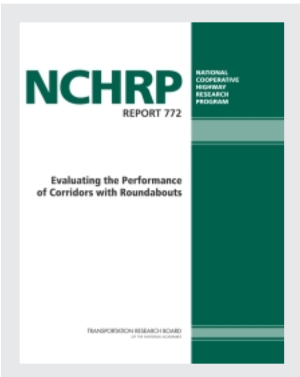

Evaluating the Performance of Corridors with Roundabouts

DETAILS

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 772

Evaluating the Performance of Corridors with Roundabouts

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NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

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FOREWORD

By Lori L. Sundstrom
Staff Officer
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NCHRP Report 772: Evaluating the Performance of Corridors with Roundabouts provides performance measurement and evaluation methods, including a predictive model for travel speed and an overarching comparison framework, for comparing the performance of a corridor with a functionally interdependent series of roundabouts to a corridor with signalized intersections in order to arrive at a design solution. For the purposes of this research, a "series of roundabouts" is defined as at least three roundabouts that function interdependently on an arterial. These evaluation methods will be of use to traffic engineers and transportation planners.

Roundabouts are increasingly recognized as an intersection control strategy that can fulfill multiple performance goals related to traffic operation and safety, as well as meet societal goals related to sustainability, livability, complete streets, context sensitive design, economic development, and others. Some transportation agencies have recently constructed or approved the use of a series of roundabouts on an arterial rather than the traditional solution of coordinated signalized intersections. While there are anecdotal reports suggesting that functionally interdependent roundabouts on a corridor are successful in meeting performance goals, little research has been conducted to determine objectively the efficacy of this alternative as compared to signalized intersections.

The performance of traffic signal systems on arterials is well researched and documented, and methods to predict their performance are well established. Performance measures for isolated roundabouts exist, and safety research has consistently shown that roundabouts have lower fatal and injury crash frequencies when compared to signalized intersections. In contrast, qualitative and quantitative information on the performance of a set of functionally interdependent roundabouts on arterials is lacking.

Under NCHRP Project 03-100, Kittelson & Associates, Inc. of Portland, Oregon, was asked to (1) conduct a literature review and gap analysis, (2) develop proposed performance measures and evaluation methods and conduct a field demonstration of the suggested performance measures and evaluation methods on existing arterials, and (3) suggest appropriate performance measures and evaluation methods. The research resulted in the development of a predictive model for travel speed on a roundabout corridor and an overall framework for comparing alternative corridor configurations that acknowledges a wide range of project catalysts that may influence decision making. Appendix A of *NCHRP Report 772* contains the "Corridor Comparison Document." Appendices B through O contain data on each of the nine roundabout corridors selected for detailed study; the appendices are available for downloading from the project webpage at http://apps.trb.org/cmsfeed/TRBNetProjectDisplay.asp?ProjectID=2950.

AUTHOR ACKNOWLEDGMENTS

The research reported herein was performed under NCHRP Project 03-100 by Kittelson & Associates, Inc.; Institute for Transportation Research and Education; Texas A&M Transportation Institute; and Write Rhetoric. Kittelson & Associates, Inc., was the Contractor for this study. The work undertaken by other project team members occurred under subcontracts with Kittelson & Associates, Inc.

Lee Rodegerdts, P.E., Principal Engineer, Kittelson & Associates, Inc., was the principal investigator. Brian Ray, P.E., Senior Principal Engineer, Kittelson & Associates, Inc., was project principal. The other authors of this report are Pete Jenior, P.E., P.T.O.E., Senior Engineer, Kittelson & Associates, Inc.; Dr. Zachary Bugg, Transportation Analyst, Kittelson & Associates, Inc.; Dr. Bastian Schroeder, Assistant Director, Institute for Transportation Research and Education, North Carolina State University; and Marcus Brewer, P.E., Associate Research Engineer, Texas Transportation Institute, Texas A & M University. During the project, Dr. Bugg graduated from North Carolina State University and joined Kittelson & Associates, Inc.

The project team acknowledges others who played significant roles in the project. Dr. James Bonneson, Kittelson & Associates, Inc., and Dr. Nagui Rouphail, Institute for Transportation Research and Education, provided senior review and guidance. Liang Ding and Steven Venglar of Texas A&M Transportation Institute conducted traffic operations analysis. Christopher Vaughn, Michael Corwin, Kyle Hovey, and David Craft of Institute for Transportation Research and Education and Martin Fuest and Christopher White of Texas A&M Transportation Institute assisted with field data collection, processing, and analysis. Ralph Bentley and Jesus Culler of Kittelson & Associates, Inc., developed graphics. Danica Rhodes of Write Rhetoric provided technical editing. Traffic counts were collected by Quality Counts, LLC.

The project team acknowledges the contributions of corridor owners who participated in interviews with the authors and provided as-built plans of study roundabouts. These individuals are: Mike Niederhauser of Maryland State Highway Administration; Siavash Pazargadi of the City of San Diego; Mike McBride of the City of Carmel; Marcos McGraw of the City of Gig Harbor; Brian Walsh, Dina Swires, and Dustin Terpening of Washington State Department of Transportation; Dan Hartman of the City of Golden; Justin Hildreth of the City of Avon; and Mark Kennedy, James Boni, and Howard McCulloch of New York State Department of Transportation.

Finally, the research team also acknowledges the guidance, support, and inspiration provided by the NCHRP Project 03-100 Panel Chair and Panel Members throughout the course of the project and research efforts. These panel members customized and adapted their participation in the research to provide an improved product.

CONTENTS

Note: Many of the photographs, figures, and tables in this report have been converted from color to grayscale for printing. The electronic version of the report (posted on the Web at www.trb.org) retains the color versions.

Evaluating the Performance of Corridors with Roundabouts

SUMMARY

EVALUATING THE PERFORMANCE OF CORRIDORS WITH ROUNDABOUTS

This study identified 58 roundabout corridors in the United States in 2011. There is diversity among these corridors in terms of length, roundabout spacing, number of lanes, surrounding land use, previous control (if not a new facility), and reasons for the selection of roundabouts. From this set of 58 corridors, nine were selected for detailed study:

- MD 216 in Scaggsville, Maryland
- La Jolla Boulevard in San Diego, California
- Old Meridian Street in Carmel, Indiana
- Spring Mill Road in Carmel, Indiana
- Borgen Boulevard in Gig Harbor, Washington
- SR 539 in Whatcom County, Washington
- Golden Road in Golden, Colorado
- Avon Road in Avon, Colorado
- SR 67 in Malta, New York

Safety evaluations of roundabouts from other research projects indicate conversion of individual signalized and two-way stop-controlled intersections to roundabouts results in a reduction in the frequency and severity of crashes. Although safety data were not collected for this project, corridor interviews revealed that safety improvements with roundabout applications were generally consistent with documented research. As a result, the research team found no evidence suggesting the safety performance of a roundabout in series differs from the safety performance of an isolated roundabout.

