



RESEARCH

# Oregon Department of Transportation Access Management Best Practices Manual

ODOT/OTREC

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# Oregon Department of Transportation

# Access Management Best Practices Manual

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# 1.0 MANUAL OVERVIEW AND PURPOSE

"Access management is the systematic control of the location, spacing, design, and operation of driveways, median openings, interchanges, and street connections to a roadway" (*TRB, 2003*). Through the strategic application of access management techniques, roadway corridors can benefit from improved traffic safety and operations. In fact, when access management techniques are consistently applied, in addition to their safety and operational benefits, they will help to maintain or improve property values, facilitate bicycle and pedestrian activities, and provide continuity to the surrounding transportation infrastructure. All of these access management outcomes help to make the road purpose more compatible with the surrounding land use and community goals and objectives.

The *Access Management Manual* (AMM) simplifies the overall application of access management to the following ten basic principles (*TRB, 2003*):

- Provide a specialized roadway system;
- Limit direct access to major roadways;
- Promote intersection hierarchy;
- Locate signals to favor through movements;
- Preserve the functional area of intersections and interchanges;
- Limit the number of conflict points;
- Separate conflict areas;
- Remove turning vehicles from through traffic lanes;
- Use non-traversable medians to manage left-turn movements; and
- Provide a supporting street and circulation system.

To effectively apply these concepts, however, it is important to also understand to what degree each will enhance the facility performance. In many cases, systematic data collection is essential to quantifying how well access management has worked at a location. The access management hierarchy can be reduced to system-wide strategies, corridor implementation strategies, and local or individual access point strategies.

This manual is provided as a resource to help Oregon transportation professionals quantify the expected benefits of various access management applications. As a result, this manual includes recommendations for how to evaluate potential access management applications including expected performance metrics and recommended data collection information.

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# 2.0 ACCESS MANAGEMENT PERFORMANCE MEASURES

The data collection, assessment, and reporting of access management performance can help Oregon transportation agencies track the effectiveness of current and prospective access techniques. A key consideration for selecting candidate performance measures is to identify variables that are directly under the control of the associated transportation agency. Characteristics of performance measures can include the following:

- Describe facility operations,
- Assess status or trends to target values,
- Diagnose issues and determine improvement strategies,
- Identify metrics an agency can use to assess performance,
- Incorporate decision making into process through evaluation, contrast, and prioritization, and
- Communicate and document how a facility is performing (Bochner & Storey, 2011).

As a first step towards identifying best practices for access management in Oregon, the description, assessment, and diagnosis of candidate treatments requires basic information about how each access management technique will perform. These individual applications are summarized in the following sections. Performance information and, where available, assessment techniques are included.

## 2.1 INTERSECTIONS

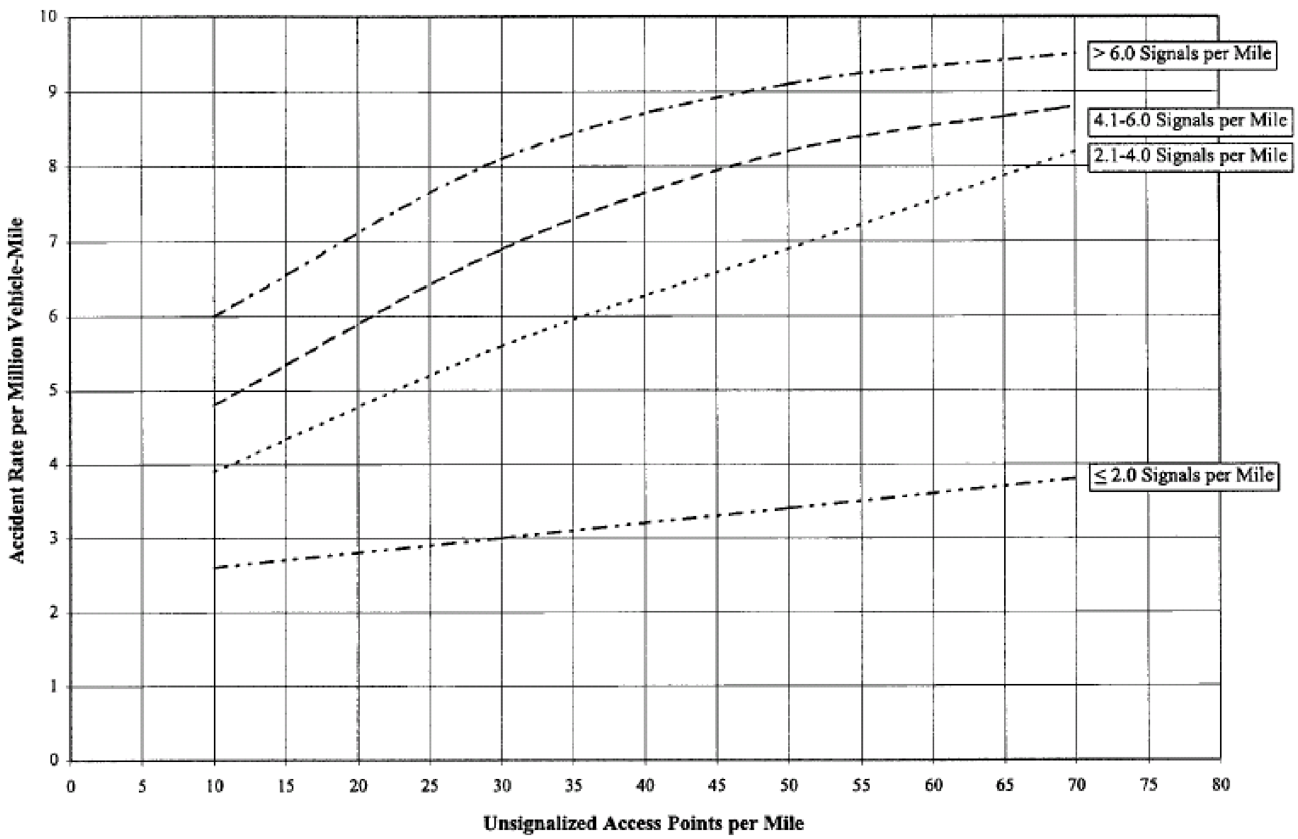
By its definition, access management focuses on the spacing and orientation of a variety of intersection types including signalized, unsignalized, and driveway locations. The following sections further describe expected access management performance at these intersection locations. The performance measures included represent the best information currently available.

### *2.1.1 SIGNALIZED INTERSECTIONS*

The strategic spacing of signalized intersections can facilitate access management. Closely spaced signalized intersections may have overlapping influence areas so that safe driveway placement is not practical. Similarly, signalized intersections spaced far apart can potentially reduce delay and accommodate driveway placement, but a substantial distance between signalized intersections may also contribute to higher operating speeds and more severe crashes. As a result, signalized intersection spacing can directly influence corridor safety and operations as reviewed in the following sections of this manual.

*Safety Assessment*

Figure 2.1 illustrates the relationship between signal spacing density and crash rates (*Gluck, Levinson, and Stover, 1999*). The figure specifically shows a relationship between signals per mile, unsignalized access points per mile, and crash rate. For example, a location with 20 unsignalized access points per mile and 4.1-6.0 signals per mile results in approximately 5.9 crashes per million vehicle miles traveled. A site with a similar number of unsignalized access points but less than 2.0 signals per mile would result in approximately 2.8 crashes per million vehicle miles travelled. As expected, the higher signalized intersection density results in a much greater crash rate. Figure 2.1 also indicates that as unsignalized access points per mile increase, crash rates also increase for all signal density configurations. The current trend in safety assessment is to, where ever possible, assess crash frequency rather than crash rates; however, at this time similar crash frequency assessment techniques are not fully developed for signalized intersection spacing. As a result, the crash rate should be used as the safety assessment metric.




*Source: NCHRP Report 420, Figure 26 (Gluck, Levinson, & Stover, 1999)*

Figure 2.1: Urban and suburban area access density versus average crash rates

*Operational Effectiveness*

The spacing of traffic signals (frequency and uniformity) is a critical access management technique. Traffic signals directly influence delay and may constrain capacity during peak hours. Poorly spaced signalized intersections can directly influence operating speeds and general traffic operation for the corridor and associated driveways. Table 2.1 provides summary information about optimum signalized intersection spacing based on cycle length and speed.

Table 2.1: Optimum Signalized Intersection Spacing Needed for Efficient Traffic Progression

| **Cycle Length** (sec.) | **Speed (mph)** | | | | | | |
|---|---|---|---|---|---|---|---|
| | 25 | 30 | 35 | **40** | 45 | 50 | 55 |
| | **Distance** | | | | | | |
| 60 | 1,100 | 1,320 | 1,540 | 1,760 | 1,980 | 2,200 | 2,430 |
| 70 | 1,280 | 1,540 | 1,800 | 2,050 | 2,310 | 2,500 | 2,820 |
| 80 | 1,470 | 1,760 | 2,050 | 2,350 | 2,640 | 2,930 | 3,220 |
| 90 | 1,630 | 1,980 | 2,310 | 2,640 | 2,970 | 3,300 | 3,630 |
| 120 | 2,200 | 2,640 | 3,080 | 3,520 | 3,960 | 4,400 | 4,840 |
| 150* | 2,750 | 3,300 | 3,850 | 4,400 | 4,950 | 5,500 | 6,050 |

* Represents maximum cycle length for actuated signal if all phases are fully used

*Source: NCHRP Report 420 (Gluck, Levinson, & Stover, 1999)*

In urban areas, one-half mile spacing is generally considered the optimal choice for signalized intersections as this configuration is associated with minimum travel times and the highest speeds (*SRF Consulting Group Inc., 2002*). For each additional traffic signal per mile, the speed reduces by about 2 to 3 mph. Table 2.2 demonstrates the expected travel time increases based on traffic signal density. As shown, for each additional traffic signal over two per mile (i.e., one-half mile signal spacing) the travel time increased by over 6-percent.

Table 2.2: Percentage Increases in Travel Times as Signal Density IncreaseS

| **Signals Per Mile** | **Percent Increase in Travel Times (Two Signals Per Mile as Base)** |
|---|---|
| 2.0 | 0 |
| 3.0 | 9 |
| 4.0 | 16 |
| 5.0 | 23 |
| 6.0 | 29 |
| 7.0 | 34 |
| 8.0 | 39 |

*Source: NCHRP Report 420 (Gluck, Levinson, & Stover, 1999)*

**Signalized Intersections Summary**

*Performance Measures:*

- Crash Frequency or Crash Rate
- Travel Time

Required Data:

- Signals per Mile
- Unsignalized Access Points per Mile
- Traffic Volume (AADT or VMT)
- Crash History (3-year minimum recommended)

### *2.1.2 Unsignalized Street Intersections and Driveways*

Unsignalized access points can include driveways as well as street intersections. Dense placement of these access points can increase conflicts between turning and through vehicles and will introduce delay to the traffic stream due to this potential disruption of traffic. Modified spacing of unsignalized intersections and driveways, therefore, is a common access management strategy. In addition, the placement of driveways too close to the influence area of an intersection can further complicate traffic operations and safety.

#### 2.1.2.1 Traditional Public Street Intersections

Traditional unsignalized public street intersections are locations where public streets intersect at either a cross or tee configuration. Traffic control configurations at these locations are typically a STOP condition for the minor road. The major road may have a STOP condition or may be free-flowing.

*Safety Assessment*

The safety associated with the number and spacing of access points (unsignalized intersections plus driveways) is often evaluated as one collective access management configuration. In general, reducing the number of these access points per mile will improve corridor safety. Section 2.1.2.2 further defines ways to assess the influence of individual driveways as well as corridor driveways and their effect on safety, but Table 2.3 demonstrates the collective safety influence of the urban and suburban area unsignalized and signalized access point spacing. In addition, a roadway with 60 access points per mile can generally be expected to have three times more crashes than a corridor with 10 access points per mile (see Table 2.4).

Table 2.3: Estimated Crash Rates by Access Density (Urban and Suburban Areas)

| **Unsignalized Access Points Per Mile** | **Signalized Access Points Per Mile** | | | |
|---|---|---|---|---|
| | ≤ 2 | 2.01-4.00 | 4.01-6.00 | > 6 |
| ≤ 20 | 2.6 | 3.9 | 4.8 | 6.0 |
| 20.01-40 | 3.0 | 5.6 | 6.9 | 8.1 |
| 40.01-60 | 3.4 | 6.9 | 8.2 | 9.1 |
| > 60 | 3.8 | 8.2 | 8.7 | 9.5 |
| All | 3.1 | 6.5 | 7.5 | 8.9 |

*Note: Crash rates have units crashes per million vehicle miles*

*Source: Adapted from NCHRP Report 420 (Gluck, Levinson, & Stover, 1999)*

Gluck et al. (1999) suggested the use of a crash rate index as a means of assessing relative crash rates for access density. A value of 1.0 is assumed to represent minimal crash risk. As the value of the crash rate index increases, the corridor safety diminishes.

Table 2.4: Relative Crash Rates for Total Access Connection Spacing

| **Total Access Points Per Mile (both directions of travel)** | **Crash Rate Index** |
|---|---|
| 10 | 1.0 |
| 20 | 1.4 |
| 30 | 1.8 |
| 40 | 2.1 |
| 50 | 2.5 |
| 60 | 3.0 |
| 70 | 3.5 |

*Source: Adapted from NCHRP Report 420, Table 4 (Gluck, Levinson, & Stover, 1999)*

<u>Operational Effectiveness</u>

In addition to the expected safety benefits, reducing the number of unsignalized access points can improve traffic flow conditions. Common traffic operations performance measures can include travel time, delay, and operating or free-flow speed. In the past, researchers have focused on these various metrics when assessing the operational impact of access management strategies. Due to the ease of measuring or estimating speed, the most frequently estimated performance measure for operational effectiveness of access points is free-flow speed.

The *Highway Capacity Manual* (HCM) (*TRB, 2010*) addresses the impacts of access density by identifying free-flow speed reduction based on the number of adjacent access points contrasted to the number of through lanes. For a location with 20 access points per mile, the HCM proposes a 1.6 mph reduction in free-flow speed for two-lane highways (one lane for each direction of travel) and a speed reduction of 0.8 mph for four-lane highways (two lanes for each direction of travel) as shown in Table 2.5. The number of access point approaches should be counted separately for each side of the segment.

Table 2.5: Access Point Density Adjustment Factors

| **Access Points Per Mile (Unsignalized Driveway and Public Street Approaches)** | **Reduction in Free-flow Speed for Number of Through Lanes in Direction of Travel (mph)** | | | |
|---|---|---|---|---|
| | **1 Lane** | **2 Lanes** | **3 Lanes** | **4 Lanes** |
| 0 | 0.0 | 0.0 | 0.0 | 0.0 |
| 2 | 0.2 | 0.1 | 0.1 | 0.0 |
| 4 | 0.3 | 0.2 | 0.1 | 0.1 |
| 10 | 0.8 | 0.4 | 0.3 | 0.2 |
| 20 | 1.6 | 0.8 | 0.5 | 0.4 |
| 40 | 3.1 | 1.6 | 1.0 | 0.8 |
| 60 | 4.7 | 2.3 | 1.6 | 1.2 |

*Source: Adapted from HCM Exhibit 17-11 (TRB, 2010)*

**Unsignalized Traditional Public Street Intersection Summary**

*Performance Measures:*

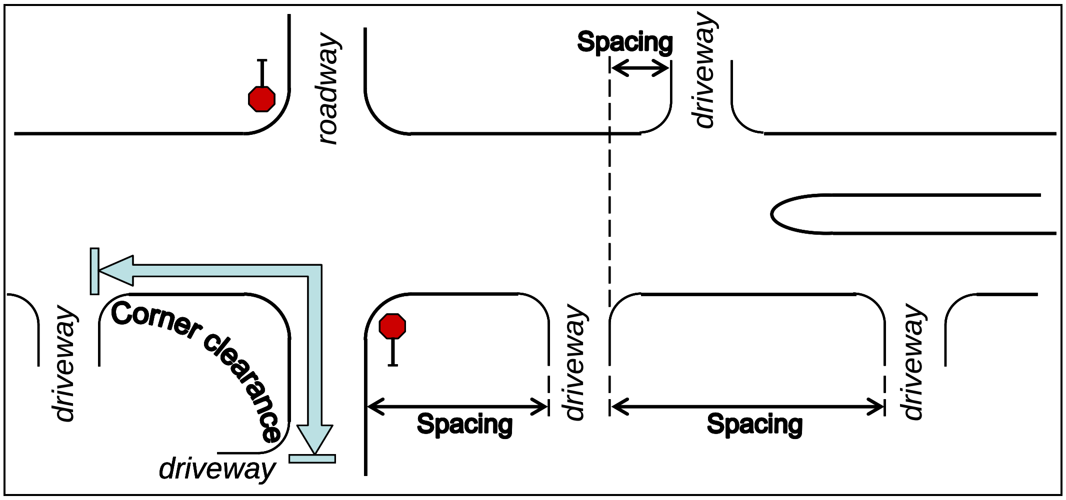
- Crash Frequency, Crash Rate, or Crash Rate Index
- Free-flow Speed

Required Data:

- Signals per Mile
- Unsignalized Access Points per Mile
- Total Access Points per Mile
- Crash History (3-year minimum recommended)
- Number of Through Lanes in Direction of Travel

### 2.1.2.2 DRIVEWAY CONNECTIONS

Within the State of Oregon and throughout the United States, thousands of driveways currently exist. Many of these driveways were in place prior to permit regulations, while others have been installed in recent years. When permit applications are filed, Oregon agencies need to be able to use quantifiable performance measures to help determine if a driveway connection should be permitted or a driveway should be relocated. Figure 2.2 depicts a schematic of unsignalized access spacing concepts including spacing measurement boundaries. Safety and operational assessment strategies for the placement of driveways are summarized in the following sections.




