements).
    - The record should indicate that every movement met the calibration standards described in section 8.3 for "Vehicles Exited" (8.3 is specific to SimTraffic, but simulations in any software should meet this criteria).
  - Traffic Development: Count locations, explanation of base and future volume development, includes land use and zoning maps.
  - Existing Year Volumes: Volume diagrams for the existing (base) year.
  - Build Year Volumes: Volume diagrams for the build year.
  - Future No-Build Volumes: Volume diagrams for the future No-Build year.
  - Alternatives Considered but Dismissed: Short description of each dismissed alternative including why it was dropped.
  - Build Alternative Volumes: Volume diagrams for each alternative. Each build and design year for each alternative will be a separate appendix.
  - Analysis Methodologies: Boilerplate text on analysis methods used.
  - EIS Traffic Data: For No-Build and Build alternatives, including link diagrams.

The volume diagrams in the report should include the Preferred Alternative, and any other alternatives that were evaluated for the purposes of the environmental review process.

Technical appendices, including all data, and all software input and output files and reports should be burned to CD or DVD, and retained in the ODOT file. For consultants doing ODOT analysis work, all input and output sheets shall be included with all technical memos and narratives.

A draft of the narrative needs to be sent to Region Traffic, TEOS, the project leader, the Roadway designer, Environmental and any others who may be affected, for review and comment.

12.4.3 Distribution

Upon incorporation of comments received on the draft, the Traffic Narrative Report should be signed and stamped, and should be distributed to the following in addition to the draft reviewers:

- Project Teams
  - Project Development Team
  - Citizen Advisory Committee
- ODOT Region/District Groups
  - Traffic Operations
  - Region Traffic
  - Roadway
  - Environmental

- Geo-Hydro
- Bridge