The following subsections highlight key findings for four topic areas:

- Corridor interviews;
- Travel-time collection, analysis, and modeling;
- Comparison to equivalent signalized corridors; and
- Development of a Corridor Comparison Document.

Corridor Interviews

Interviews with the owners of each of the nine corridors provided an insight into the creation and history of these roundabout corridors, agency and community goals for the corridors, and their effectivness at meeting those goals. The interviews revealed a variety of contexts in which roundabout corridors have come into being. Some of the corridors were designed and constructed in their entirety at one time; others started with one or two roundabouts and more were

Evaluating the Performance of Corridors with Roundabouts

added over time. The variety of motivations for considering roundabouts, the variety of levels of interaction with the public, and the design treatments ultimately constructed reinforce the notion that each corridor is a unique installation.

Specific themes and trends that emerged from the interviews include the following:

- Once several roundabouts are built on a corridor, new or upgraded intersections are more likely to be roundabouts than signalized intersections. Reasons for this include good performance of the roundabouts in place, increased public and agency awareness and acceptance of roundabouts, concerns about queue spillback from signals into roundabouts, access management, and consistency within the corridor.
- Traffic analysis of roundabout corridors prior to their construction was typically conducted by analyzing each roundabout in isolation. However, several corridors were analyzed with microsimulation. It is anticipated the predictive tools for operational performance developed in this project, combined with the new tools intended for the *Highway Capacity Manual* (HCM), will provide practitioners with a simpler alternative to microsimulation.
- The safety effect of each corridor was not studied in detail in this project. However, owners of two roundabout corridors constructed as retrofits stated crash frequency decreased on the corridor following the construction of roundabouts. The consistent safety findings reported elsewhere suggest this trend is likely to continue in a corridor context.
- An agency champion was often the key to a corridor being constructed with roundabouts.

Travel-Time Collection, Analysis, and Modeling

A data collection crew visited each of the nine corridors for two or three days. Floating car runs using probe vehicles equipped with GPS units recorded vehicle activity with one-second resolution and produced speed and travel-time trajectories. Field travel times were recorded during the a.m. peak, off-peak, and p.m. peak periods. Aggregating data within these times periods together for each corridor indicates the following:

- Study corridors operated at level of service (LOS) A through C based on travel speed as a percent of free-flow speed (the HCM 2010 performance measure for Urban Streets).
- Most routes had the same LOS for the three time periods. Some changed by one letter grade.
- Travel speed and LOS for through routes and left-turn routes were generally similar, with no pattern apparent of one performing better than the other.

Traffic operations models were developed from the field data for the purpose of enhancing the Urban Streets methodology in Chapter 17 of the HCM 2010. The methodology in the HCM 2010 was developed primarily for signalized corridors and does not explicitly account for certain attributes of roundabout corridors such as geometric delay of a through movement at a roundabout. The four models specifically developed for roundabout corridors as part of this research are:

- A free-flow speed model, which estimates the speed that most drivers would choose at mid-segment locations beyond the extent of the roundabout influence areas when other vehicles are not present.
- A roundabout influence area model, which estimates the length of roadway upstream and downstream of a roundabout over which speeds are reduced due to the presence of the roundabout. In other words, the roundabout influence area includes the deceleration zone prior to the roundabout and the acceleration zone following the roundabout.
- A geometric delay model, which estimates delay incurred at a roundabout node due to speed-limiting characteristics of the roundabout. The model was based on data collected by probe vehicles passing through roundabouts in the absence of other vehicles.
- An impeded delay model, which estimates delay incurred at a roundabout node due to the presence of other vehicles. Impeded delay includes control delay.

These models were incorporated into a methodology for estimating travel speed and LOS of roundabout corridors. Field data from two of the nine corridors were reserved for validation, and one of these contains a mix of roundabouts and signals. The validation exercise showed the developed corridor methodology correctly predicted the LOS for all four analysis routes on the first validation corridor, and was within one letter grade for the second validation corridor.

The TRB Committee on Highway Capacity and Quality of Service was notified of this project, and a conceptual overview of the roundabout corridor methodology was presented to the relevant subcommittees.

Comparison to Equivalent Signalized Corridors

Comparisons of field-measured vehicle travel times and simulated "equivalent" corridors with signal or two-way stop control confirmed a need for case-by-case evaluations. Specific findings include the following:

- Neither roundabout nor signalized/stop-controlled corridor configurations consistently result in reduced travel time or intersection delays for through routes. Approximately half of the through-movement routes resulted in lower travel time under a roundabout configuration, and approximately half resulted in lower travel time under non-roundabout configuration. Evidence suggests roundabout corridors have a good likelihood of improving travel-time performance, but site-specific operational conditions may favor signalization or stop control. This finding reinforces the need for a case-by-case evaluation.
- Corridors with irregular intersection spacing show a higher likelihood for having better travel times under a roundabout configuration rather than a signalized configuration.

Evaluating the Performance of Corridors with Roundabouts

- Corridors that can use two-way stop-controlled intersections (rather than signals) in the place of roundabouts generally produce better end-to-end travel times, even if intersection delays are lower under a roundabout configuration.
- For corridors where turning movements entering or departing the corridor are of similar or greater importance than end-to-end travel times, roundabout corridors appear more likely to improve those travel times. This may be due to higher side-street delays and the general practice of prioritizing signal timing for progression of through traffic over left turns and side-street movements. Among left-turn routes, the roundabout corridor usually had lower travel time than the non-roundabout corridor.
- Some findings for specific corridors:
  - Approach delay was lower with roundabouts for all intersections in both major street directions except for SR 539.
  - Through-route travel time (average of both directions) increased with roundabouts on La Jolla Boulevard, Old Meridian Street, and Golden Road; decreased with roundabouts on MD 216, Spring Mill Road, Avon Road, and SR 67; and remained virtually unchanged on SR 539.
  - Travel time for routes with a left turn off the major street (average of both directions) increased with roundabouts on La Jolla Boulevard; decreased on MD 216, Old Meridian Street, Spring Mill Road, SR 539, Avon Road, and SR 67; and remained virtually unchanged on Golden Road.
  - Travel time for routes with a left turn onto the major street (average of both directions) increased with roundabouts on La Jolla and decreased on the other corridors.
  - The La Jolla Boulevard corridor performs quite differently from the other corridors studied in this project. It is the most urban of the corridors studied, with considerable pedestrian, bicycle, and on-street parking activity. As a result, through vehicular traffic experiences more friction than was observed for other corridors. As confirmed in the corridor interviews, this outcome is consistent with the multimodal focus desired for this particular corridor.