*Source: Gattis et al., 2010*

Figure 2.2: Schematic of Unsignalized Access Spacing Concepts

<u>*Safety Assessment*</u>

In addition to the general safety measures associated with intersection and driveway unsignalized access points (see Section 2.1.2.1), ODOT recently developed a procedure that will enable assessment of the safety implications of modifying driveway configurations on arterial corridors in urban and rural locations. This effort is described in full detail in a separate research report, but the basic procedure is shown in this section. A “smart spreadsheet” is available for applying this procedure.

In addition, understanding the level of risk for an individual driveway or intersection is important when considering placement of that facility. Appendix A of this report includes an alternate procedure for assessing relative access point risks. A “smart” spreadsheet is also available for the access point specific relative risk assessment techniques.

The following summaries present the corridor assessment procedures separately for urban and rural arterial corridors.

**<u>Urban Arterial Crash Prediction Model Computational Tools</u>**

To predict the number of segment crashes for urban arterial locations, the following information is needed:

- Length of the road segment to analyze (in miles),
- AADT for the segment,
- Speed limit for the road segment,
- Cross-section information: Number of travel lanes and presence of TWLTL median,
- Total number of driveways dedicated to commercial and industrial land uses, and
- Total number of driveways dedicated to other land uses.

The procedure for determining the overall safety performance of an urban arterial corridor relative to driveway configuration and land use can be accomplished using the following procedure:

**Step 1:** Using Equation 1, compute the baseline effect of exposure factors.

$$\text{Baseline Exposure Values} = (2.521 \times 10^{-6}) \times (AADT^{1.686}) \times (\text{Segment Length}^{\,0.358}) \qquad (1)$$

Where:

*AADT* = Annual Average Daily Traffic (vehicles per day), and
*Segment Length* = study corridor length (miles).

**Step 2:** Using Equation 2 or Table 2.6, determine the effect from the roadway cross-section.

$$\text{Effect from Roadway} = \exp\,[1.098 \times MedianTWLTL{:}Four.Travel.Lanes - (0.898 \times MedianTWLTL) - (1.631 \times Four.Travel.Lanes) - (0.469 \times Speed.Limit.over.35)] \qquad (2)$$

Where:

*MedianTWLTL* = 1 if a two-way left-turn lane is present (0 value if not)
*Four.Travel.Lanes* = 1 if segment has 4 through lanes (2 lanes in each direction) or a value of zero if the segment has only 2 lanes (1 lane in each direction)
*Speed.Limit.over.35* = 1 if the speed limit is greater than 35 mph and zero if the speed limit is 35 mph or less

Table 2.6: Table of Possible Cases of the Effect of Roadway at Urban Environments

| | **Case 1: Speed Limits up to 35 mph** | | **Case 2: Speed Limits above 35 mph** | |
|---|---|---|---|---|
| **Median Type \ # of Lanes** | Two Travel Lanes | Four Travel Lanes | Two Travel Lanes | Four Travel Lanes |
| TWLTL Median | 0.4074 | 0.2391 | 0.2549 | 0.1496 |
| Other types of Medians or No Median Present | 1.0000 | 0.1957 | 0.6256 | 0.1225 |

**Step 3:** Using Equation 3, compute the effect of the driveways.

$$\text{Effect from Roadside/Driveways} = \exp\,[0.058 \times (Com.and.Ind.DW - 2.259 \times Other.DW)] \qquad (3)$$

Where:

*Com.and.Ind.DW* = number of commercial plus industrial driveways
*Other.DW* = number of driveways that are not commercial or industrial (Note: *Com.and.Ind.DW* + *Other.DW* = Total Driveways)

**Step 4:** Using Equation 4, multiply the results from Steps 1-3 to obtain the expected number of crashes for the study segment.

$$\textit{Predicted Number of Crashes} = (\textit{Baseline Exposure Values}) \times (\textit{Effect from Roadway}) \times (\textit{Effect from Roadside / Driveways}) \quad (4)$$

This procedure provides the expected number of crashes for an urban corridor based primarily on driveway configuration, land use, median configuration, traffic volume, and corridor length. A sample application is included in Appendix B of this manual.

**Rural Crash Prediction Model Computational Tools**

To predict the number of segment crashes for rural arterial locations, the following information is needed:

- Length of the road segment to analyze (in miles),
- AADT for the segment,
- In this case, the model is specified for speed limits of either 50 or 55 mph only,
- Cross-section information: Number of travel lanes,
- Total number of driveways in the segment, regardless of kinds of land use,
- Total number of driveways dedicated to Industrial land use. Total number of clusters of closely located driveways. A ‘cluster of closely located driveways’ is defined as the set of driveways such that the distance between two consecutive driveways on one side of the street can be traveled in 1.5 seconds or less. This distance is 121 feet and 110 feet for roads with speed limits of 55 mph and 50 mph, respectively.

Following the same general estimation of crashes methodology, outlined in the urban procedure, complete the following four steps:

**Step 1:** Using Equation 5, compute the baseline effect of exposure factors.

$$\textit{Baseline Exposure Values} = (3.418 \times 10^{-3}) \times (\textit{AADT}^{\,0.7825}) \times (\textit{Segment Length}^{\,0.2864}) \quad (5)$$

Where:

*AADT* = Annual Average Daily Traffic (vehicles per day), and
*Segment Length* = study corridor length (miles).

**Step 2:** Using Equation 6 or Table 2.7, determine the effect from the roadway cross-section.

$$\textit{Effect from Roadway} = \exp\,[0.7862 \times \textit{Four.Travel.Lanes}] \quad (6)$$

Where:

*Four.Travel.Lanes* = 1 if segment has 4 through lanes (2 lanes in each direction) or a value of zero if the segment has only 2 lanes (1 lane in each direction)

Table 2.7: Possible Cases of the Effect of Roadway at Rural Environments

| *Two Travel Lanes* | *Four Travel Lanes* |
|---|---|
| 1.0000 | 2.1950 |

**Step 3:** Using Equation 7, compute the effect of driveways.

$$Roadside.effect = \exp[(1.2918 \times Prop.of.Ind.DW) + (0.1048 \times Total.\#.Clusters)] \; / \; (Total.\#.Driveways + 0.5)^{0.2864} \qquad (7)$$

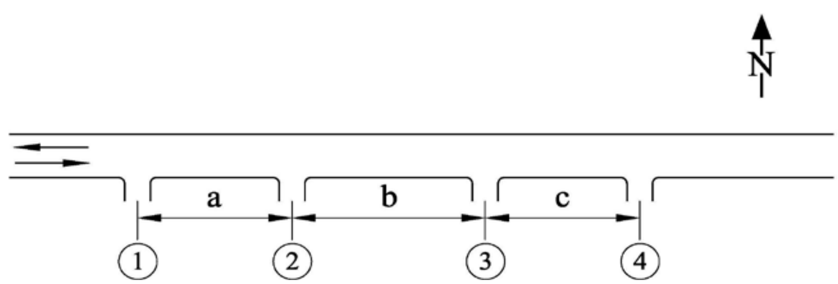
Where:

*Prop.of.Ind.DW* = proportion of industrial driveways (number of industrial driveways divided by the total number of driveways),

*Total.#.Clusters* = number of directional driveway clusters with a 1.5 second travel time (see Figure 2.4 and Figure 2.4 for example directional driveway cluster calculations), and

*Total.#.Driveways* = number of individual driveways (all land uses) located in the study corridor.

**Case I. Driveways on Only One Side of the Road (based on 1.5 second spacing)**

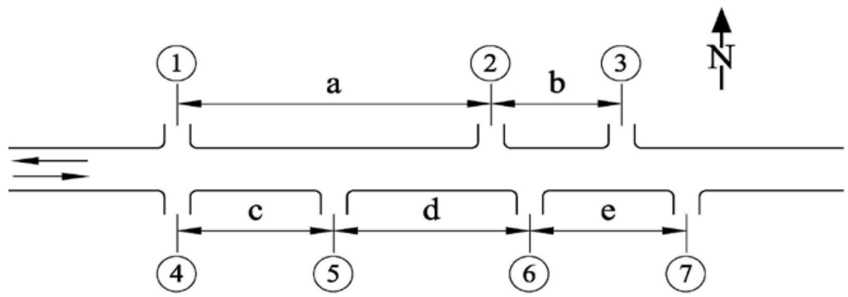



*Example Calculation of Directional Clusters for Various Spacings:*

| a (ft) | b (ft) | c (ft) | # Drives | 50 mph Speed Limit | | | 55 mph Speed Limit | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | # Cluster | Clusters Noted | Comment | # Cluster | Clusters Noted | Comment |
| 140 | 200 | 160 | 4 | 4 | 1, 2, 3, 4 | All > 110’ | 4 | 1, 2, 3, 4 | All > 121’ |
| 115 | 200 | 160 | 4 | 4 | 1, 2, 3, 4 | All > 110’ | 3 | 1-2, 3, 4 | a < 121’ |
| 80 | 115 | 125 | 4 | 3 | 1-2, 3, 4 | a < 110’ | 2 | 1-2-3, 4 | a & b < 110’ |
| 80 | 105 | 115 | 4 | 2 | 1-2-3, 4 | a & b < 110’ | 1 | 1-2-3-4 | All < 121’ |
| 80 | 105 | 90 | 4 | 1 | 1-2-3-4 | All < 110’ | 1 | 1-2-3-4 | All < 121’ |

Figure 2.3: Calculations for Rural Directional Driveway Clusters (One Side of Road)

**Case II. Driveways on Both Sides of the Road (based on 1.5 second spacing)**




*Example Calculation of Directional Clusters for Various Spacings:*

| a (ft) | b (ft) | c (ft) | d (ft) | e (ft) | # Drives | 50 mph Speed Limit | | | 55 mph Speed Limit | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | # Cluster | Clusters Noted | Comment | # Cluster | Clusters Noted | Comment |
| 200 | 125 | 130 | 125 | 150 | 7 | 7 | WB: 1,2,3 EB: 4,5,6,7 | All > 110’ | 7 | WB: 1,2,3 EB: 4,5,6,7 | All > 121’ |
| 200 | 115 | 130 | 125 | 150 | 7 | 7 | WB: 1,2,3 EB: 4,5,6,7 | All > 110’ | 6 | WB: 1,2-3 EB: 4,5,6,7 | b < 121’ |
| 200 | 105 | 120 | 125 | 150 | 7 | 6 | WB: 1,2-3 EB: 4,5,6,7 | b < 110’ | 5 | WB: 1,2-3 EB: 4-5,6,7 | b & c < 121’ |
| 200 | 105 | 105 | 105 | 150 | 7 | 4 | WB: 1,2-3 EB: 4-5-6,7 | b, c, & d < 110’ | 4 | WB: 1,2-3 EB: 4-5-6,7 | b, c, & d < 121’ |
| 120 | 90 | 90 | 95 | 105 | 7 | 3 | WB: 1,2-3 EB:4-5-6-7 | b, c, d, & e < 110’ | 2 | WB: 1-2-3 EB:4-5-6-7 | All < 121’ |
| 105 | 90 | 90 | 95 | 105 | 7 | 2 | WB: 1-2-3 EB:4-5-6-7 | All < 110’ | 2 | WB: 1-2-3 EB:4-5-6-7 | All < 121’ |

Figure 2.4: Calculations for Rural Directional Driveway Clusters (Both Sides of Road)

**Step 4:** Using Equation 4 , multiply the results of Steps 1-3 to obtain the expected number of crashes for the study segment.

This procedure provides the expected number of crashes for a rural corridor based primarily on driveway configuration, land use, number of lanes, traffic volume, and corridor length. A sample application is included in Appendix C of this manual.

*Operational Effectiveness*

In addition to potential safety impacts, the strategic spacing of unsignalized access points can help improve vehicle egress capacity. Table 2.8 depicts the various spacing criteria considered to enhance operations. Generally, the egress capacity would dictate greater spacing at speeds above 30 mph; however, at a minimum the access spacing should accommodate stopping sight distance.

Table 2.8: Comparison of Various Unsignalized Spacing Criteria

| Operating Speed (mph) | Stopping Sight Distance [1] (ft) | Intersection Sight Distance [2] (ft) | Right-turn Entry [3] (ft) | Influence Distance [4] (ft) | Egress Capacity [5] (ft) |
|---|---|---|---|---|---|
| 30 | 200 | 335 | 185 | 380 | 315 |
| 35 | 250 | 390 | 245 | 405 | 450 |
| 40 | 305 | 445 | 300 | 460 | 625 |
| 45 | 360 | 500 | 350 | 530 | 850 |
| 50 | 425 | 555 | N/A [6] | 620 | 1125 |
| 55 | 495 | 610 | N/A [6] | 725 | N/A [6] |

[1] From AASHTO, 2011 (p. 3-4, Table 3-1), level terrain, rounded up to nearest 5 ft
[2] From AASHTO, 2011 (p. 9-38, Table 9-6), conservatively assumes left-turns
[3] From Stover and Koepke, 1988 (p. 109), uses “preferable” spacing
[4] From Gluck, Levinson, and Stover, 1999 (p. 55), spillback rate of 2-percent, shorter distances for higher spillback assumptions
[5] Adapted from methodology presented by Stover and Koepke, 2002 (, p. 6-22)
[6] No value given

Table 2.9 further demonstrates that as right-turn volumes increase at unsignalized intersections, the number of through vehicles located in the curb lane that were affected by turning vehicles also increase.

Table 2.9: Percentage of Through Vehicles at a Single Driveway as Right-Turn Volume Increases

| Right-Turn Volume Entering Driveway (vehicles per hour) | Percent of Through Vehicles Affected (%) |
|---|---|
| ≤ 30 | 2.4 |
| 31 to 60 | 7.5 |
| 61 to 90 | 12.2 |
| > 90 | 21.8 |

*Source: NCHRP Report 420 (Gluck, Levinson, & Stover, 1999)*

Additional unsignalized access point operations as they relate to free-flow speed are included in the content summarized in Section 2.1.2.1.

*Corner Clearance for Driveway Placement*

Corner clearance is the distance between the extended curb line at an intersection and the edge of the nearest driveway. Inadequate corner clearance can contribute to operational and safety problems. Minimum corner clearance values should be based on the intersection functional distance (this is the influence area of the intersection where approaching vehicles decelerate and queue). The AASHTO *Green Book* (2011) indicates that "Ideally, driveways should not be located within the functional area of an intersection or in the influence area of an adjacent driveway." These values vary based on approach speed and lane configurations. A summary of these values is depicted in Table 2.10, but expected vehicle storage length should be added to the perception reaction plus maneuver distances shown for the desirable as well as limiting conditions.

Table 2.10: Upstream Functional Intersection Area, Excluding Storage

| | **Desirable Conditions** | | **Limiting Conditions** | |
|---|---|---|---|---|
| **Speed** (mph) | **Maneuver Distance** [3,4] (ft) | **Perception Reaction** [1] **Plus Maneuver Distance** [2,4] (ft) | **Maneuver Distance** (ft) | **Perception Reaction** [2] **Plus Maneuver Distance** [3,5] (ft) |
| 20 | 70 | 130 | 70 | 100 |
| 25 | 110 | 185 | 105 | 140 |
| 30 | 160 | 250 | 145 | 190 |
| 35 | 215 | 320 | 190 | 240 |
| 40 | 275 | 395 | 245 | 305 |
| 45 | 345 | 475 | 300 | 365 |
| 50 | 425 | 570 | 365 | 440 |
| 55 | 510 | 670 | 435 | 515 |
| 60 | 605 | 780 | 510 | 600 |
| 65 | 710 | 900 | 590 | 685 |
| 70 | 820 | 1025 | 680 | 785 |

[1] 2.0 second perception-reaction time
[2] 10 mph speed differential, 5.8 $fps^2$ deceleration while moving from the through lane into the turn lane; 6.8 $fps^2$ deceleration after completing lateral shift into the thru lane.
[3] 10 mph speed differential, 5.8 $fps^2$ deceleration while moving from the through lane into the turn lane; 9.2 $fps^2$ deceleration after completing lateral shift into the thru lane.
[4] 1.0 second perception-reaction time
[5] Assumes turning vehicle has "cleared" the through lane and the following vehicle can pass without encroaching upon the adjacent through lane; this is assumed to be possible when the turning vehicle has moved laterally at least 9 ft.

*Source: Stover and Koepke (2002)*

**Driveway Connections Summary**

*Performance Measures:*

- Crash Frequency
- Percent of Through Vehicles Affected by Turning Movements

Required Data:

- Length of road segment
- Traffic Volume (AADT)
- Speed Limit
- Number of Travel Lanes and Presence of a TWLTL Median
- Number of Driveways and Type of Land Use Serviced
- Driveway placement or proximity to each other
- Crash History (3-year minimum recommended)
- Stopping Sight Distance (as a minimum)
- Right-turning Volume Entering Driveway
- Corner Clearance Distance
- Intersection Functional Area

#### 2.1.2.3 ROUNDABOUTS

Roundabouts represent a potential solution for intersections that typically have numerous conflict points. Though not appropriate for all situations, roundabouts may offer potential access management benefits by providing another alternative to direct left-turn movements at intersections. Potential benefits attributable to roundabouts range from increased safety and vehicular capacity (up to 50-percent) to additional items including reduced fuel consumption, improved air quality, lower cost, aesthetics, convenient U-turns, and traffic calming (*Ewing, 1999*).

*Safety Assessment*

One common concern related to the construction of modern roundabouts in the United States is that they are confusing for unfamiliar users and can pose potential safety risks. Following a brief period where drivers learn to navigate the roundabouts, the overall result is that they actually improve safety by removing the likelihood of more severe crashes typically associated with intersection left-turn maneuvers.

Table 2.11 provides a summary table of expected changes in crash severity attributed to modern roundabout construction in the United States. The use of multi-lane roundabouts may increase the likelihood of sideswipe crashes; however, single-lane roundabouts are known to decrease most crash types with the possible exception of rear end collisions. Due to the large variance in expected roundabout safety performance, a range of crash reductions and companion crash modification factors is depicted in Table 2.11.

Table 2.11: Expected Reduction in Crash Frequency at Roundabouts

| Crash Severity Level | Percent Reduction in Crashes Compared to Traditional Intersections | Crash Modification Factor |
|---|---|---|
| Property Damage Only | 30 to 35 | 0.65 to 0.70 |
| Injury | 63 to 88 | 0.12 to 0.37 |
| Fatality | 90 | 0.10 |

*Sources: (Myers, 1999; Jacquemart, 1998;USDOT, 2004; Ariniello, 2004)*

*Operational Effectiveness*

The construction of roundabouts is appropriate at moderate traffic volume locations that could benefit from reduced delay. Improved operations occur when driver expectancy is realized, so the construction of consecutive roundabouts along a corridor can be expected to further enhance traffic operations. The associated performance measurements would include a decrease in the average speed as well as a reduced overall travel time (*Arineiello, 2004*).

Roundabouts can provide a 30- to 50-percent increase in traffic capacity (when contrasted to a traditional intersection) with a volume increase from 800 to 1,200 vehicles per lane (*USDOT, 2004*).

**Roundabouts Summary**

*Performance Measures:*

- Crash Frequency
- Corridor Travel Time
- Operating Speed

Required Data:

- Traffic Volume (AADT) for Major and Minor Roads
- Crash History (3-year minimum recommended) including Crash Severity Level
- Periodic travel time studies

## 2.2 INTERCHANGES

Interchanges facilitate access between high performance arterial streets and crossroads. As a result, commercial land use development frequently occurs in the regions near interchanges. A lack of access management on crossroads in the vicinity of interchanges can result in safety and operational deficiencies. The placement of intersections and driveways in the immediate proximity of ramp termini for interchanges coupled with heavy weaving volumes can result in frequent crashes and operational constraints. One common strategy to help mitigate safety and operational problems near interchanges is to increase separation distances between intersections or driveways and interchanges. The following sections review known safety and operational issues as they relate to interchanges and access management.

*Safety Assessment*

As the distance between the first access points and interchange terminals increases, the number of interchange-related crashes will decrease (*Flintsch, 2008*). Table 2.12 uses a variety of traffic volume values to demonstrate how the number of expected crashes per year decreases as the initial access point location and its proximity to the interchange increases.  Similarly, the number of crashes increases as the traffic volume increases. As a result, an increase in the minimum access spacing from 300 to 600 feet equates to a 50 percent reduction in the crash rate.

Table 2.12: Expected Number of Crashes per Year per Miles as a Function of the Access Section Length and AADT

| L (ft) | AADT (veh/day) | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 5,000 | 10,000 | 15,000 | 20,000 | 25,000 | 30,000 | 35,000 | 40,000 | 45,000 | 50,000 | 75,000 |
| 0 | | | | | | | | | | | |
| 50 | 19.98 | 36.28 | 51.42 | 65.86 | 79.80 | 93.35 | 106.59 | 119.57 | 132.32 | 144.88 | 205.35 |
| 100 | 17.99 | 32.67 | 46.30 | 59.31 | 71.86 | 84.06 | 95.99 | 107.67 | 119.15 | 130.46 | 184.92 |
| 150 | 16.20 | 29.42 | 41.69 | 53.40 | 64.71 | 75.70 | 86.43 | 96.95 | 107.29 | 117.47 | 166.51 |
| 200 | 14.59 | 26.49 | 37.55 | 48.09 | 58.27 | 68.16 | 77.83 | 87.30 | 96.62 | 105.78 | 149.94 |
| 250 | 13.14 | 23.85 | 33.81 | 43.30 | 52.47 | 61.38 | 70.08 | 78.62 | 87.00 | 95.25 | 135.02 |
| 300 | 11.83 | 21.48 | 30.44 | 38.99 | 47.25 | 55.27 | 63.11 | 70.479 | 78.34 | 85.77 | 121.58 |
| 350 | 10.65 | 19.34 | 27.41 | 35.11 | 42.54 | 49.77 | 56.83 | 63.75 | 70.54 | 77.24 | 109.48 |
| 400 | 9.59 | 17.42 | 24.69 | 31.62 | 38.31 | 44.82 | 51.17 | 57.40 | 63.52 | 69.55 | 98.58 |
| 450 | 8.64 | 15.68 | 22.23 | 28.47 | 34.50 | 40.36 | 46.08 | 51.69 | 57.20 | 62.63 | 88.77 |
| 500 | 7.78 | 14.12 | 20.02 | 25.64 | 31.06 | 36.34 | 41.49 | 46.54 | 51.51 | 56.39 | 79.94 |
| 550 | 7.00 | 12.72 | 18.02 | 23.09 | 27.97 | 32.72 | 37.36 | 41.91 | 46.38 | 50.78 | 71.98 |
| 600 | 6.31 | 11.45 | 16.23 | 20.79 | 25.19 | 29.47 | 33.64 | 37.74 | 41.77 | 45.73 | 64.82 |
| 650 | 5.68 | 10.31 | 14.61 | 18.72 | 22.68 | 26.53 | 30.30 | 33.98 | 37.61 | 41.18 | 58.37 |
| 700 | 5.11 | 9.28 | 13.16 | 16.86 | 20.42 | 23.89 | 27.28 | 30.60 | 33.87 | 37.08 | 52.56 |
| 750 | 4.61 | 8.36 | 11.85 | 15.18 | 18.39 | 21.51 | 24.57 | 27.56 | 30.49 | 33.39 | 47.33 |
| 800 | 4.15 | 7.53 | 10.67 | 13.67 | 16.56 | 19.37 | 22.12 | 24.81 | 27.46 | 30.07 | 42.62 |
| 850 | 3.73 | 6.78 | 9.61 | 12.31 | 14.91 | 17.44 | 19.92 | 22.34 | 24.73 | 27.07 | 38.37 |
| 900 | 3.36 | 6.10 | 8.65 | 11.08 | 13.43 | 15.71 | 17.94 | 20.12 | 22.27 | 24.38 | 34.55 |
| 950 | 3.03 | 5.50 | 7.79 | 9.98 | 12.09 | 14.15 | 16.15 | 18.12 | 20.05 | 21.95 | 31.12 |
| 1000 | 2.73 | 4.95 | 7.02 | 8.99 | 10.89 | 12.74 | 14.54 | 16.31 | 18.05 | 19.77 | 28.02 |
| 1050 | 2.46 | 4.46 | 6.32 | 8.09 | 9.80 | 11.47 | 13.10 | 14.69 | 16.26 | 17.80 | 25.23 |
| 1100 | 2.21 | 4.01 | 5.69 | 7.29 | 8.83 | 10.33 | 11.79 | 13.23 | 14.64 | 16.03 | 22.72 |
| 1150 | 1.99 | 3.61 | 5.12 | 6.56 | 7.95 | 9.30 | 10.62 | 11.91 | 13.18 | 14.43 | 20.46 |
| 1200 | 1.79 | 3.25 | 4.61 | 5.91 | 7.16 | 8.37 | 9.56 | 10.73 | 11.87 | 13.00 | 18.42 |
| 1250 | 1.61 | 2.93 | 4.15 | 5.32 | 6.45 | 7.54 | 8.61 | 9.66 | 10.69 | 11.70 | 16.59 |
| 1300 | 1.45 | 2.64 | 3.74 | 4.79 | 5.80 | 6.79 | 7.75 | 8.70 | 9.63 | 10.54 | 14.94 |
| 1350 | 1.31 | 2.38 | 3.37 | 4.31 | 5.23 | 6.11 | 6.98 | 7.83 | 8.67 | 9.49 | 13.45 |
| 1400 | 1.18 | 2.14 | 3.03 | 3.88 | 4.71 | 5.51 | 6.29 | 7.05 | 7.80 | 8.55 | 12.11 |
| 1450 | 1.06 | 1.93 | 2.73 | 3.50 | 4.24 | 4.96 | 5.66 | 6.35 | 7.03 | 7.69 | 10.91 |
| 1500 | 0.96 | 1.73 | 2.46 | 3.15 | 3.82 | 4.46 | 5.10 | 5.72 | 6.33 | 6.93 | 9.82 |

*Source: Rakha et al., 2008*

<u>Operational Effectiveness</u>

Operationally based access spacing at interchanges is depicted in Table 2.13. This table includes Oregon-based recommendations for minimum access spacing near freeway interchanges. The table indicates, for example, that the nearest major signalized intersection on both sides of the interchange should be located at least 1320 feet from the interchange.

Table 2.13: Suggested Minimum Access Spacing Standards for Two and Four-Lane Cross Routes at Freeway Interchanges, Oregon

| **Access Type** | **Area Type** | | |
|---|---|---|---|
| | Fully Developed Urban (45 mph) | Suburban (45 mph) | Rural (55 mph) |
| Two-lane Cross Roads | | | |
| First Access (ft) | 750 | 990 | 1,320 |
| First Major Signalized Intersection (ft) | 1,320 | 1,320 | 1,320 |
| Four-lane Cross Roads | | | |
| First Access from Off-Ramp (ft) | 750 | 990 | 1,320 |
| First Median Opening (ft) | 990 | 1,320 | 1,320 |
| First Access Before On-Ramp (ft) | 990 | 1,320 | 1,320 |
| First Major Signalized Intersection (ft) | 2,640 | 2,640 | 2,640 |

*Source: (Layton, 1996)*

**Interchange Access Management Summary**

*Performance Measures:*

- Crash Frequency
- Corridor Travel Time/Delay
- Operating Speed

Required Data:

- Traffic Volume (AADT)
- Distance from Interchange to Access Location
- Crash History (3-year minimum recommended) including Crash Severity Level
- Periodic travel time studies

## 2.3 AUXILIARY LANES

The addition of turning lanes or bays is an effective strategy for enhancing safety and operations by relocating slowing or stopped turning vehicles out of the path of through vehicles. In addition to turn lanes at median crossovers, there is a need for both left-turn and right-turn lanes at signalized and unsignalized locations. More information is available for left-turn lanes than for right-turn lanes; however, many of the observations associated with the left-turn lanes or bays similarly apply to the right-turn lanes.

Safety Assessment

Intersection turning maneuvers can contribute to congestion and delay and may require more complex multi-phase traffic signal timing at locations with high turning volumes. One common method for addressing left and right turns, particularly at intersection locations, is to provide exclusive turn lanes. An alternative method for accommodating left turns is to include shared lanes that are occupied by turning vehicles as well as through vehicles. For any given traffic cycle, the presence of five or more left turning vehicles located in a shared lane will preempt the safe practical use of that lane (*Levinson, 1989*). As a result, access management can be enhanced through the use of exclusive turn lanes at signalized and unsignalized access locations.

AASHTO's *Highway Safety Manual* (HSM) includes combined assessments of the body of literature as it relates to turn lanes through the use of crash modification factors (CMF). A CMF is a multiplicative adjustment value that can be used to estimate the number of crashes. The base condition (circumstance prior to the construction of the turn lane) is an absence of any turn lanes. If a CMF has a value of 1.0 then that treatment is assumed to have no real influence on safety (thus multiplying the number of estimated crashes by 1.0). On the other hand, if a CMF is less than 1.0 as shown in Table 2.14 for left-turn lanes and Table 2.15 for right-turn lanes, then the treatment is expected to reduce crashes. These values translate into a reduction in intersection crashes ranging from 7 percent for signalized three-leg urban/suburban arterial left-turn lanes up to 55 percent for fatal and injury crashes at rural multilane unsignalized three-leg intersection locations. Right-turn lanes exhibit more modest crash reductions ranging from 6 percent up to 23 percent in crash reductions.

Table 2.14: Left-turn Lane Crash Modification Factors

| **Major Road Type** | **3-leg Intersection** | | **4-leg intersection** | |
|---|---|---|---|---|
| | **Unsignalized** | **Signalized** | **Unsignalized** | **Signalized** |
| Rural two-lane highway | 0.56 | NA | $0.72^n$ | $0.82^n$ |
| Rural multilane highway | | | | |
| Total Crashes | 0.56 | NA | $0.72^n$ | NA |
| Fatal and Injury Only | 0.45 | NA | $0.65^n$ | NA |
| Urban/suburban arterial | $0.67^n$ | $0.93^n$ | $0.73^n$ | $0.90^n$ |

NA: not available
n = number of non-STOP-controlled intersection approaches with left-turn lanes.
For example, a four-leg STOP-controlled rural two-lane highway intersection with left-turn lanes on both major street approaches will have a CMF of $(0.72)^2$, or 0.52. This value can be interpreted as a 48 percent reduction in the total number of intersections crashes.