In general, the findings of this project indicate a need for a corridor-specific evaluation to determine which form of intersection control is preferred on a given corridor. Furthermore, there are many performance measures other than traffic operations that are used to choose intersection control on a corridor.

Development of a Corridor Comparison Document

Finally, a Corridor Comparison Document (CCD) was developed to provide an overall framework for users to compare alternative corridor configurations and objectively inform project decisions based on the unique context of each project. It has the following overall features:

Evaluating the Performance of Corridors with Roundabouts

- The CCD provides a *broad approach* for helping to inform corridor solution concepts by enabling case-specific comparisons and evaluations.
- The CCD is *flexible* to adapt to the broad range of potential catalysts that might be the impetus for a particular project. These are illustrated through a series of example applications.
- The CCD *presents many performance measures* for various project and corridor contexts and refers the reader to other documents (e.g., HCM and *Highway Safety Manual* [HSM]) for specific assessment techniques. Other performance measures are generally evaluated through a best practices approach or are more qualitative in nature.
- The CCD is intended to be an *evaluation and decision-making framework* rather than a guideline or standard.

The following specific elements are included in the CCD:

- Information on different users of arterials, including passenger cars, buses, pedestrians, bicycles, trucks, and emergency vehicles;
- An overview of the project planning process written from the perspective of a practitioner evaluating alternatives for reconstructing an existing corridor or constructing a new roadway where the alignment has already been determined;
- Typical performance measures, assessment techniques for performance measures, and methods for selecting and prioritizing performance measures, grouped into broad categories of quality of service, safety, environmental, costs, community values, and others; and
- Four example applications illustrating use of the CCD, three of which result in roundabouts being selected and one of which results in signals being selected. The example applications are as follows:
    - A new suburban arterial being built in a greenfield to create access to undeveloped land and to provide increased connectivity.
    - A community enhancement project on an existing urban arterial.
    - An existing two-lane highway in a rural, context-sensitive environment that is beginning to experience suburban-style development as it transforms into a vacation and second-home community.
    - An existing suburban corridor being evaluated for safety and operational improvements due to changing context and a need for pavement rehabilitation.

Evaluating the Performance of Corridors with Roundabouts

CHAPTER 1. BACKGROUND

This report summarizes the findings of NCHRP Project 03-100, Evaluating the Performance of Corridors with Roundabouts. The intended audience for this report is researchers, practitioners, and policy makers who establish federal, state, and local guidelines for roundabouts.

This introductory chapter presents the problem statement and research objective, scope of study, research approach, and a summary of the literature review conducted for this project.

1.1. PROBLEM STATEMENT

Roundabouts are increasingly recognized as an intersection control strategy that can fulfill multiple performance goals related to traffic operation and safety and that meet societal goals related to sustainability, livability, complete streets, context-sensitive design, economic development, and others. Some transportation agencies have recently constructed or approved the use of a series of roundabouts on an arterial rather than the traditional solution of coordinated signalized intersections. While anecdotal reports suggest that functionally interdependent roundabouts on a corridor are successful in meeting performance goals, little research has been conducted to objectively determine the efficacy of this alternative as compared to a series of coordinated signalized intersections.

The performance of traffic signal systems on arterials is well researched and documented, and methods to predict their performance are well established. Performance measures for isolated roundabouts exist, and safety research has consistently shown that signalized intersections have higher injury crash rates when compared to roundabouts. In contrast, qualitative and quantitative information on the performance of a set of functionally interdependent roundabouts on arterials is lacking.

1.2. RESEARCH OBJECTIVE

The objective of this research is to provide traffic engineers, transportation planners, and other practitioners with performance measurement and evaluation methods to comprehensively evaluate the performance of functionally interdependent roundabouts on arterials, thus enabling a comparison with signalized intersections, in order to arrive at a design solution. For the purposes of this research, a "series of roundabouts" shall include at least three roundabouts that function interdependently on an arterial.

The research plan developed to achieve this objective focused on the delivery of two key products:

1. Performance measurement tools and techniques based on quantitative, empirical data that can assist in the evaluation of a roundabout corridor.

2. A set of guidelines for corridor comparisons that incorporates both quantitative and qualitative components.

The results of this study can be grouped into two major categories:

- An assessment of the performance of existing corridors that employ a series of roundabouts. This assessment comprises a general evaluation of their development and success, obtained through field observations and interviews with corridor operators, and a detailed data-collection effort for operational data to aid in performance prediction.
- Tools to enable alternatives analysis for corridors employing a variety of intersection control treatments, whether they be roundabouts, traffic signals, or stop control. These tools include an overall framework for comparison (called a Corridor Comparison Document) and predictive tools for operational performance intended to supplement existing predictive tools in the *Highway Capacity Manual*.

As a result of these two categories of research products, this report is a hybrid of (1) content that is intended for inclusion in other major resource documents (e.g., *Highway Capacity Manual* and *Roundabouts: An Informational Guide)* and (2) stand-alone content in the form of a Corridor Comparison Document that can be used directly. To achieve these objectives, the research team undertook the following broad tasks:

- Conducted focused outreach efforts with operators of roundabout corridors to understand the actual characteristics and lessons learned.
- Identified the quantitative elements related to the operational models, data collection, and recommended analysis methodology.
- Identified the qualitative elements that could augment the quantitative elements and predictive operations models to support corridor treatment evaluations, comparisons, and recommendations. Examples include access management considerations, safety performance, access to non-motorized transportation users, constructability, and how well the arterial treatment fits within the broader city design and cultural context.
- Collected traffic operations field data at nine roundabout corridors using proven and emerging tools and techniques to develop a field data–driven methodology for evaluating roundabout corridors, and compared them to signalized intersection treatments.
- Developed a predictive procedure for travel time on a roundabout corridor. The procedure incorporates models developed from the field data collected as part of this project, and is presented in a manner consistent with the auto procedure of the *Highway Capacity Manual* (HCM) 2010 Urban Streets Chapter. The procedure is recommended for inclusion into the next edition of the HCM.
- Created a practitioner-focused guidance framework (called a Corridor Comparison Document) that provides a holistic approach to considering, evaluating, and supporting corridor treatment decisions. The framework

Evaluating the Performance of Corridors with Roundabouts

includes groups of performance measures and prioritized "tiers" of evaluation considerations consistent with multiple corridor contexts. Four example applications illustrate the use of the guidance framework.

1.3. BACKGROUND AND LITERATURE REVIEW

Comparatively speaking, the transportation profession's understanding of signalized intersection corridor operation is more developed than its understanding of roundabout corridors. For signalized intersections, considerable research and experience has gone into evaluating and timing intersections in both isolated and coordinated models. Analytical and simulation modeling has been common for considering the interaction between adjacent signalized intersections, and significant documentation exists for optimizing flow along signalized corridors. Practitioners have a solid base of experience and well-developed "gut feels" for how the familiar signalized corridor should operate. In contrast, roundabout evaluations have largely focused on isolated intersections, using only simulation tools with any regularity in practice to evaluate roundabouts in corridors. Many practitioners have never seen a roundabout corridor in person, much less have a "gut feel" for how it would operate. The intent of this project is to close that gap for practitioners by improving the basis for good decisions.