*Source: Adapted from AASHTO, 2010*

Table 2.15: Right-turn Lane Crash Modification Factors

| Major Road Type | 3-leg Intersection | | 4-leg intersection | |
|---|---|---|---|---|
| | **Unsignalized*** | **Signalized** | **Unsignalized*** | **Signalized** |
| Rural two-lane highway | 0.86 | NA | $0.86^n$ | $0.96^n$ |
| Rural multilane highway | | | | |
| Total Crashes | 0.86 | NA | $0.86^n$ | NA |
| Fatal and Injury Only | 0.77 | NA | $0.77^n$ | NA |
| Urban/suburban arterial | $0.86^n$ | $0.96^n$ | $0.86^n$ | $0.96^n$ |

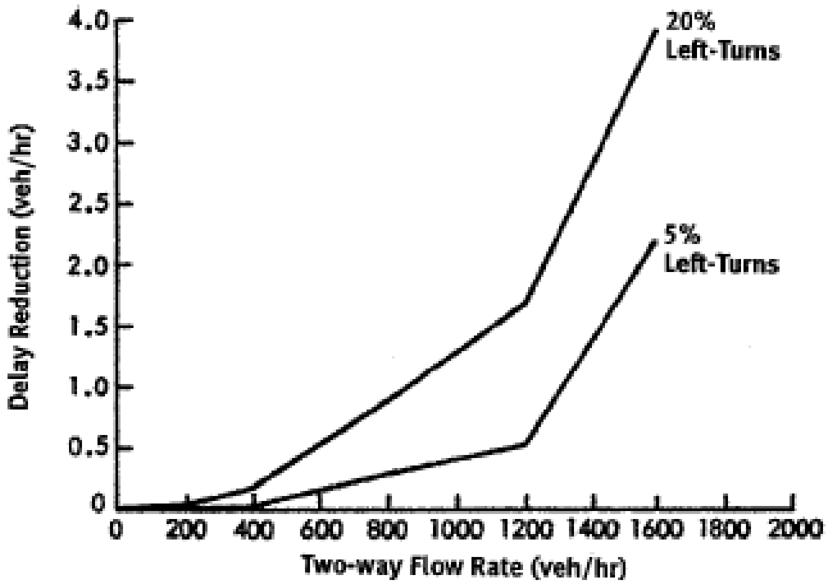
*Unsignalized refer to locations with STOP signs located only on the minor approaches
NA: not available
n = number of intersection approaches without STOP sign control that have right-turn lanes.
For example, a 4-leg unsignalized intersection with right-turn lanes on both major street approaches will have a CMF of $(0.86)^2$, or 0.74. This value can be interpreted as a 26 percent reduction in the total number of intersection crashes.
Note: STOP controlled approaches are not considered when determining the number of approaches with right turn lanes.

*Source: Adapted from AASHTO, 2010*

*Operational Effectiveness*

Though the use of exclusive turn lanes is expected to enhance safety, the inclusion of these lanes can also influence delay at intersections. The installation of left-turn lanes, for example, removes these vehicles from through lanes and ultimately reduces the delay for vehicles in adjacent lanes. For right-turn lanes, the vehicle does not have to cross the path of approaching vehicles and so the benefits for right-turn lanes occur primarily at locations with high right turn volumes or locations where right-turn-on-red is permitted.

Figure 2.5 demonstrates that constructing exclusive left-turn lanes at road segments with 20 percent left-turning vehicles results in a greater reduction in delay than would be achieved at locations with only 5 percent left-turning vehicles (*Harwood & Hoban, 1997*). Stover and Koepke (*2000*) found, for example, that adding a left-turn lane on four-lane roads could help to increase capacity by as much as 25 percent.




*Source: (Harwood and Hoban, 1997)*
Figure 2.5: Delay Savings of Left-Turn Lanes on Two-Lane Rural Highways

Gluck, Levinson, and Stover (1999) developed at table for NCHRP Report 420 based on the 1994 *Highway Capacity Manual* that includes capacity information for two-lane and four-lane roads with various left-turn treatments as shown in Table 2.16.

Table 2.16: Capacity Implications of Shared and Exclusive Left-Turn Lanes

| Condition | Two-Lane Road (vphpl) | Four-Lane Road (vphpl) |
|---|---|---|
| No Left-Turns | 840 | 1,600 |
| Shared Through/Left-Turn Lane Left-Turns/Hour: | | |
| 50 | 650 | 1,000 |
| 100 | 500 | 960 |
| 150 | 425 | 900 |
| Exclusive Left-Turn Lane | | |
| Unsignalized | 960 | 1,100 |
| Left-Turn Phase | 750-800 | 1,250-1,460 |
| Note: Computation assumes 60-90 cycle, 50 percent green plus clearance time per cycle, 3 seconds lost time, and 1,900 vehicles per hour per lane (vphpl) saturation flow. | | |

*Source: (Gluck, Levinson, & Stover, 1999)*

**Auxiliary (Turn Lane) Summary**

*Performance Measures:*

- Crash Frequency
- Delay

Required Data:

- Major Road Type
- Intersection Configuration and Traffic Control
- Peak Hour Traffic Volume
- Percent Left and Right Turns
- Crash History (3-year minimum recommended) including Crash Severity Level

## 2.4 MEDIAN TREATMENTS

One of the most common access management strategies is the use of median treatments to help channelize traffic flow conditions and consequently reduce vehicle conflicts. In general, a divided roadway is assumed to have a median with a raised, non-traversable median, though a traversable (flush) median is also a common median alternative. One common traversable median option is the two-way left-turn lane (TWLTL). Median configurations, in general, remove left-turning vehicles from the adjacent active travel lane, thereby reducing the likelihood of turning conflicts between vehicles in the same direction of travel. Non-traversable medians, however, further separate opposing traffic from turning vehicles or errant straight vehicles. Non-traversable medians, when constructed an adequate width, also provide refuge for pedestrians crossing the street.

At locations with TWLTL configurations, left-turning movements are permitted and are distributed along the roadway segment, whereas roads with raised medians concentrate the left-turn movements at median break locations such as signalized intersections or mid-block crossings. Two common median analysis options include: (1) modifying undivided highways to add median configurations, and (2) comparing the safety benefits of TWLTLs versus raised medians. This comparison is often based on the number of adjacent driveways and the demand for left turn movements.

Safety and operational aspects of raised medians, TWLTL configurations, and median openings are addressed in the following sections.

### *2.4.1 RAISED (NON-TRAVERSABLE) MEDIANS*

The installation of a raised or non-traversable median can restrict left-turn maneuvers from local access points as well as help to streamline motor vehicle operations and provide additional refuge opportunities for pedestrians. Since the raised median physically separates opposing directions of travel, it introduces unique safety and operational characteristics to the corridor. A raised median is often contrasted to an undivided configuration. Table 2.17 depicts common safety and operational aspects and the “preferred” option as defined in the literature. For comparisons between the raised median and the TWLTL, refer to Section 2.4.2. More specific performance characteristics as they relate to traffic safety and operations are presented in the following sections.

Table 2.17: Comparison of Different Alternatives Median Treatments

| | **"Preferred" Mid-block Left-turn Treatment [1]** |
|---|---|
| **Comparison Factor** | **Raised Median vs. Undivided** |
| Safety effects | |
| Vehicle crash frequency | Raised median |
| Pedestrian crash frequency | Raised median |
| Turning driver misuse/misunderstanding of markings | Raised median |
| Design variations can minimize conflicts (e.g. islands) | Raised median |
| Positive guidance (communication to motorists) | Raised median |
| Operation effects | |
| Major street through movement delay | Raised median |
| Major street left-turn movement delay | Raised median |
| Minor street left and through delay (two-stage entry) | Raised median |
| Pedestrian refuge area | Raised median |
| Operational flexibility | Undivided |

[1] The "Preferred" left-turn treatment is based on the findings of the research and more commonly found opinion during a review of the literature.

*Source: Adapted from Bonneson & McCoy, 1997*

<u>Safety Assessment</u>

Non-traversable medians physically separate opposing vehicles and so restrict left-turning movements at mid-block locations; therefore, the installation of a raised or non-traversable median can enhance access management by reducing the number of conflicts at driveway locations. This type of median can then generate an improved traffic flow for the corridor as well as an overall reduction in crashes when contrasted to similar undivided highways. Over the years, safety comparisons of raised median facilities contrasted to undivided facilities have consistently demonstrated that the raised medians provide significant opportunities for reductions in crashes. Though other factors such as number of access points will directly influence the enhanced safety performance, a recent study in Utah demonstrated that a reduction of 25 percent could be expected for total crashes, while the number of severe crashes could be reduced by as much as 36 percent following the conversion from an undivided road to one with a raised median (Schultz et al., 2011). This finding is consistent with a large body of literature dating back to the early 1980’s (*Parker, 1983; City of Arlington, 1983; New York State DOT, 1984; Murthy,1992, Long, Gan, & Morrison, 1993; Bowman & Vecellio, 1994; Harwood et al., 1995; Frawley, 2004*).

Section 2.6 includes combined effects information regarding raised medians and other corridor characteristics.

*Operational Effectiveness*

A raised median can extend continuously along a corridor or may be strategically positioned to compliment access management needs at select locations. Table 2.18 depicts the expected influence that the median configuration will have on free-flow speed conditions.

Table 2.18: Restrictive (Raised) Median Type Operational Influence

| **Percent with Restrictive Median (%)** | **Reduction in Free-flow Speed Based on Cross Section and Curb (mph)** | |
|---|---|---|
| | **No Curb*** | **Curb** |
| 20 | -0.3 | 0.9 |
| 40 | -0.6 | 1.4 |
| 60 | -0.9 | 1.8 |
| 80 | -1.2 | 2.2 |
| 100 | -1.5 | 2.7 |

*Speeds reflected in the "No Curb" option increase (thus the negative sign for reduced speeds)

*Source: Adapted from HCM Exhibit 17-11 (TRB, 2010)*

**Raised (Non-traversable) Median Summary**

*Performance Measures:*

- Crash Frequency
- Free-flow Speed

Required Data:

- Median Configuration
- Presence of Curb
- Percent of Corridor with Restrictive Median
- Crash History (3-year minimum recommended) including Crash Severity Level

### 2.4.2 TWO-WAY LEFT-TURN LANES

The performance of TWLTLs varies depending on access density as well as corridor traffic volume. TWLTLs permit the use of a center lane for left turns in both directions of travel. The ability for drivers of left-turning vehicles to wait for a gap in opposing traffic without obstructing a through lane makes these traversable medians attractive for business owners. Typically, continuous left-turn lanes transition into a conventional left-turn lane at major intersection locations. The TWLTL is often contrasted with the undivided roadway as well as the raised median configuration. Table 2.19 demonstrates how the TWLTL performs when compared to alternative median treatments.

Table 2.19: Comparison of TWLTL to Alternatives Median Treatments

| Comparison Factor | "Preferred" Mid-block Left-turn Treatment [1] | |
|---|---|---|
| | **Raised Median vs. TWLTL** | **TWLTL vs. Undivided** |
| Safety effects | | |
| Vehicle crash frequency | Raised median | TWLTL |
| Pedestrian crash frequency | Raised median | ND |
| Turning driver misuse/misunderstanding of markings | Raised median | Undivided |
| Design variations can minimize conflicts (e.g. islands) | Raised median | TWLTL |
| Positive guidance (communication to motorists) | Raised median | ND |
| Operation effects | | |
| Major street through movement delay | ND | TWLTL |
| Major street left-turn movement delay | ND | TWLTL |
| Minor street left and through delay (two-stage entry) | ND | TWLTL |
| Pedestrian refuge area | Raised median | ND |
| Operational flexibility | TWLTL | ND |

Note: ND = negligible difference or lack of a consensus of opinion on this factor.
[1] The "Preferred" left-turn treatment is based on the findings of the research and more commonly found opinion during a review of the literature.

*Source: Adapted from Bonneson & McCoy, 1997*

A TWLTL is the appropriate median treatment for the following four conditions (*Bonneson & McCoy, 1997; Parsonson, 1990; Gluck, Levinson, & Stover, 1999; Stover &Koepke, 2000*):

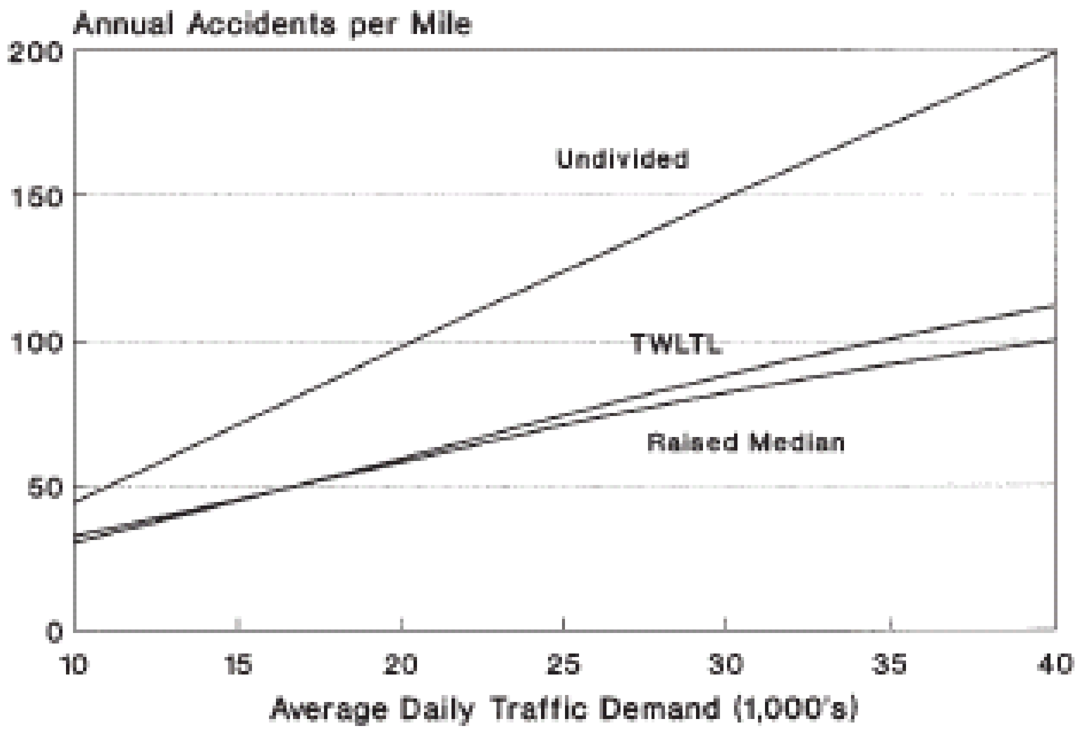
- The current and projected ADT for the facility is 24,000 vpd or less;
- Corridors are located in developing residential areas where individual properties obtain access from a local street connected to the particular facility;
- Corridors are located in developing suburban areas where direct access is needed to small adjacent properties; and
- Corridors are located in developed urban and suburban areas where crash history suggests that a raised median would be unlikely to improve safety along the particular corridor.

The following sections review the safety and operational expectations of these traversable medians.

*Safety Assessment*

A wide variety of studies, dating back to the 1970's, have focused on the performance of TWLTLs when compared to undivided roads. Most of these studies were before-after evaluations or comparison site studies. The corridors with TWLTLs generally resulted in a reduction in the number of crashes, when contrasted to undivided facilities. Expected reductions in crashes, when converting undivided roads to TWLTL corridors, can range from 21 to 41 percent (*Busbee, 1974; Committee #10 of the Southern Section ITE, 1975; Burritt & Coppula, 1978; Walton, Horne, & Fung, 1978; Thakkar, 1984; ITE, 1986; Box, 1989, and Bowman & Vecellio, 1994; Stover & Koepke, 2000*).

Figure 2.6 graphically demonstrates the expected safety performance of TWLTL configurations when contrasted to undivided corridors and raised median treatments. As shown, safety benefits of raised medians and TWLTL configurations are more significant at higher traffic volume conditions (*Bonneson & McCoy, 1997*).




*Source: (Bonneson and McCoy, 1997)*
Figure 2.6: Predicted Average Crash Frequency Comparison

## <u>*Operational Effectiveness*</u>

As a general rule, the construction of TLWLT treatments contributes to smooth traffic operations by removing left-turning vehicles from the through lanes. The free-flow speed is the most common operational performance measure used for assessing the TWLTL. As shown in Table 2.20, an undivided facility with similar characteristics is expected to have an average free-flow speed approximately 1.6 mph below that of a corridor with TWLTL median treatments.