A variety of analytical methods and tools are available to quantify projected roundabout intersection operations, yet prior to this project very little documentation was available that quantifies the operational attributes of roundabout corridors. Data collection for this project focused on elements affecting traffic operations, so that a predictive model for operations could be developed. Other project activities, such as corridor owner interviews, were more comprehensive and included elements such as planning, pedestrian and bicycle user experience, public involvement, construction, and maintenance.

1.3.1. SAFETY AND ACCESS MANAGEMENT CONSIDERATIONS

Roundabouts have well-documented safety benefits compared to other types of traffic control, and these safety benefits are the predominant attractiveness compared to signalized intersections (Gross et al. 2012, NCHRP Report 572, NCHRP Report 672, Persaud et al. 2001). Safety relationships of isolated roundabouts are likely to transfer to roundabout corridors; there are no specific characteristics of roundabouts in series diminishing the safety performance of the roundabout junction itself.

On a corridor level, roundabouts create more access management opportunities compared to signalized intersections. One key differentiating consideration between corridor types may be safety at midblock access points. Opportunities to use roundabout U-turning qualities could potentially eliminate left turns to or from driveways along the corridor. Reducing turns at driveways would reduce vehicle conflicts at these locations and positively influence overall corridor safety performance. In addition to reduced conflicts, depending on the spacing of the roundabouts, segment operating speeds could be reduced compared to signalized corridors and, therefore, could reduce crash severity. Roundabout

Evaluating the Performance of Corridors with Roundabouts

corridors could possibly reduce crash frequency because slower operating speeds decrease stopping sight distance requirements and, therefore, increase the opportunity to avoid crashes. Access management principles are well established in the literature (e.g., TRB *Access Management Manual*).

1.3.2. OPERATIONAL CONSIDERATIONS

The transportation profession has an extensive body of knowledge for traffic operations at isolated roundabouts and urban corridors with signalized intersections. Little research has been documented on traffic operations on roundabout corridors, particularly in the United States.

Roundabout corridors have unique operational characteristics compared to their signalized intersection counterparts. Fundamentally, the notion of moving platoons of vehicles to maximize the performance efficiency of signalized intersections is not applicable to roundabouts, where gap-acceptance principles allow more dispersed flows to mingle within the intersections. Travel time is a natural performance measure for roundabout and signalized corridors. Roundabouts have increased geometric delay compared to signalized intersections by virtue of their shape; therefore, defining travel-time performance measures is of paramount interest.

1.3.2.1. *Roundabouts in Isolation*

Prior to development of models based on observed performance of roundabouts in the United States, operational analysis of individual roundabouts in the United States has been mostly conducted using methodologies and software developed internationally. There have been some contributions from countries such as France and Germany, but methods from the United Kingdom and Australia have dominated US practice. There are conceptual differences between UK and Australian schools of thought discussed below.

The UK's Transport Research Laboratory (TRL) developed their capacity analysis techniques using empirical regression methods (Kimber 1980). Within this methodology, roundabout approach capacity is highly dependent upon geometric features of entries, such as the width, radius, and angle. The methodology was developed based upon extensive field observations of near-capacity roundabouts in the UK. The RODEL and Arcady software packages implement the results of the TRL research findings. The most recent version of Arcady at the time of this publication includes a "linked roundabout" feature for analyzing adjacent roundabouts. Arcady adjusts flow entering the downstream roundabout based upon operations at the upstream roundabout.

SIDRA, an Australian software package developed by Akçelik and Associates Pty Ltd., analyzes roundabouts as well as stop-controlled and signalized intersections. Within SIDRA's methodology, roundabout capacity is primarily a function of gap acceptance (i.e., entering vehicles accepting gaps in the flow of circulating traffic). At one time, practitioners in the United States generally applied a capacity reduction factor ("environmental factor" in SIDRA's terminology) of 1.2 to account for observed reductions in capacity compared to

Evaluating the Performance of Corridors with Roundabouts

Australian roundabouts. However, the most recent version of SIDRA at the time of this publication incorporates the methodology of the HCM 2010 directly.

The HCM 2010 (TRB 2010) contains a roundabout analysis procedure based upon data collected in the US as part of NCHRP Report 572. The methodology is limited to one- or two-lane roundabouts with no more than four legs. This procedure took a hybrid approach, where approach capacity was empirically derived from regression-based analysis, while also incorporating behavioral gap-acceptance parameters that can be user-calibrated.

Regardless of the way capacity is estimated, all analytical methods principally compare approach capacity to approach volume. They use equations to then predict performance measures such as approach delay and vehicle queuing.

Mauro (2010) compiled a selection of analysis techniques for roundabout capacity and performance. Those techniques contain calculations for additional service measures, such as queue length and waiting time, which are varied based on the level of saturation at the intersection. He also discusses time spent in the intersection, which contributes to calculating a level of service for the roundabout using the methodology in the 2000 edition of the HCM. He ultimately uses his method for determining capacity to estimate a roundabout's reliability (i.e., the probability that the intersection does not fail and that demand does not exceed the capacity of any single entry). While the HCM methodology has changed for the 2010 edition, queue length and waiting time remain valid measures for consideration in any roundabout analysis procedure.

Few of the international or domestic procedures explicitly account for the impacts of adjacent intersections (including roundabout intersections), nor do they provide a means of analyzing multiple roundabouts at once to gauge cumulative performance. Users analyzing a roundabout corridor as a series of isolated roundabouts may not account for platooning and queue spillback effects. Additionally, there is no means of assessing corridor-wide metrics such as travel time. Some practitioners in the United States have used microsimulation software (such as VISSIM and Paramics) to analyze individual and multiple roundabouts. This does represent a means of analyzing roundabout corridors, but like all applications of microsimulation it requires more time and specialized skills on the part of the analyst compared to other analysis tools.

1.3.2.2. *Roundabout Corridors*

A study of a roundabout corridor in Golden, Colorado (Ariniello, 2004), reviewed crash rates, operating speeds, travel times, and sales tax revenue along the corridor. A portion of South Golden Road between Ulysses Street and Johnson Street was considered for study, where four roundabouts were installed in a corridor of approximately a half mile in length. The five-lane corridor served several residential areas and many businesses, including several fast-food restaurants, a large grocery store, and a small shopping center. The composition of the traffic mix was not specified, but a high number of driveways were identified in the report, suggesting that turning traffic was substantial and included large delivery and service vehicles in addition to passenger vehicles

Evaluating the Performance of Corridors with Roundabouts

carrying customers and residents. The site description specifically mentioned the existence of horse trailers entering and exiting a veterinary office in the corridor.