Table 2.20: Free-flow Speed Reduction for TWLTL and Undivided Medians

| Median Type | Reduction in Free-flow Speed (mph) |
|---|---|
| TWLTL | 0.0 |
| Undivided | 1.6 |

*Source: Adapted from HCM Exhibit 14-10 (TRB, 2010)*

**TWLTL Median Summary**

Performance Measures:
- Crash Frequency
- Free-flow Speed

Required Data:
- Median Configuration
- Traffic Volume (ADT)
- Crash History (3-year minimum recommended) including Crash Severity Level

## 2.5 U-TURNS

The use of U-turns for access management can be accommodated in several ways. One of the most common methods is the restriction of left-turns at driveways followed by the use of U-turns (at the next intersection or at median crossovers) as alternatives to direct left turns. A second common U-turn treatment is the use of directional crossovers instead of bidirectional crossovers at median locations. Both of these U-turn applications are reviewed in the following sections.

*Safety Assessment*

***U-Turns as Alternatives to Direct Left-Turns***

U-turns may be used to reduce conflicts by redirecting left turns from driveway access to intersection locations. U-turn laws vary between states. In Oregon, U-turns are prohibited unless specifically indicated at the following locations: intersections with a traffic signal, between intersections on highways within the limits of an incorporated city, and any location where the turning vehicle is not clearly visible to approach cars (see ORS § 811.365 - Illegal U-turn – 2011 Oregon Revised Statutes). Because the prohibition of left-turn movements at driveways may result in increased left-turn volumes at intersections and contribute to longer left-turn phases, U-turns are commonly used to divert these left-turning vehicles at intersection or median locations.

On average, replacing driveway left turns with right turns plus U-turns will reduce the crash rate by approximately 20 percent (*Gluck, Levinson, & Stover, 1999; Zhou et al., 2000*).

***Directional Crossovers as Alternatives to Bidirectional Crossovers***

In addition to redirecting left-turn movements to nearby U-turns, another strategy is to restrict turning movements in the median or at intersection locations by using directional crossovers rather than bidirectional crossovers.

Safety performance at median locations where bidirectional crossovers are replaced with directional crossovers can be expected to have an approximate 24 to 32 percent reduction with a substantial portion of this reduction being the more severe head-on and angle crashes (*Scheuer & Kunde, 1996; Taylor et al, 2001*).

At signalized intersections with directional U-turn median crossovers, a 50 percent lower crash rate can be expected than for a conventional intersection (*Castronovo et al., 1998*). The observed crash rate reductions can be expected to increase as the number of signalized intersections per mile also increases.

<u>*Operational Effectiveness*</u>

The use of U-turns as an alternative to direct left-turns can directly affect roadway capacity and travel time. The use of directional U-turns in place of TWLTL configurations can be expected to increase corridor capacity from 15 to 50 percent, depending on the number of left-turning vehicles and prevailing traffic conditions (*Savage, 1974; Stover 1990; Koepke & Levinson, 1993; Maki, 1996*). Replacing left turns with right turns plus median U-turns experience much less average waiting delay than for road segments with direct left turns (*Zhou et al., 2000; Lu et al., 2005*).

**U-Turn Summary**

<u>*Performance Measures:*</u>

- Crash Frequency
- Capacity & Average Waiting Delay

<u>Required Data:</u>

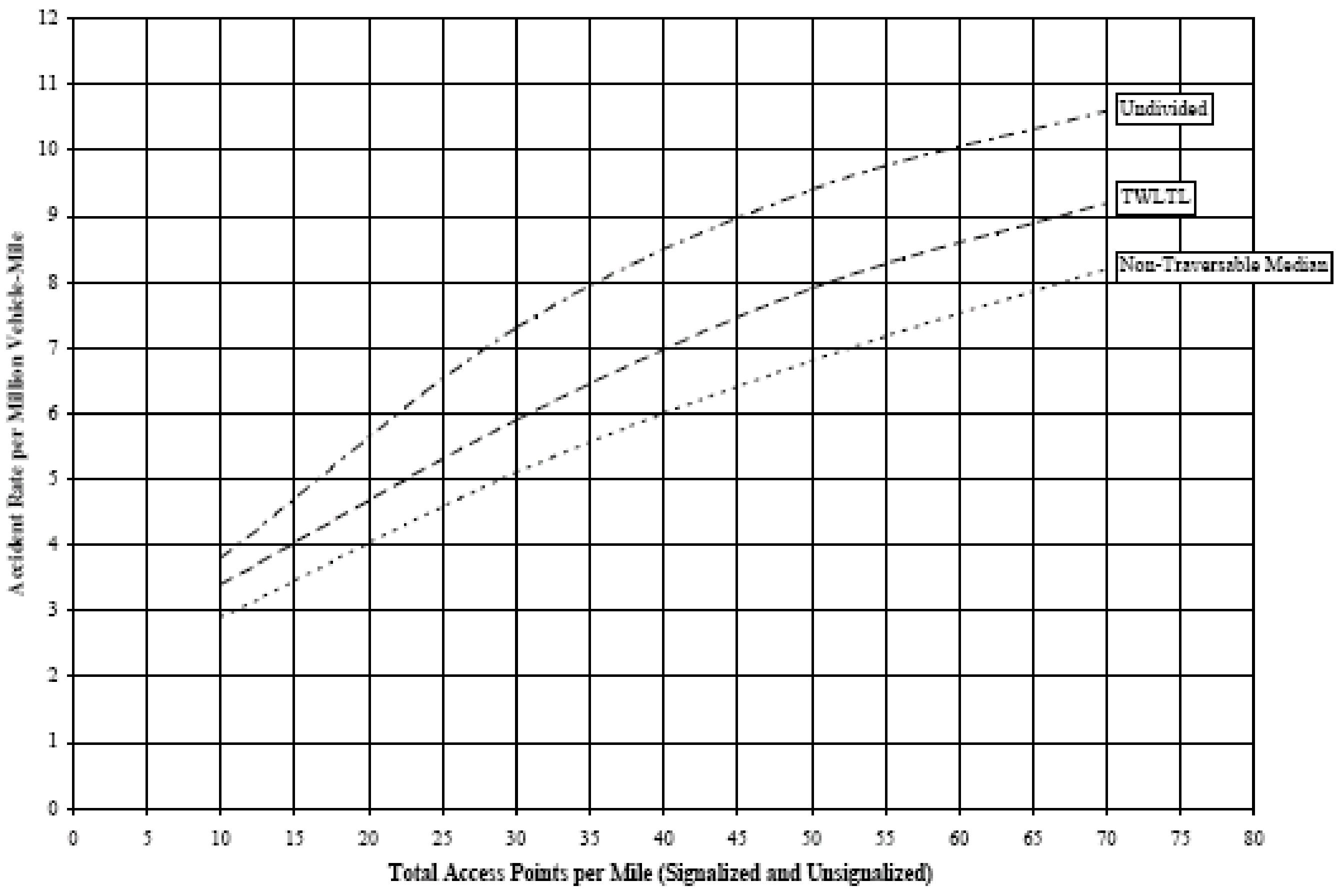
- U-Turn Configuration
- Traffic Volume (ADT)
- Crash History (3-year minimum recommended) including Crash Severity Level
- Periodic Delay Studies

## 2.6 COMBINED EFFECTS

The safety and operational effects of access management techniques are not always independent of other physical site characteristics such as traffic volume, access/driveway density, or similar. For example, although raised medians are generally safer than TWLTLs, TWLTLs tend to perform better operationally along corridors with high driveway densities and low-to-medium traffic volumes (*Margiotta & Chatterjee, 1995*). The following sections review these known combined effects of access management treatments.

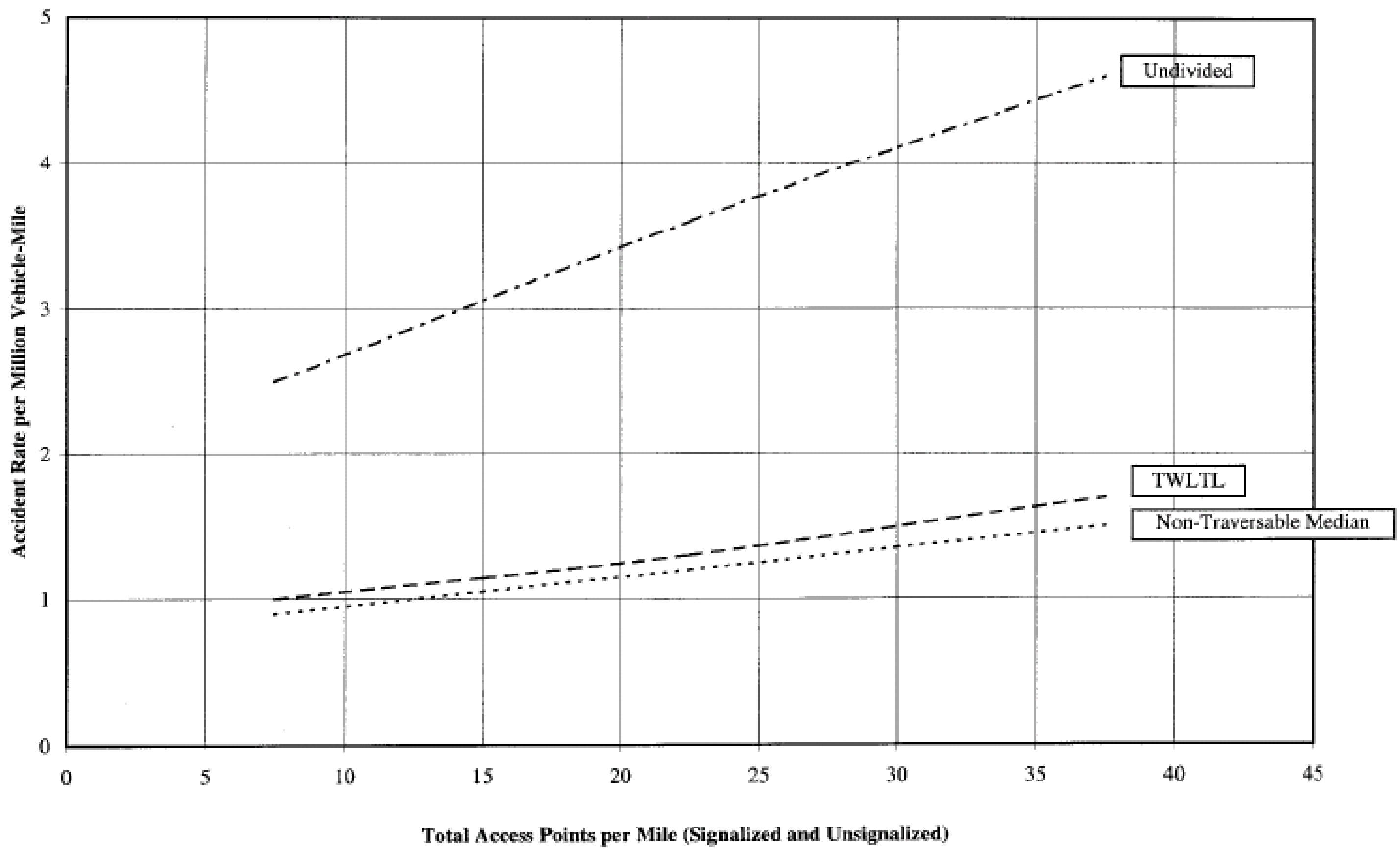
<u>*Safety Assessment*</u>

Due to the land use differences between urban and suburban corridors when contrasted to rural locations, median type, location, and access points per mile collectively can influence corridor safety. Figure 2.7 and Figure 2.8 illustrate these expected access density and crash rate relationships (*Gluck, Levinson, & Stover, 1999*). Figure 2.7 shows predicted crash rates by median type and total access density for urban and suburban roadways. Each access point would increase the annual crash rate by about 0.11 to 0.18 crashes per million VMT on undivided highways and by 0.09 to 0.13 on highways with TWLTLs or non-traversable medians. Figure 2.8 shows estimated crash rates by type of median for rural facilities. Each access point is expected to increase the annual crash rate by 0.07 crashes per million VMT on undivided highways, and 0.02 crashes per million VMT on highways with TWLTLs or non-traversable medians.




*Source: NCHRP Report 420 (Gluck, Levinson, & Stover, 1999)*

Figure 2.7: Estimated Crash Rates by Type for Median (Urban and Suburban Areas)




*Source: NCHRP Report 420 (Gluck, Levinson, & Stover, 1999)*

Figure 2.8: Estimated Crash Rates by Type for Median (Rural Facilities)

<u>*Operational Effectiveness*</u>

TWLTLs and non-traversable medians reduce delays, especially when the roads experience large traffic volumes (*Bonneson & McCoy, 1997*). Table 2.21 demonstrates that TWLTLs and raised medians have less annual delay than for undivided roadways. The combined effects include median type, traffic volume, and number of driveways per mile.

Table 2.21: Annual Delay to Major Street Left-Turn and Through Vehicles (hours/year)

| Driveways/Mile | Undivided | TWLTL | Raised Median |
|---|---|---|---|
| | | ADT = 22,500 vpd | |
| 30 | 2,200 | 1,300 | 1,300 |
| 60 | 2,200 | 1,400 | 1,400 |
| 90 | 2,200 | 1,400 | 1,400 |
| | | ADT = 32,500 vpd | |
| 30 | 7,100 | 3,000 | 3,100 |
| 60 | 7,800 | 3,200 | 3,500 |
| 90 | 8,000 | 3,200 | 3,400 |

Note: Assumes 10-percent left-turns, 1320-foot segment, and four through lanes

*Source: Adapted from NCHRP Report 395 (Bonneson and McCoy, 1997)*

**Combined Effects Summary**

*Performance Measures:*

- Crash Frequency or Crash Rate
- Delay

Required Data:

- Median Configuration
- Traffic Volume (ADT)
- Total Number of Access Points per Mile
- Number of Driveways per Mile
- Intersection Traffic Control
- Rural versus Urban Location
- Crash History (3-year minimum recommended) including Crash Severity Level

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# 3.0 DATA NEEDS

As noted in Section 2.0, a variety of data elements are recommended for use in assessing the continuing performance of access management measures. These elements can be further divided into three general categories: physical site characteristics, operational characteristics, and safety characteristics as shown in Table 3.1. Though each data element may not be needed to evaluate the performance of a specific access management strategy, the systematic acquisition of this information will enable an agency to, over time, establish a straightforward procedure for assessing and justifying access management decisions.

Table 3.1: Key Data Elements for Assessing Access Management Performance Measures

| Type | Data Elements |
|---|---|
| Physical Site Characteristics | • Location (Urban vs. Rural)<br>• Access Management Configuration (Median type or presence, U-turn orientation, Intersection configuration)<br>• Land use type and location of access points (number of signals per mile, unsignalized public intersections per mile, roundabouts, driveways per mile)<br>• Number of through lanes in each direction of travel<br>• Length of road segment (if applicable)<br>• Required and available stopping sight distance<br>• Intersection functional area<br>• Major road type<br>• Type of traffic control devices<br>• Presence of Curb<br>• Percent corridor with restrictive median (if applicable) |
| Operational Characteristics | • Traffic Volume (AADT)<br>• Speed Limit<br>• Peak Hour Traffic Volume (only at select locations)<br>• Percent left-turns and right-turns<br>• Right-turning volume into driveway (if applicable)<br>• Travel time and delay (accomplished with annual studies) |
| Safety Characteristics | • Three year crash history (include severity as well as crash type data) |

An important underlying assumption regarding development of such a data needs plan is to note the origin of the performance measure and permit this data plan to evolve as assessment procedures change. For example, the focus of the performance measures in this manual has been on safety and operational characteristics. Economic assessment is a common evaluation procedure that typically occurs at construction and rehabilitation stages; however, this manual does not explicitly address the positive or negative impacts on the region.

In Section 2.0, a summary of performance measures and associated data needs are listed for the various treatments. These performance measures are further separated into safety and operational performance. This section of the manual briefly indicates the typical data needs required for intersection and segment locations. If, for example, a jurisdiction in Oregon should elect to focus on only a few specific type of access management treatment, then this summary will help narrow down the various data needs and associated performance measures

## 3.1 INTERSECTION DATA NEEDS

### *3.1.1 PERFORMANCE MEASURE SUMMARY*

Access management treatments may occur at intersection locations or they may be corridor-specific and apply to a segment of a road. For intersection-specific assessments, common access management technique reviewed in Section 2.0 include:

- Signalized Intersections,
- Traditional Public Street Intersections,
- Driveway Connections,
- Roundabouts, and
- Interchanges,

Each of these intersections is characterized by unique performance measures, but the list depicted in Table 3.2 represents the intersection-related performance needs.

Table 3.2: Intersection Access Management Performance Measures

| **Performance Measure** | **Signalized Intersection** | **Public Street Intersection** | **Driveway Connection** | **Roundabouts** | **Interchanges** |
|---|---|---|---|---|---|
| Crash Frequency or Crash Rate | ✓ | ✓ | ✓ | ✓ | ✓ |
| Crash Rate Index | | ✓ | | | |
| Travel Time and Delay | ✓ | | | ✓ | ✓ |
| Free-flow Speed | | ✓ | | | |
| Operating Speed | | | | ✓ | ✓ |
| Percent of Through Vehicles Affected by Turning Movements | | | ✓ | | |

As indicated in Section 2.0, the use of crash rate as a performance measurement technique is known to have limitations for variable traffic volume locations, so as safety assessment methods continue to evolve that enable the use of crash frequency as a performance measure instead of crash rate, Oregon agencies should try to make this transition. At this time, however, only a few of the access management methods can be evaluated for safety assessment using crash frequency. As a result, the use of crash rate should be considered until alternative assessment strategies are available.

### *3.1.2 Data Requirements Summary*

For each of the performance measures indicated in Section 3.1.1, a subset of data requirements can be identified to help establish long-term evaluation of the access management strategy.

Table 3.3: Intersection Data Requirements to Assess Performance

| **Data Requirement** | **Signalized Intersection** | **Public Street Intersection** | **Driveway Connection** | **Roundabouts** | **Interchanges** |
|---|---|---|---|---|---|
| Physical Site Characteristics | | | | | |
| Location (Urban vs. Rural) | | | ✓ | | |
| Signals per Mile | ✓ | ✓ | | | |
| Unsignalized Intersections per Mile | ✓ | ✓ | | | |
| Driveways per Mile | ✓ | ✓ | | | |
| Land Use Type at Driveway | | | ✓ | | |
| Intersection / Driveway Position and Proximity | | | ✓ | | ✓ |
| Number of Through Lanes per Direction | | ✓ | | | |
| Length of Road Segment | | | ✓ | | |
| Presence and Type of Median | | | ✓ | | |
| Required Stopping Sight Distance | | | ✓ | | |
| Corner Clearance distance | | | ✓ | | |
| Intersectional Functional Area | | | ✓ | | |
| Operational Characteristics | | | | | |
| Traffic Volume (AADT or VMT) | ✓ | | ✓ | ✓ | ✓ |
| Speed Limit | | | ✓ | | |
| Right-turning Volume Entering Driveway | | | ✓ | | |
| Travel time and delay (annual studies) | | | | ✓ | ✓ |
| Safety Characteristics | | | | | |
| Crash Data (three years minimum) | ✓ | ✓ | | ✓ | ✓ |

## 3.2 CORRIDOR OR SEGMENT DATA NEEDS

### *3.2.1 PERFORMANCE MEASURE SUMMARY*

Corridor or segment-specific access management techniques apply when a length of road must be treated and assessed. In Section 2.0, these techniques include:

- Auxiliary Lanes,
- Raised (Non-Traversable) Medians,
- Two-way Left-turn Lanes,
- U-turns, and
- Combined Effects.

Each of these corridor or segment treatments is characterized by unique performance measures as shown in Table 3.4.

Table 3.4: Corridor or Segment Access Management Performance Measures

| **Performance Measure** | **Auxiliary Lanes** | **Raised (Non-Traversable) Medians** | **Two-way Left-turn Lanes** | **U-turns** | **Combined Effects** |
|---|---|---|---|---|---|
| Crash Frequency or Crash Rate | ✓ | ✓ | ✓ | ✓ | ✓ |
| Travel Time and Delay | ✓ | | | ✓ | ✓ |
| Free-flow Speed | | ✓ | ✓ | | |
| Capacity | | | | ✓ | |

### *3.2.2 DATA REQUIREMENTS SUMMARY*

The corridor or segment-specific performance measures indicated in Section 3.2.1 require a subset of the overall data elements indicated in Table 3.1. These corridor or segment-specific data requirements are summarized in Table 3.5l

Table 3.5: Corridor or Segment Data Requirements to Assess Performance

| Data Requirement | Auxiliary Lanes | Raised (Non-Traversable) Medians | Two-way Left-turn Lanes | U-turns | Combined Effects |
|---|---|---|---|---|---|
| Physical Site Characteristics | | | | | |
| Location (Urban vs. Rural) | | | | | ✓ |
| Signals per Mile | | | | | ✓ |
| Unsignalized Intersections per Mile | | | | | ✓ |
| Driveways per Mile | | | | | ✓ |
| Presence and Type of Median | | ✓ | ✓ | | ✓ |
| U-turn Configuration | | | | ✓ | |
| Major road type | ✓ | | | | |
| Type of traffic control devices | ✓ | | | | ✓ |
| Presence of Curb | | ✓ | | | |
| Percent corridor with restrictive median | | ✓ | | | |
| Operational Characteristics | | | | | |
| Traffic Volume (AADT or VMT) | | | ✓ | ✓ | ✓ |
| Peak Hour Traffic Volume | ✓ | | | | |
| Percent left-turns and right-turns | ✓ | | | | |
| Travel time and delay (annual studies) | | | | ✓ | |
| Safety Characteristics | | | | | |
| Crash Data (three years minimum) | ✓ | ✓ | ✓ | ✓ | ✓ |

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# 4.0 DOCUMENTATION AND IMPLEMENTATION

To effectively evaluate access management technique performance, assessment of the strategy should be systematically incorporated into routine documentation techniques. Much of the physical site characteristic information can be acquired and documented at the time of construction. Subsequently, operational and crash assessment information can be acquired on an annual basis.

## 4.1 DOCUMENTATION

The explicit assessment of access management performance can help an agency justify expenditures as well as long-term access related decisions. In addition to the documentation of physical data at the time of construction, a few general steps can help to extend the assessment to future conditions. These are noted as follows:

- Document initial physical site characteristics,
- At time of design and construction, develop an "Investigations File" that will identify the site as one that requires continuing assessment. Include in this file, at a minimum, the following items:
    - Physical site characteristics,
    - Crash data for the three years preceding construction,
    - Field studies documenting free-flow speed, operating speed, and travel time and delay. Include in these operational studies information about percent of turning vehicles and their influence on the through traffic.
    - Establish an assessment schedule and responsible party for annual evaluations.
- Based on the schedule created in the initial investigations file record, establish periodic (annual) evaluations of the field studies.
- Following a three year period after construction, acquire the "after" crash data and use this information to assess continuing performance of the strategy.
- Document findings of performance in the investigations file and develop an annual report that would also incorporate these findings.

## 4.2 IMPLEMENTATION

The success of the performance measurement and evaluation of the access management techniques will require systematic processes. Though initially these may sound demanding, in fact most of the information identified in this documentation process is common and may be part of current processes. The assessment, however, should be incorporated as part of the overall process.

Though it may be necessary to identify a "champion" to initially establish and maintain this continuing performance assessment, once the process is established it can become part of a systematic process. The construction and evaluation of proposed access management techniques, however, is not effective if the information gained from this process is not then used to further improve access management applications in Oregon. As a result, a formal access management performance evaluation program should then include a feedback feature where the findings of these evaluations are then used to inform future funding decisions. The annual report recommended as the final document step could be used to help an agency begin this implementation process.

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# 6.0 APPENDIX A – RELATIVE RISK FACTOR PROCEDURE

The driveway or intersection relative risk factor is based on a procedure independently developed by Dr. Robert Layton, Dr. Karen Dixon, and Lacy Brown. The content that follows summarizes the procedure.

## 6.1 RISK RATING PROCEDURE FOR ASSESSING DRIVEWAY CONFIGURATIONS

Access management along major facilities, i.e. arterials and major collectors, relies on effective driveway configurations and associated median or channelization treatments to achieve safe, smooth arterial operations and adequate service to adjacent land use activities. One common safety consideration at driveway locations is the number and type of conflict points. Conflict analysis has been used for many years to subjectively determine the safety or complexity of operations at a site.

The purpose of this paper is to develop and apply a risk assessment method to analyze and evaluate conflicts for a variety of driveway configurations. In addition to issues associated with the physical location and configuration, potential angle of impact, relative speed of conflicting vehicles, driver perception-reaction type, and type of potential crash, the volume of traffic then can be used to further assess the probability of crashes at a specific driveway location.

Since the purpose of this paper is the development of a risk assessment rating for driveways (a rating not currently available), the authors have elected to simplify this initial effort by primarily focusing on motor vehicle interactions; however, non-motorized operations such as bicycle and pedestrian should ultimately be included and are peripherally addressed in this paper.

### *6.1.1 DETERMINE LOCATION AND LAYOUT*

To adequately assess the expected risks, a first step is to determine the spatial orientation and layout of the driveway configuration and associated conflict points. The location, orientation, and type of conflict should be defined as follows:

<u>Location</u> – develop a plan view of the location with key distances between conflict points;

<u>Orientation</u> – determine the relative orientation of the vehicle paths between conflict points in sufficient detail to determine the angles of impact of conflicting vehicles and to represent the nature of crashes that would occur at the location; and

<u>Type of Conflict</u> – establish descriptions for the various conflicts (i.e., crossing, merge, diverge, etc.).

### *6.1.2 DETERMINE THE INDIVIDUAL LEVEL OF CONFLICT VALUES*

The level of conflict for a specific point is a factor of the orientation of the conflicting vehicles, their associated operation speeds, and the expected level of protection based on the frequency and type of conflict and impact angle. To provide relative risk assessments, this analysis uses an extreme (severe) crash condition as a comparison crash. This "base" crash is a head-on collision at speeds of 55 mph (88 km/hr) or greater [referred to as HO-55 in subsequent discussion]. All other levels of conflict will ultimately be adjusted to equivalent HO-55 crashes.

The level of conflict, LC, is a function of the relative speeds between conflicting vehicles and their angle of impact and conflict type. The LC can be extended to an effective level of conflict, ELC, which represents the increased likelihood of exposure due to other conflicts in close proximity. The following sections summarize how the LC and the ELC can be derived.

#### 6.1.2.1 RELATIVE OPERATING SPEED

The kinetic or impact energy for a crash is a factor of the speed (or speed differences) and can be determined from the following well known relationship:

$$\text{Kinetic Energy} = \text{KE} = \frac{1}{2}mu^2 \qquad (1)$$

Where: m = mass of vehicle (variable – units will cancel so use consistent values) and u = velocity (ft/sec).

For the HO-55 crash condition, this equation can be modified as follows (where the 1.47 constant is used to convert mph to ft/sec units):

$$\text{KE}_{\text{HO-55}} = \frac{1}{2}m(1.47 \times 55)^2 = 3254m \qquad (2)$$

A speed adjustment factor, f $_{spd}$, can then be developed by contrasting the kinetic energy for the HO-55 to alternative relative speeds:

$$f_{spd} = \frac{KE_S}{KE_{HO-55}} = \frac{\frac{m}{2}(1.47 \times S)^2}{3254m} = \frac{S^2}{3025} \qquad (3)$$

where S = speed (mph).

If, for example, a vehicle travelling at 40 mph (64 km/hr) impacts another vehicle traveling in the same general direction at 30 mph (48 km/hr), the relative speed difference would be 10 mph (16 km/hr) and this relative speed would be directly associated with the resulting kinetic energy if the vehicles were involved in a crash. As a result, the relative speed for the crash can be used to determine the speed adjustment factor.

#### 6.1.2.2 CONFLICT ORIENTATION FACTOR

In a manner similar to procedures used for assigning costs to crashes, a severity factor based on crash type and vehicle orientation can be used to represent associated crash risk due to the conflict configuration. This conflict orientation factor, c, defines bicycle and pedestrian-involved crashes as extremely severe (c=1.0) followed by head-on crashes (c=0.8), right-angle crashes (c=0.6), sideswipe crashes (c=0.4), and rear-end crashes (c=0.3). The larger c value of 1.0 for the bicycle and pedestrian crashes is because these crashes are considered injury-related without regard to angle of impact. The use of a scale from zero to 1.0 enables a multiplicative comparison. For head-on, right-angle, sideswipe, and rear-end crashes, the weighted maximum injury severity code used in the report *Crash Cost Estimates by Maximum Police-Reported Injury Severity Within Selected Crash Geometries* (Council et al., 2005) has been adjusted to the 1.0 scale to represent the rounded "c" values shown.

#### 6.1.2.3 CALCULATING THE LEVEL OF CONFLICT

The value of the LC is based on a combination of the speed adjustment factor and the conflict orientation factor or $LC = f_{spd} \times c$. To demonstrate this calculation consider Figure 6.1, Alternative I. For the purposes of this discussion, the intersections shown in Figure 6.1 are presumed to be isolated and not directly influenced by intersections upstream or downstream of this location. For this sample configuration, a driveway intersects a road as a T-intersection. The road has a restrictive median so driveway movements are constrained to right-in right-out operations. This configuration can be contrasted to Alternative II where a median break provides full driveway access to and from both directions of travel. For Alternative I, the expected crash type at merge point "A" and diverge points "B" and "C" would be a rear-end crash (c=0.3). At merge point "D", however, rear-end as well as sideswipe crashes could be expected. Since the conflict orientation factor for a sideswipe crash is larger (c=0.4), it will be conservatively used for this analysis. Table 6.1 summarizes the type of conflict, speed, and summary calculations for Alternative I. The major and minor speeds shown in this table reflect typical operating speeds for maneuvers at driveway locations. The relative speed reflects the speed relationship between the major and minor movements. At locations where the angle of conflict approaches a right angle, the speed of the impact would be equivalent to the speed of the involved vehicles. As a result, at those locations the relative speed would be the same as the vehicle operating speed.

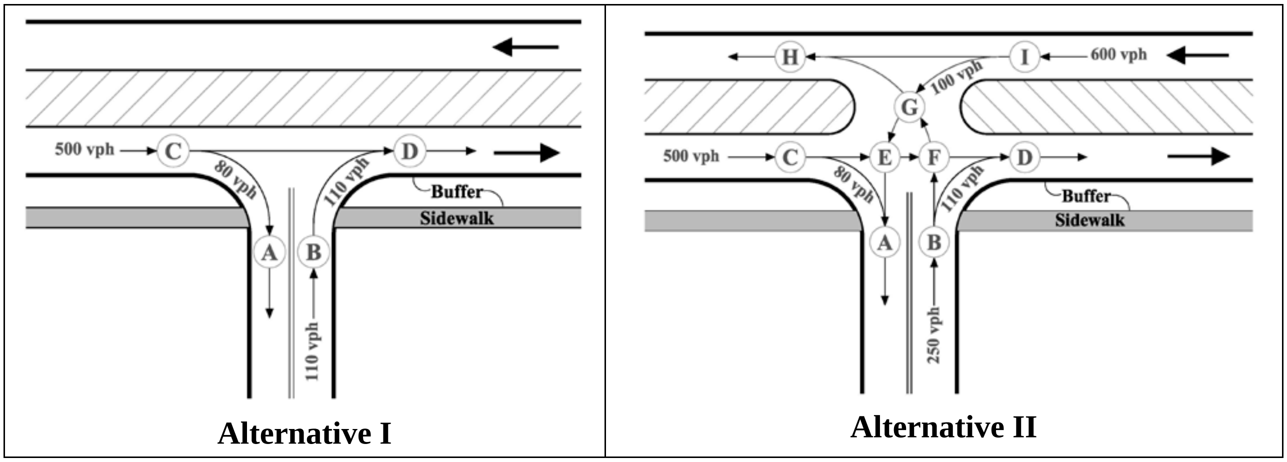



Figure 6.1: Alternative I and Alternative II Layouts and Volumes

Table 6.1: Alternative I Level of Conflict Calculations

| Conflict Point | Type | Major Speed, $S_{Major}$ (mph) | Minor Speed, $S_{Minor}$ (mph) | Relative Speed, S (mph) | $f_{spd}$ | c | LC | ELC |
|---|---|---|---|---|---|---|---|---|
| A | Rear-End | 15 | 15 | 15 | 0.074 | 0.3 | 0.022 | 0.022 |
| B | Diverge* | 0 | 10 | 10* | 0.033 | 0.3 | 0.010 | 0.010 |
| C | Diverge* | 50 | 15 (5**) | 45* | 0.669 | 0.3 | 0.201 | 0.401 |
| D | Merge | 50 | 10 | 40 | 0.529 | 0.4 | 0.212 | 0.212 |
| | | | | | | | $ELC_{INT}$ | 0.646 |

*Use larger speed

**Speed vector for 15 mph exiting vehicle along arterial at point of exit is approximately 5 mph.

### 6.1.2.4 NEARNESS INDEX

The nearness index, NI, addresses how closely oriented conflicts can introduce additional risk. For example, if three conflicts all occur at the same point, they are treated as three full conflicts. As the conflict points are pulled apart, the level of conflict decreases.