Ariniello concluded that installing the roundabouts resulted in slower speeds between major intersections in the corridor, but there were also lower travel times compared to when the corridor was signalized (reduced from 78 to 68 seconds through the corridor). The analysis also revealed less delay at business access points. Before installing the roundabouts, the average measured delay was 28 seconds with a maximum of 118 seconds. After the installation, the average delay was reduced to 13 seconds with a maximum of 40 seconds. Between 1996 and 2004, traffic volumes increased from 11,500 to 15,500 vehicles per day, while the number of annual crashes dropped from 123 to 19. Calculated crash rates declined by 88 percent, from 5.9 to 0.4 crashes per million vehicle miles; injury crashes were reduced from 31 in the three years prior to installation to one in the 4.5 years after—a 93 percent decline in injury crashes. Sales tax revenue along the corridor increased 60 percent and 75,000 square feet of retail/office space was built after installation.

Isebrands et al. (2008) reviewed corridors in Brown County, Wisconsin, and Edina, Minnesota. They found total crashes at one of the Wisconsin roundabouts were reduced by one per year and injury crashes were nearly eliminated. Another roundabout in the Wisconsin corridor did not have enough data after installation to make a definitive conclusion on crashes. Access management treatments and a series of three roundabouts were used along the Minnesota corridor to address traffic operation and safety performance. Although the roundabouts were open for only a relatively short time when the study was conducted, the city indicated to Isebrands et al. that vehicle operations improved from levels of service (LOS) between B and F prior to opening to LOS ranging from A to D after opening. They also found no reduction or change in access to local businesses.

1.3.2.3. *Signalized Corridors*

Numerous studies on the operations of signalized corridors have been conducted and documented. Signalized corridor analysis is a mature area of study, and the fundamentals of signalized corridor operation can be related to those of roundabout corridors for metrics such as travel time and delay. Signalized corridor studies provide relevant information to better understand and compare the performance of roundabout corridors for similar measures of effectiveness.

The HCM 2010 introduces a method for evaluating the quality of service on an urban street using measures for four travel modes—automobiles, transit, pedestrians, and bicycles—based on user perceptions of quality of service. Exhibit 8-3 of the HCM 2010 lists components of traveler-perception models used to generate service measures contributing to quality of service. The portion of that exhibit pertaining to urban street segments and intersections is depicted as Exhibit 1-1. The automobile traveler-perception model for urban street segments is not used to determine LOS, but it is provided in the HCM 2010 as a performance measure to facilitate multimodal analyses. Other automobile-related components (e.g., through delay), as well as components from other

Evaluating the Performance of Corridors with Roundabouts

modes (e.g., vehicle volume and speed), do contribute to the calculation of LOS for intersections and segments.

Exhibit 1-1: Components of Traveler-Perception Models Used to Generate Service Measures (TRB 2010)

System Element	Mode	Model Components
Urban Street Segment	Automobile	Stops per mile, left-turn lane presence
	Pedestrian	Pedestrian density, sidewalk width, perceived separation between pedestrians and motor vehicles, motor vehicle volume and speed
	Bicycle	Perceived separation between bicycles and motor vehicles, pavement quality, automobile and heavy vehicle volume and speed
	Transit	Service frequency, perceived speed, pedestrian LOS
Signalized Intersection	Pedestrian	Street crossing delay, pedestrian exposure to turning vehicle conflicts, crossing distance
	Bicycle	Perceived separation between bicycles and motor vehicles, crossing distance

For the automobile mode, Dowling et al. (2008) found that stops per mile was the key quality of service measure for signalized arterials based on extensive driver surveys. Stops are a significant consideration to drivers and are key inputs in evaluating energy consumption and exhaust emissions. The research also emphasized the importance of incorporating all road users. While the research did not explicitly incorporate roundabouts, the parameters for describing pedestrian and bicycle quality of service (e.g., sidewalk width, buffer separation to vehicular traffic, presence of on-street parking, or expected delay at crossing points) may rate a roundabout corridor favorably over an equivalent-capacity signalized arterial.

Bonneson et al. also contributed to methodologies and service measures considered by the HCM 2010. In the first of two reports from NCHRP Project 03-79 (Bonneson et al. 2008a), researchers summarized findings of then-current practices in real-time performance measurement of urban streets. They specifically described three measurement concepts: area-wide measurement, segment-based measurement, and signal-based measurement.

Area-wide measurement techniques typically use probe vehicles and some type of wireless technology. This technique is used to sample a large number of vehicles on the urban street system at a few dispersed locations; the sample is then used to estimate aggregate performance measures that describe facility performance for the previous hour or more. Segment-based measurement techniques are used to measure performance on a specified street segment by monitoring traffic flow along the segment.

Segment-based techniques typically use one or more vehicle detectors, such as inductance loops or cameras, to monitor traffic flow on the segment. This technique estimates the performance of the monitored segment with a reasonable accuracy and with a frequency suitable for responsive signal-control applications.

Signal-based measurement techniques measure performance on a specified street segment by monitoring traffic flow along the segment and the signal timing status of the signalized intersection that bounds the segment. Techniques following this approach typically use detectors to monitor traffic flow and receive information about the status of the phase serving the through-traffic movement.

According to the researchers' findings, these techniques estimate the segment performance with a high degree of accuracy and with a frequency suitable for responsive or adaptive signal-control applications. They noted travel time and travel speed were not directly measured by any of the techniques. Rather, they were estimated by combining the delay and running time measurements. This approach to travel-speed estimation was intended to overcome challenges they identified in previous research that were associated with the direct measurement (or prediction) of travel speed on urban street segments.

Using the findings and recommendations from efforts documented in the first report, the NCHRP Project 03-79 researchers proceeded to evaluate a selection of alternative performance prediction procedures (Bonneson et al. 2008b). The focus of their evaluation was on procedures that predicted measures (i.e., running time, delay, and stop rate) to describe the operational performance of automobile traffic flow on urban streets. One procedure was used to estimate running time and the other was used to estimate signal-control delay. They found several factors affecting those two service measures, as shown in Exhibit 1-2. Ultimately, the researchers developed several procedures that were included in the HCM 2010 urban street performance evaluation methodology, with the intention of improving the accuracy of the estimated running time and control delay. Many of these procedures are also applicable to roundabout corridors, although they have not yet been applied to or calibrated for roundabout corridor evaluation.

Those procedures were:

- delay due to turning vehicles,
- running time (including free-flow speed),
- arrival flow profile,
- actuated phase duration,
- stop rate at a signalized intersection, and
- capacity constraints.