A driver should ideally have sufficient time and distance to deal with one conflict before encountering a second conflict. If the distance between conflicts is less than the associated stopping sight distance, the second conflict point will increase the level of conflict for the initial conflict point. For the purposes of this analysis, the authors have employed the perception-reaction time and deceleration rates from the American Association of State Highway and Transportation Officials (AASHTO) document *A Policy on Geometric Design of Highways and Streets* (*1*) [referred to as the *Greenbook* from this point forward]. A perception-reaction time of 2.5 seconds is assumed for the initial conflict and 1.5 seconds for each additional conflict. This assumes that the stopping sight distance would be based on a perception-reaction time of 4.0

seconds for two conflict points. If the driveway is located along a busy urban arterial where the driver must remain on high alert, the perception-reaction time could be moderately reduced. The deceleration rate is assumed to be 11.2 ft/sec$^2$ (3.4 m/sec$^2$) as adopted by the *Greenbook*.

To assess the nearness effect of conflict points, the distance between the conflict points ($d_{ab}$) as well as the stopping sight distance (SSD) between the two points should be determined. Though the SSD can be reduced to reflect the speed of operation at the second conflict point (if it is not zero), a conservative value assumes the full SSD. At locations where the $SSD_{ab}$ is greater than $d_{ab}$, the authors recommend the use of a nearness index, NI, to account for the fact that a driver does not have the ability to perceive and stop safely from one conflict point to the next conflict point, e.g. point “C” to point “A.” At locations where the $SSD_{ab}$ is less that $d_{ab}$, the NI is then equal to zero. The NI is based on the negative exponential for the distance to conflict point to SSD ratio as follows:

$$NI = e^{-\frac{d_{ab}}{SSD}} \qquad (4)$$

where $d_{ab}$=distance between conflict points a and b (ft or m) and SSD = stopping sight distance (ft or m). Using the SSD methods summarized in the *Greenbook* (*1*) and the conservative SSD value based on the highest conflicting speed, the authors developed a series of NI curves for the varying $d_{ab}$ and SSD values as shown in Figure 6.2. This figure demonstrates a non-linear relationship captured by the use of a negative exponential function and more closely represents the expected reduction in conflict influence as the conflict points are separated a distance up to the SSD value. Table 6.2 shows the distance and NI values for Alternative I.

#### 6.1.2.5 EQUIVALENT LEVEL OF CONFLICT

The total equivalent level of conflict, ELC, reflects the relative LC value for each conflict point combined with the influence of other closely spaced conflict points. As an example, the $ELC_C$ for Alternative I can be determined as follows:

$$ELC_C = LC_C + LC_D\,(NI_{CD}) + LC_A\,(NI_{CA}) = 0.201 + (0.212)(0.87) + (0.022)(0.69) = 0.401 \qquad (5)$$

At locations that do not have downstream conflicts (such as merge Point D), $ELC_D = LC_D$. As a final step in the ELC evaluation, the individual ELC values for each conflict should be added together. As shown in Table 1, the $ELC_{INT}$ = 0.646 or the sum of the individual ELC values for points A, B, C, and D. This $ELC_{INT}$ value can be interpreted as an aggregate equivalent conflict measure of 0.646 HO-55 crashes for the driveway configuration presented in Alternative I. The $ELC_{INT}$ value is based on vehicle interactions, relative speed, proximate conflict points, and impact angles. To fully assess risk, the traffic exposure must be considered. The following section reviews how the major and minor traffic volumes can be used to estimate the total number of conflicts and associated risk assessment at a known location.

Table 6.2: Alternative I Nearness Index Assessment

| Movement | Prevailing Speed, $S_0$ (mph) | Distance, d (ft) | $NI = e^{-\frac{d}{SSD}}$ |
|---|---|---|---|
| B to D | 0 (stopped) | 41 | -- |
| C to A | 15 | 41 | 0.69 |
| C to D | 50 | 74 | 0.87 |

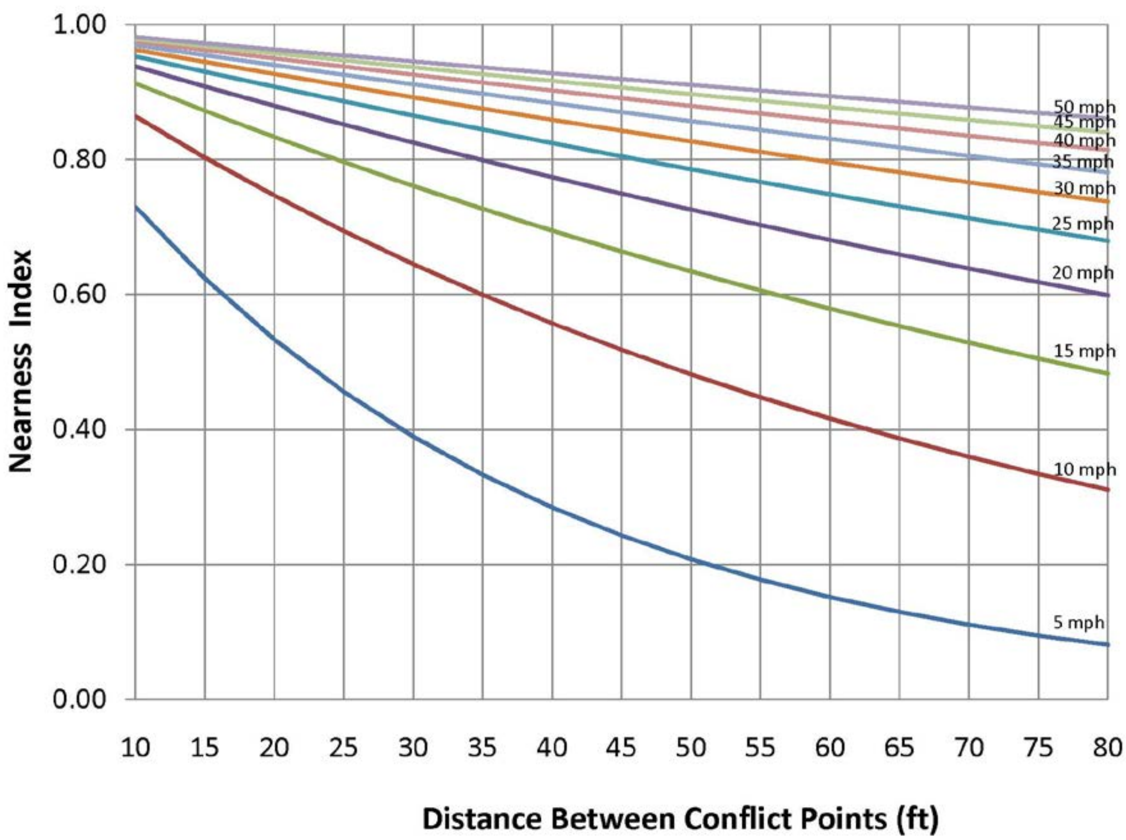



Figure 6.2: Nearness Index

### *6.1.3 VOLUME AND NUMBER OF CONFLICTS*

The expected number of conflicts that may occur should be based on the traffic exposure. For the purposes of this study, the authors have used the design hourly volume as a basis for identifying the magnitude of total conflicts. The conflicts occur when gaps in the major traffic stream are not large enough to accommodate the minor traffic stream operations of crossing, merging, or diverging. To determine the expected number of substandard gaps in the traffic stream, the probability that the required time (time to maneuver plus perception-reaction time) is less than the available time can be determined using the following relationship:

$$\Pr\left(t_{available} > t_{required}\right) = e^{-V_{major}\left(\frac{t_{required}}{3600}\right)} \tag{6}$$

Where $V_{major}$ = Major volume (vph) and $t_{required}$ = maneuver plus perception-reaction time in seconds

The number of gaps available along the arterial is then a factor of the probability of gaps and the major traffic volume. This value can be computed as:

$$\text{Gaps} = V_{major}\, e^{-V_{major}\left(\frac{t_{required}}{3600}\right)} \tag{7}$$

The proportion of movements that can occur successfully without a conflict can be determined as:

$$\text{Without Conflicts} = \frac{V_{major}\left[e^{-V_{major}\left(\frac{t_{required}}{3600}\right)}\right]}{V_{major}} = e^{-V_{major}\left(\frac{t_{required}}{3600}\right)} \tag{8}$$

The proportion of movements, therefore, that can be expected to experience conflicts resulting from minor movement vehicle operations is calculated as follows:

$$\text{Number of Conflicts} = N = V_{minor}\left[1 - e^{-V_{major}\left(\frac{t_{required}}{3600}\right)}\right] \tag{9}$$

Where $V_{minor}$ = Minor volume (vph).

The required time will differ for diverging, crossing, or merging conflict types. For the diverging conflict, the deceleration time plus the perception-reaction time should be included in the required time. Since we know the minor movement speed should be similar to the major movement speed minus the deceleration, we can represent this relationship as follows:

$$S_{minor} = S_{major} - \frac{(a)(t_{decel})}{1.47} \tag{10}$$

Where a = 11.2 ft/sec$^2$ and S = speed (mph).

The total diverging time, depicted in seconds, could then be determined by solving this equation for the deceleration time and adding the perception-reaction time and conversion factors as shown in the following equation:

$$t_{diverge} = \frac{1.47(S_{major} - S_{minor})}{a} + t_{pr} \tag{11}$$

Where $t_{pr}$ = perception-reaction time (seconds).

Table 6.3 shows this diverging time equation as well as the crossing and merging times used for estimating the number of conflicts. The deceleration rate of 11.2 ft/sec$^2$ is a conservative value as 90-percent of drivers generally decelerate at or above this rate (*1*). Table 6.3 also demonstrates the relationship of the major and minor traffic volume for the specific conflict configuration.

To demonstrate the estimate for the number of conflicts, consider diverging conflict point "C" for Alternative I. The time to diverge can be calculated as:

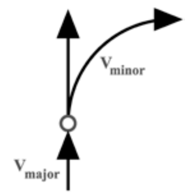
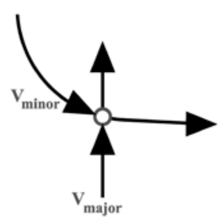
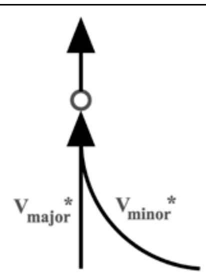
$$t_{diverge} = \frac{1.47(50-5)}{11.2} + 2.5 = 5.9 + 2.5 = 8.4 \text{ sec} \tag{12}$$

The number of conflicts can then be calculated as:

$$N = 80\left[1 - e^{-500\left(\frac{8.4}{3600}\right)}\right] = 55.1 \text{ conflicts/hr} \tag{13}$$

Once the number of conflicts for a specific location has been determined, this value can be combined with the ELC (previously identified) to determine the relative risk assessment index (RAI) for the specific conflict point as well as for the entire intersection. This RAI is developed in the following section.

Table 6.3: Time Estimation for Calculating the Number of Conflicts

| **Type** | **Schematic** | **Time, t (sec)** |
|---|---|---|
| Diverge |  | $t_d = \frac{1.47(S_{major} - S_{minor})}{a} + t_{pr}$ (14)<br><br>Where<br><br>$S_{major}$ = Major [arterial] speed (mph)<br><br>$S_{minor}$ = Minor [diverging] vehicle speed (mph)<br><br>$a$ = 11.2 ft/sec$^2$<br><br>$t_{pr}$ =perception reaction time (sec) |
| Crossing |  | $t_{crossing} = t_{maneuver} + t_{pr}$ (15)<br><br>Where<br><br>$t_{maneuver}$ = 6.5 sec. [Approximate crossing speed per Highway Capacity Manual (2) ]<br><br>$t_{pr}$ =perception reaction time (sec) |
| Merge |  | $t_{merge} = t_{acceleration} + t_{pr}$ (16)<br><br>Where<br><br>$t_{acceleration}$ = 2 to 4.5 (assume 3 ft/sec$^2$)<br><br>$t_{pr}$ =perception reaction time (sec) |

### *6.1.4 RISK ASSESSMENT INDEX*

The intersection risk assessment index (RAI) is determined by multiplying the individual expected level of conflict values at each point (this is the previously defined ELC) times the actual anticipated conflicting volumes at that point. This relationship can be demonstrated by the following equation:

$$RAI_{INT} = \sum_{x} \left( N_x \times ELC_x \right) \tag{17}$$

Where $N_x$ = number of conflicts at point x, and $ELC_x$ = expected level of conflict at point x. Both the value of ELC and RAI are unitless.

Table 6.4 summarizes the number of conflicts, ELC, and RAI for the intersection depicted in Alternative I.

Table 6.4: Number of Conflicts and Risk Assessment Index for Alternative I Intersection

| Conflict Point | Type | Relative Speed, S (mph) | t (sec) | Major Volume, $V_{Major}$ (vph) | Minor Volume, $V_{Minor}$ (vph) | Number of Conflicts, N (conflicts/hr) | ELC | Risk Assessment Index, RAI |
|---|---|---|---|---|---|---|---|---|
| A | Rear-end | 15 | 3+2.5=5.5 | 80 | 80 | 9.2 | 0.022 | 0.20 |
| B | Diverge | 10 | 1.3+2.5=3.8 | 110 | 110 | 12.1 | 0.010 | 0.12 |
| C | Diverge | 45 | 5.9+2.5=8.4 | 500 | 80 | 55.1 | 0.401 | 22.09 |
| D | Merge | 40 | 3+2.5=5.5 | 420 | 110 | 47.4 | 0.212 | 10.05 |
| | | | | | | | $RAI_{INT}$ | 32.46 |

The Alternative I $RAI_{INT}$ value of 32.46 can be interpreted that there are approximately 32 equivalent HO-55 conflicts per hour for the T-intersection configuration, conflict orientation, and traffic volume condition. This value could then be contrasted to other RAI values with different geometric configurations to determine the relative level of risk introduced by alternative design treatments.

To demonstrate how the RAI can vary for different design treatments, the following Alternative II example depicts a more complex configuration.

### *6.1.5 EXAMPLE -- ALTERNATIVE II*

In Figure 6.1, the authors introduced a T-intersection with a median break as a contrast to the Alternative I right-in right-out T-intersection design. The procedure used for Alternative I can be directly applied to the Alternative II configuration as demonstrated below.

### 6.1.5.1 STEP 1. CALCULATE THE LC FOR ALTERNATIVE II

To calculate the LC, first determine the type of conflict, relative speed, speed adjustment factor, and the conflict orientation factor as input into the LC calculations (see Table 6.5).

Table 6.5: Alternative II Level of Conflict Values

| Conflict Point | Type | Major Speed, $Speed_{Major}$ (mph) | Minor Speed, $Speed_{Minor}$ (mph) | Relative Speed, RS (mph) | $f_{spd}$ | c | LC | ELC |
|---|---|---|---|---|---|---|---|---|
| A | Merge | 15 | 15 | 15 | 0.074 | 0.4 | 0.030 | 0.030 |
| B | Diverge | 0 | 10 | 10 | 0.033 | 0.3 | 0.010 | 0.100 |
| C | Diverge | 50 | 15 (5**) | 45 | 0.669 | 0.3 | 0.201 | 1.334 |
| D | Merge | 50 | 10 | 40 | 0.529 | 0.4 | 0.212 | 0.212 |
| E | Crossing* | 50 | 15 | 50* | 0.826 | 0.6 | 0.496 | 1.203 |
| F | Crossing* | 50 | 10 | 50* | 0.826 | 0.6 | 0.496 | 0.864 |
| G | Crossing* | 20 | 20 | 20* | 0.132 | 0.6 | 0.079 | 0.669 |
| H | Merge | 50 | 20 | 30 | 0.298 | 0.4 | 0.119 | 0.119 |
| I | Diverge | 50 | 25 | 25 | 0.207 | 0.3 | 0.062 | 0.649 |
| | | | | | | | $ELC_{INT}$ | 5.180 |

*Use larger speed for kinetic energy calculations (LC estimate)

**Speed vector for 15 mph exiting vehicle along arterial at point of exit is approximately 5 mph.

### 6.1.5.2 STEP 2. DETERMINE NEARNESS INDEX (ASSUME COMBINED PERCEPTION-REACTION TIME OF 4 SECONDS)

Using the prevailing speed and the distance between the various conflict points, the NI can be calculated as shown in Table 6.6.

### 6.1.5.3 STEP 3. FIND THE EFFECTIVE LEVEL OF CONFLICT AT EACH POINT

To determine the $ELC_{INT}$, add the ELC values for each conflict point. Example ELC calculations are shown below for conflict point “C” and “H.” Since point “H” is a merge point without downstream conflict points, the value of $ELC_H = LC_H$.

$$ELC_C = LC_C+(LC_A)(NI_{CA})+(LC_E)(NI_{CE})+(LC_F)(NI_{CF})+(LC_D)(NI_{CD})$$

$$= 0.201+(0.030)(0.69)+(0.496)(0.95)+(0.496)(0.92)+(0.212)(0.87) = 1.334$$

$$ELC_H = LC_H = 0.119 \text{ (merge)}$$

Table 6.6: Alternative II Nearness Index Assessment

| Movement | Prevailing Speed, $S_0$ (mph) | Distance, d (ft) | $NI = e^{-\frac{d}{SSD}}$ |
|---|---|---|---|
| B to D | 0 (stopped) | 41 | -- |
| B to F | 0 (stopped) | 26 | -- |
| B to G | 15 | 36 | 0.73 |
| B to H | 25 | 72 | 0.71 |
| F to G | 15 | 10 | 0.91 |
| F to H | 25 | 46 | 0.80 |
| G to H | 25 | 36 | 0.84 |
| C to A | 15 | 41 | 0.69 |
| C to E | 50 | 32 | 0.95 |
| C to F | 50 | 42 | 0.92 |
| C to D | 50 | 74 | 0.87 |
| E to F | 50 | 10 | 0.98 |
| E to D | 50 | 42 | 0.93 |
| F to D | 50 | 32 | 0.95 |
| I to G | 25 | 36 | 0.84 |
| I to E | 25 | 46 | 0.80 |
| I to A | 25 | 72 | 0.71 |
| G to E | 20 | 10 | 0.94 |
| G to A | 20 | 36 | 0.80 |
| E to A | 15 | 26 | 0.80 |
| I to H | 50 | 80 | 0.86 |

#### 6.1.5.4 STEP 4. INTERPRETATION OF ALTERNATIVE II ELC VALUE

The conflicts for this driveway configuration have the aggregate equivalent conflict measure of 5.18 head-on collisions at a speed of 55 mph. The conflict rate without a median break (see Alternative I) was 0.646 so that would equate to 12.5% less risk at a location without the median opening as compared to the location where the median break is present (Alternative II).

#### 6.1.5.5 STEP 5. DETERMINE THE NUMBER OF CONFLICTS AND INTERSECTION RISK ASSESSMENT INDEX

Using the required time (maneuver plus perception-reaction time) and the major and minor traffic volumes, the number of conflicts for Alternative II is then multiplied by the previously calculated ELC values (see Table 6.5) to determine the RAI at each conflict point. These values are then totaled to develop the $RAI_{INT}$ for this location (see Table 6.7).

#### 6.1.5.6 STEP 6. INTERPRETATION OF THE $RAI_{INT}$

The $RAI_{INT}$ value of 314.23 can be interpreted as approximately 314 equivalent HO-55 conflicts per hour for the Alternative II T-intersection (with an uncontrolled median break). By comparison (to the Alternative I values), the inclusion of a median opening can increase the level of risk, based on equivalent HO-55 conflicts), by 314 divided by 32 or a value of 9.8 times that of a location with the controlled median design.

Table 6.7: Number of Conflicts and Risk Assessment Index for Alternative II Intersection

| Conflict Point | Type | Relative Speed, S (mph) | Required time, t (sec) | Major Volume, $V_{Major}$ (vph) | Minor Volume, $V_{Minor}$ (vph) | Number of Conflicts, N (conflicts/ hr) | ELC | Risk Assessment Index, RAI |
|---|---|---|---|---|---|---|---|---|
| A | Merge | 15 | 3+2.5=5.5 | 100 | 80 | 11.3 | 0.030 | 0.34 |
| B | Diverge | 10 | 1.3+2.5=3.8 | 250 | 110 | 25.5 | 0.100 | 2.55 |
| C | Diverge | 45 | 5.9+2.5=8.4 | 500 | 80 | 55.1 | 1.334 | 73.49 |
| D | Merge | 40 | 3+2.5=5.5 | 420 | 110 | 52.1 | 0.212 | 11.04 |
| E | Crossing | 50 | 6.5+2.5=9.0 | 420 | 100 | 65.0 | 1.203 | 78.20 |
| F | Crossing | 50 | 6.5+2.5=9.0 | 420 | 140 | 91.0 | 0.864 | 78.63 |
| G | Crossing | 20 | 6.5+2.5=9.0 | 140 | 100 | 29.5 | 0.669 | 19.76 |
| H | Merge | 30 | 3+2.5=5.5 | 500 | 140 | 84.0 | 0.119 | 10.00 |
| I | Diverge | 25 | 3.3+2.5=5.8 | 600 | 100 | 62.0 | 0.649 | 40.22 |
| | | | | | | | $RAI_{INT}$ | 314.23 |

### 6.1.6 *ADDITIONAL ISSUES FOR CONSIDERATION*

Often a location does not have discrete conflict points but is characterized by typical movements that could be considered conflict paths. For example, at locations with multiple lanes, vehicles may change lanes or enter alternative lanes (such as a continuous left-turn lane) at staggered locations, and the resulting conflict points may not be separated by fixed distances. The driver's decisions to alter path may be based on prevailing traffic, physical driveway locations, or other factors. These staggered conflict paths will have varying perception-reaction times as well as deceleration rates.

Additional conflict paths can be expected with increased pedestrian and bicycle activity. Pedestrians may cross driveways (approaching from the left and the right) or traverse the road at multiple locations. Bicycles also create a conflict point where they cross the lane or lanes at the driveway intersection. These conflicts should be directly considered in the evaluation of risk; however, if the bicycle operates within a bicycle lane that is oriented parallel to the vehicle lane, an additional conflict path (and potential distraction) is introduced between turning motor vehicles and non-turning bicycles. Future enhancements to the risk assessment rating could potentially expand to incorporate these less specific conflicting pathways.

### *6.1.7 CONCLUSIONS*

This paper introduces a structure for the risk assessment index rating tool for assessing conflicts at driveway locations. Historically the number of conflict points and their orientation has been used exclusively to compare the relative safety of proposed driveway construction configurations. This rating method extends the conflicts analysis using vehicle dynamics, site specific characteristics, and traffic volumes to provide a more comprehensive view of the relative safety of conflicts with various driveway configurations.

The authors have developed a measure of relative safety for conflict points that yields an estimate of the equivalent number of HO-55 collisions at each conflict point (the ELC value) as well as an intersection equivalent value ($ELC_{INT}$). The RAI then combines the equivalent number of head-on collisions at 55 mph with the expected number of conflicts at each point during the design hour to determine the aggregate RAI for the intersection. This value yields the expected number of equivalent HO-55 conflicts for the driveway per hour.

Ultimately, the ELC and RAI values should be used to evaluate the relative risk and effectiveness of various driveway configurations and designs. This information can then be useful in determine locations for driveways, median openings, intersections, and their associated orientations.

### *6.1.8 REFERENCES*

1. AASHTO (2004). *A Policy on Geometric Design of Highways and Streets.* Washington, DC, 896 pages
2. Council, F., E. Zaloshnja, T. Miller, and B. Persuad (2005). *Crash Cost Estimates by Maximum Police-Reproted Injury Severity Within Selected Crash Geometries.* Publication No. FHWA-HRT-05-051, U.S. Department of Transportation, Federal Highway Administration, McLean, VA.
3. Transportation Research Board (2000). *Highway Capacity Manual*. Washington, DC.

# 7.0 APPENDIX B – EXAMPLE URBAN CORRIDOR SAFETY PERFORMANCE PROCEDURE

(See Section 2.1.2.2 for procedure details)

The following example demonstrates how to use the urban corridor safety performance methodology. For this demonstration, a sample site is located in Redmond, Oregon as illustrated in Figure 7.1:


Figure 7.1: Sample Site -- Redmond, Oregon

Table 7.1 summarizes the required site information needed for the safety assessment.

Table 7.1: Sample Input for Urban Example Problem from Redmond, Oregon

| Urban Segment Features | Characteristics |
|---|---|
| Segment length | 0.12 miles |
| AADT | 24,800 vpd |
| Speed limit | 45 |
| Number of travel lanes | 4 |
| TWLTL median | Yes |
| Total commercial and industrial driveways | 7 |
| Total driveways for other land uses | 1 |

**Step 1: Compute the Baseline Effect of Exposure Factors using Equation 1.**

*Baseline Exposure Values* = $(2.521 \times 10^{-6}) \times (AADT^{1.686}) \times (Segment\ Length^{0.358})$

*Baseline Exposure Values* = $(2.521 \times 10^{-6}) \times (24{,}800^{1.686}) \times (0.12^{0.358}) = 30.26$

**Step 2: Select the adjustment factor for roadway design characteristics from Table 2.6.**

Since this segment has a speed limit above 35 mph, has a TWLTL median, and has 4 travel lanes, the adjustment factor should be 0.1496 (from Table 2.6).

**Step 3: Compute the effect of driveways using Equation 3**

*Effect from Roadside/Driveways = exp [0.058* x *(Com.and.Ind.DW - 2.259* x *Other.DW)]*

*Effect from Roadside/Driveways = exp [0.058* x *(7 - 2.259* x *1)] = 1.32*

**Step 4: Obtain the predicted number of crashes for the segment by multiplying all of the above results**

*Predicted Number of Crashes = (Baseline Exposure Values)* x *(Effect from Roadway)* x *(Effect from Roadside / Driveways)*

*Predicted Number of Crashes = 30.26* x *0.1496* x *1.32 = 5.9589 predicted crashes in 5 years*

| Example problem interpretation: |
|---|
| Based on exposure, roadway, and roadside characteristics we can predict that over a period of 5 years approximately 6 (rounded from 5.96) segment crashes are expected to occur. |

# 8.0 APPENDIX C – EXAMPLE RURAL CORRIDOR SAFETY

(See Section 2.1.2.2 for procedure details)

This section demonstrates how to use the rural corridor safety performance methodology. For this demonstration, a sample site is located on highway US 20, between Corvallis and Newport, is illustrated in Figure 8.1.

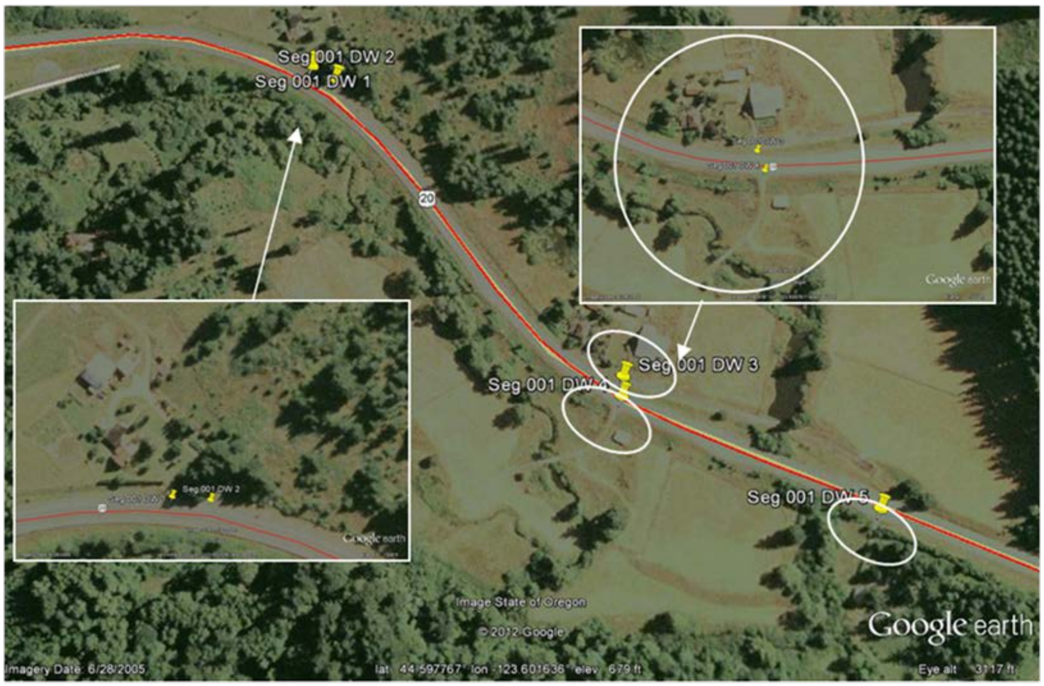



Figure 8.1: Sample Site -- Corvallis-Newport, Oregon

The required site information is summarized in Table 8.1.

Table 8.1: Sample Input for Rural Example Problem for Corvallis-Newport, Oregon

| Rural Segment Features | Characteristics |
|---|---|
| Segment length (MP 33.78 to 34.34) | 0.56 miles |
| AADT | 4,940 vpd |
| Speed limit | 55 |
| Number of travel lanes | 2 |
| Total driveways in segment | 5 |
| Proportion of industrial driveways | 0.00 |
| Number of clusters of closely located driveways (such that the maximum distance between two driveways in a cluster is 121 ft for the 55 mph speed of this road) | 4 |

Since there are no industrial driveways in this segment, the proportion of industrial driveways is then: 0 ÷ 5 = 0.00.

To determine the directional clusters, note that Figure 8.1 shows five different driveways within the segment. While driveways one and two constitute a cluster because they are both located on the same side of the road and at approximately 75 feet from each other, driveways four and five do not. This is because they are on opposite sides of the road. With the exception of driveways one and two, therefore, each driveway in this segment is at least 122 feet from each neighbor driveway on the same side of the road. As a result, the number of resulting clusters is four.

To estimate the predicted number of crashes associated with this segment, follow the rural safety procedure.

**Step 1: Compute the Effect of Exposure Factors using Error! Reference source not found.**.

*Baseline Exposure Values* = $(3.418 \times 10^{-3}) \times (AADT^{\,0.7825}) \times (Segment\ Length^{\,0.2864})$

*Baseline Exposure Values* = $(3.418 \times 10^{-3}) \times (4940^{\,0.7825}) \times (0.56^{\,0.2864}) = 2.249$

**Step 2: Select the adjustment factor for roadway design characteristics from** Table 2.7**.**

Since this segment has two travel lanes, the adjustment factor is simply 1.000 (from Table 2.7).

**Step 3: Compute the effect of driveways using Error! Reference source not found.**.

$$Roadside.effect = \exp[(1.2918 \times Prop.of.Ind.DW) + (0.1048 \times Total.\#.Clusters)] \; / \; (Total.\#.Driveways + 0.5)^{0.2864}$$

$$Roadside.effect = \exp[(1.2918 \times 0.00) + (0.1048 \times 4)] \; / \; (5.5)^{0.2864} = 0.9333$$

**Step 4: Obtain the predicted number of crashes for the segment by multiplying all of the above results (as established in Equation 4).**

*Predicted Number of Crashes = (Baseline Exposure Values)* x *(Effect from Roadway)* x *(Effect from Roadside / Driveways)*

*Predicted Number of Crashes = 2.249* x *1.000* x *0.9333 = 2.099 expected crashes in 5 years*

| |
|---|
| Example problem conclusion:<br><br>Based on exposure, roadway, and roadside characteristics we can predict that over a period of 5 years approximately 2 (rounded from 2.099) segment crashes will occur. |