The stop-rate prediction procedure was developed to extend the range of performance measures predicted by the HCM 2010 methodology. The accuracy of the proposed procedures was evaluated by comparing the predicted performance measures with those obtained from a traffic simulation model. The findings from their analysis indicated the predicted delay from the proposed procedures was within one or two seconds of that obtained from the simulation model. A similarly good fit was found when comparing the predicted stop rate with that obtained from the simulation model. The researchers' analysis also indicated the proposed procedures yielded a reasonably good estimate of the

Evaluating the Performance of Corridors with Roundabouts

simulated travel speed. Although the urban street procedure was developed for signalized corridors, many of the factors, including running time, delay due to turning vehicles, capacity constraints, and LOS, are relevant to roundabout corridors and could be measured and applied when analyzing a roundabout corridor.

Exhibit 1-2: Factors Affecting Service Measures in Estimating Travel Time on Urban Streets (Bonneson et al. 2008b)

Service Measure	Factor
Running Time	• Influence of segment length on free-flow speed • Delay due to vehicles turning right from a through lane • Delay due to vehicles turning left from a through lane • Factors influencing free-flow speed (e.g., access point density, lane width, lateral clearance) • Delay due to proximity of other vehicles (i.e., effect of traffic density on speed) • Delay due to on-street parking maneuvers
Signal-Control Delay	• Basic signal coordination (i.e., platoon dispersion) • Green interval timing (i.e., average phase duration) • Semi-actuated signal coordination (i.e., signal offset relationship) • Upstream signal metering and queue spillback

1.3.2.4. *Unique Features of Auto Travel on Roundabout Corridors*

The concept of geometric delay is an important one in comparing the total delay of roundabouts to that of signalized intersections since it is a significant difference between the two corridor types. All vehicles are expected to slow to an appropriate speed for negotiating a roundabout; therefore, they experience a delay based on the geometry of the intersection. According to Akçelik (2011), geometric delay is determined as a function of approach and exit cruise speeds as well as negotiation speeds, which depend on the geometric characteristics of the roundabout. Akçelik added that steps could be taken to approximate the value of geometric delay and add it to the control delay computed by the HCM procedure.

1.3.2.5. *Mixed Signal/Roundabout Corridors*

Research has been conducted on corridors containing signals and roundabouts. This section summarizes the research most relevant to this project.

Bared and Edara (2005) simulated the traffic impacts of roundabouts. They investigated two scenarios:

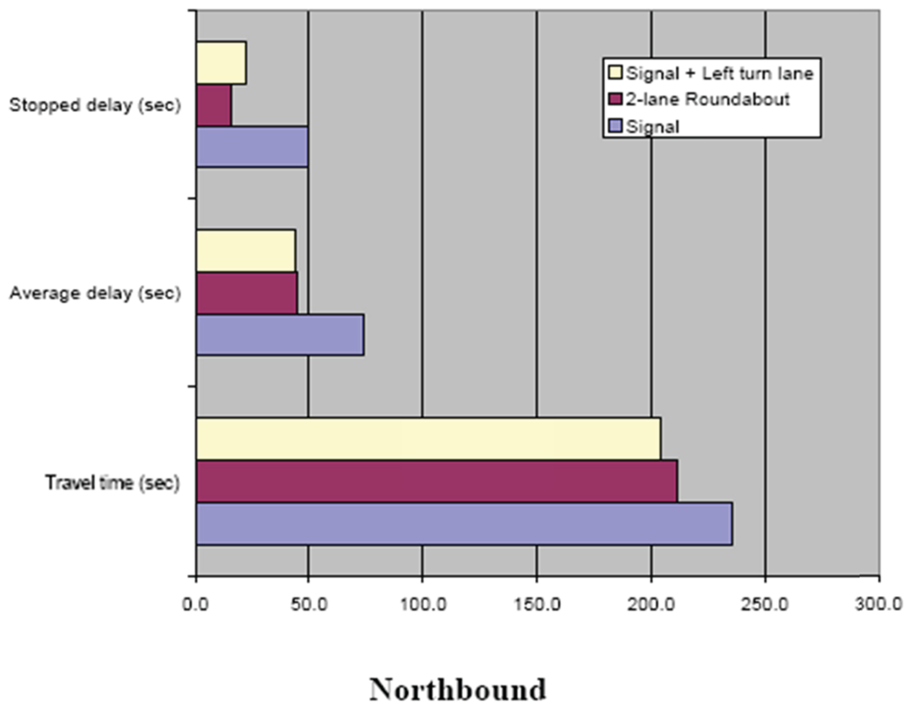
1. Urban single-lane and dual-lane roundabouts were modeled in VISSIM and compared with the results of RODEL and SIDRA. Their comparison with data collected from various sites in the United States showed VISSIM results were closer to field data than the RODEL and SIDRA results.

Evaluating the Performance of Corridors with Roundabouts

2. The impact of signalized intersection proximity to roundabouts was studied using a model developed by the researchers. More specifically, they studied the impact of a coordinated signalized arterial when a roundabout is inserted within an arterial corridor. Results of average delay measures were comparable to the signalization alternative when the roundabout was operating below capacity. However, at heavy volumes, when the roundabout was operating at capacity, the performance of signalization in the model was slightly better. The researchers did not report on comparing the model's results with field data or how well they were correlated.

Isebrands et al. (2008) examined two signalized corridor location case studies that contained roundabouts: one in Ames, Iowa, and one in Woodbury, Minnesota. For the Ames corridor, researchers coded the details into VISSIM, using existing vehicle volumes and intersection timing plans. They evaluated three alternatives: (1) optimized signal timing with the existing signalized corridor, (2) a two-lane roundabout at one selected intersection, and (3) optimized signal timing with left-turn lanes at the same intersection. Once the system was calibrated to replicate existing conditions, they attempted to optimize signal timings and coordinate the system for each alternative, but they were unable to achieve an optimal coordination plan due to geometry and other constraints. However, the best possible progression was sought with offsets and signal timings. The resulting timing plan with the existing geometry alternative had much higher travel time, stopped delay, and average delay than the other two alternatives, as shown in Exhibit 1-3. The signal with left-turn lanes had slightly more stopped delay for both the northbound and southbound directions of travel than the roundabout alternative. However, the two alternatives had similar amounts of average delay for both directions. The signal with the left-turn alternative had slightly less average delay for the northbound direction of travel, while the roundabout had slightly less delay for the southbound direction of travel.

The corridor in Woodbury had three major intersections: signals for the two northernmost and a roundabout at the southern intersection. Two alternatives were evaluated for the southern intersection: a four-way stop and a two-lane roundabout. Both alternatives were modeled in VISSIM, and results are shown in Exhibit 1-4 for average delay, stopped delay, and travel time for passenger vehicles. Vehicles turning onto and off of the system mid-corridor were not included in the analysis. Data in Exhibit 1-4 indicate little difference in total travel time for both the northbound and southbound corridors between the two alternatives. Average delay was 10 and 17 seconds longer with the four-way stop alternative for both northbound and southbound directions of travel, respectively, than for the roundabout alternative. Stopped delay was slightly longer with the four-way stop alternative for both northbound and southbound directions than for the roundabout alternative.



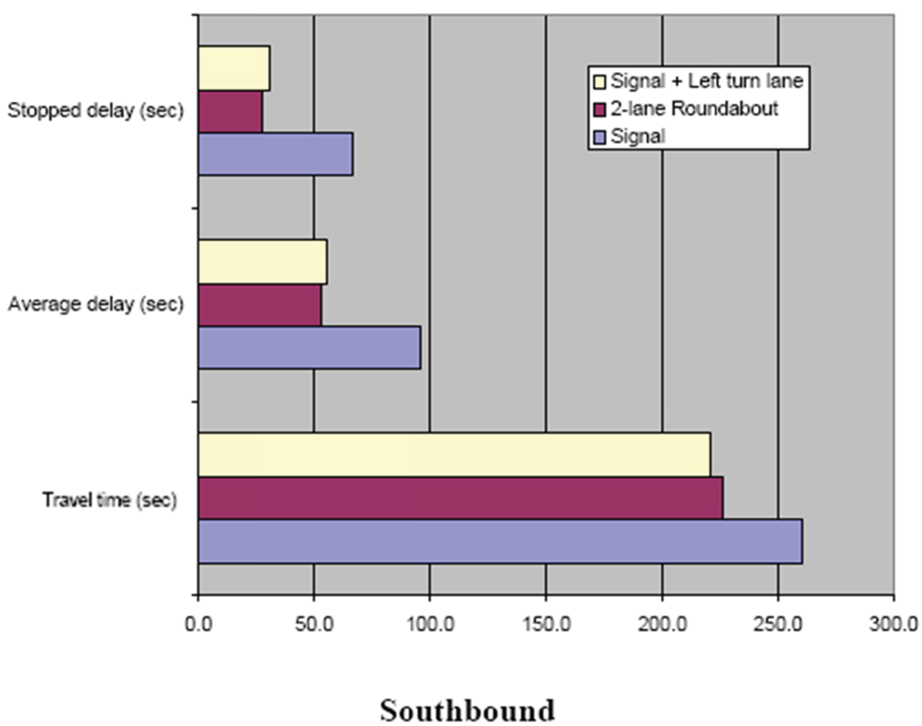
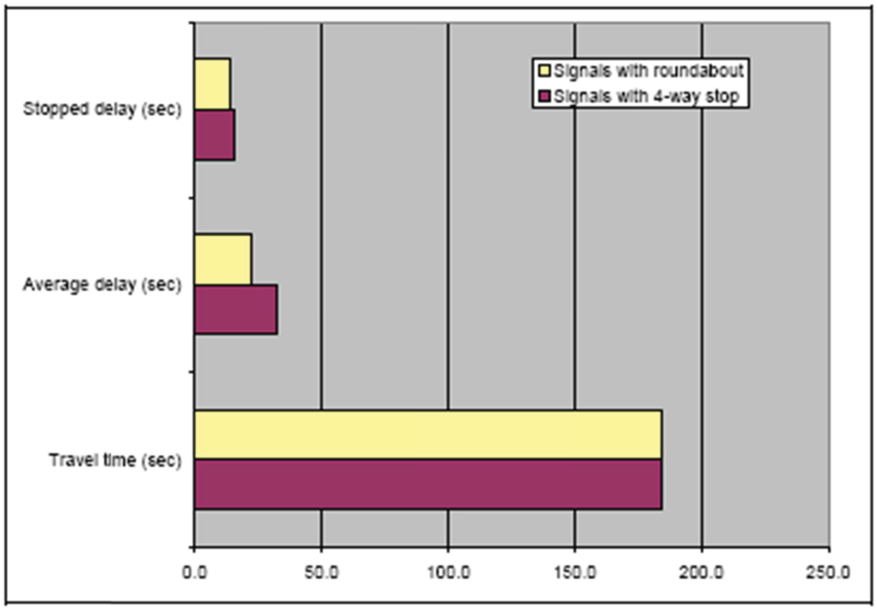


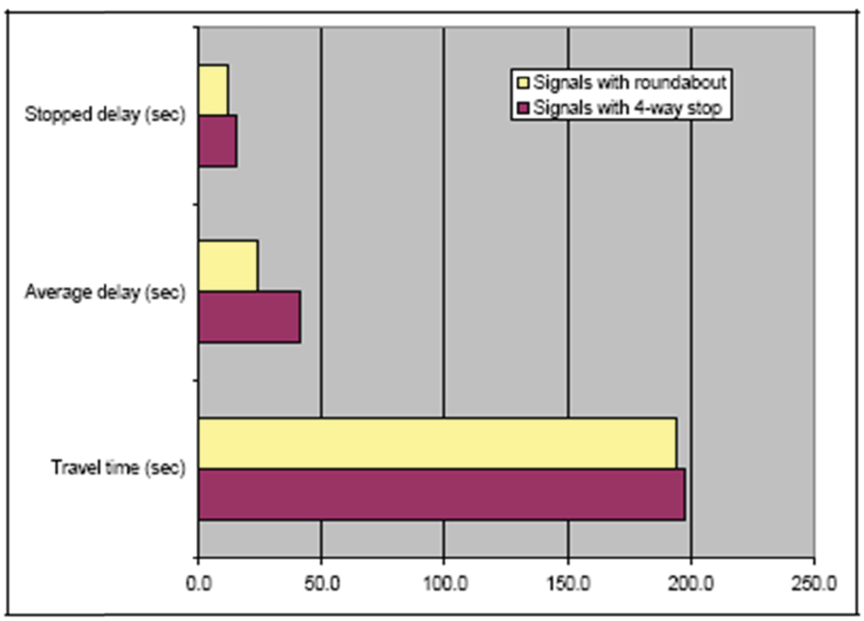
Exhibit 1-3: Comparison of Alternatives for the Ames, Iowa Corridor (Isebrands et al. 2008)

Evaluating the Performance of Corridors with Roundabouts

Exhibit 1-4: Comparison of Alternatives for the Woodbury, Minnesota Corridor (Isebrands et al. 2008)



Northbound



Southbound

A subsequent review of the data from Ames and Woodbury (Hallmark et al. 2010) using VISSIM led researchers to conclude that, based on results from the two case studies, roundabouts had minimal impact on corridor travel time. At the Ames site, signals with left-turn lanes and roundabout alternatives had similar results, considering both directions of travel together, suggesting a roundabout in this scenario did not provide a significant advantage in terms of traffic operations through the corridor as compared to the alternative where the

Evaluating the Performance of Corridors with Roundabouts

left-turn lanes were added. In Woodbury, average stopped delay was 10 and 17 seconds longer for the four-way stop alternative for both directions, respectively, compared to the roundabout alternative.

1.3.2.6. *International Experience in Roundabout Corridor Evaluations*

There is little documented international experience of roundabout corridor evaluations similar to those considered for this project. An informal survey at a roundabout workshop held during the International Symposium on Highway Capacity and Quality of Service in Stockholm, Sweden (June 2011) revealed few cited examples of roundabout corridors in other countries, and limited experience or guidance for analysis practices. The workshop featured attendees from Germany, Sweden, Finland, Denmark, Australia, Poland, Portugal, Spain, Italy, and the United Kingdom, among others. Of all the attendees, the only work toward a roundabout corridor evaluation method was being conducted in Australia.

1.3.3. OTHER ROUNDABOUT CORRIDOR CONSIDERATIONS

Emissions, non-motorized transportation modes, constructability, and corridor context are additional considerations potentially providing differentiating characteristics between corridor types. The literature review explored past research in these areas and found they are generally less documented in comparison to operations and safety.

1.3.3.1. *Emissions*

Studies examining effects on emissions generally determined isolated roundabouts performed at least as well as traffic signals for key pollutants.

In addition to providing traffic operations analysis, the SIDRA software package provides emissions and fuel consumption data for roundabouts and other types of intersections. SIDRA uses a “four-mode elemental model” to calculate emissions, considering time vehicles are cruising, decelerating, idling, and accelerating (Akçelik & Associates Pty Ltd 2011). Myers et al. (2005) used SIDRA to compare the performance of roundabout and existing control devices at 13 study intersections in Northern Virginia. The study showed a slight decrease in fuel consumption and emissions of four gases (carbon monoxide, hydrocarbons, nitric oxide, and carbon dioxide) during peak periods at the intersections where single-lane roundabouts would be appropriate. At intersections where multi-lane roundabouts would be appropriate, SIDRA predicted fuel savings of 14%, 9%, and 15% during the a.m., midday, and p.m. peak hours, respectively. The savings were in comparison to the existing control devices at the intersection.

The majority of roundabout air quality research in the United States is relatively simplistic and similar to the Northern Virginia study in the sense that it merely reports the outputs of traffic analysis software such as SIDRA. As such, though the actual software may change from study to study, and the actual study sites may vary, the Northern Virginia study is representative of the types of studies that have been conducted and documented in the United States.

Evaluating the Performance of Corridors with Roundabouts

Coelho et al. (2009) developed a traffic and emission decision support (TEDS) tool for urban highway corridors. They analyzed a highway corridor in Portugal containing a roundabout, a traffic signal, and a speed-control traffic signal; the first two treatments are typical of those found in use in the United States, while a speed-control traffic signal is not. A speed-control signal is installed with speed detection devices as part of a system used to reduce speeds. In the system, individual vehicle speeds are detected upstream of the signal and if the detected speed remains below a programmed speed threshold, the signal rests in green. When the signal detects a vehicle traveling over the speed threshold, it displays a fixed clearance time, followed by a red time (of fixed or variable length) and a minimum green time, to the approaching driver.

The Coelho et al. analysis suggested the roundabout intersection produced emissions similar to those of the traffic signal but more than those of the speed-control signal in three of four emission types, as shown in Exhibit 1-5. The primary conclusion of the report was that the greatest percentage of vehicle emissions in the highway segment occurred at the traffic interruptions (signals and roundabout), due to the final acceleration back to cruise speed and to stop-and-go cycles where there were queues. The traffic interruptions were only 24 percent of the total segment distance, but together produced more than 50 percent of total emissions of the segment for all pollutants and, in the worst situation, 75 percent of overall carbon monoxide (CO) emissions.

Exhibit 1-5: Percentage of Emissions Produced by Zone Type (Coelho et al. 2009)

Zone	CO	NO	HC	CO_2	Percentage of Total Distance
Basic Highway Segments	25	41	38	49	76
Traffic Signal	32	21	25	19	8
Speed-Control Traffic Signal	26	17	15	11	8
Roundabout	17	21	22	21	8

Note:
CO – Carbon monoxide
NO – Nitric oxide
HC – Hydrocarbon
CO_2 – Carbon dioxide

1.3.3.2. *Non-motorized Transportation Modes*

NCHRP Report 616 *Multimodal Levels of Service Analysis for Urban Streets* documents user perspectives of a facility's quality of service for pedestrian and bicycle modes, in addition to auto and transit modes (Dowling et al. 2008). This research was incorporated into the HCM 2010. Exhibit 1-1, presented earlier in this chapter, lists components influencing pedestrian and bicycle LOS on urban arterials with signals. It is likely the same components would influence pedestrian quality of service and, thus, pedestrian LOS on roundabout corridors.

In some cases, design and operating differences between signalized corridors and roundabout corridors may generally increase or decrease pedestrian or bicycle LOS or components of LOS. For example, increasing travel speed is

Evaluating the Performance of Corridors with Roundabouts

associated with decreasing pedestrian LOS. On a roundabout corridor, speeds in the vicinity of roundabouts are limited by the geometry of the roundabouts. If roundabouts are close enough together to limit speed on an entire corridor, pedestrian LOS may increase compared to an equivalent signalized corridor. At an unsignalized roundabout, pedestrian delay is generally reduced compared to a signalized intersection because a crossing can legally be made whenever a gap is present instead of waiting for a Walk indication. However, the lack of a signalized crossing may be perceived as detrimental to the quality of service for pedestrians.

1.3.3.3. *Constructability*

In new arterial corridors there is relatively little difference in constructing roundabout treatments compared to signalized intersections. However, in retrofit conditions a roundabout's footprint and approach geometry treatments increase right-of-way needs, construction staging, and traffic maintenance during construction requirements.

1.3.3.4. *Context-Sensitive Design*

Finally, roundabouts offer unique supporting design qualities to particular corridor contexts. Fundamentally, roundabouts offer distinct physical and visual separations between roadways approaching and continuing through an urban environment. A roundabout arterial corridor could strongly support redevelopment objectives for a community. Roundabouts offer gateway and speed reduction changes and could be especially effective on highways that become a "main street." Landscaped medians and landscaped central islands may be particularly conducive to desired land-use contexts. As an example, the La Jolla Boulevard corridor retrofit in the Bird Rock neighborhood of San Diego, California, supported a road diet and dramatically changed the corridor context from the former traditional arterial treatment.

1.3.4. LITERATURE REVIEW SUMMARY

The literature review shows progress in better understanding the operational characteristics of roundabouts in the United States, particularly isolated roundabouts. Studies of signalized corridors and corridors with mixed control also provide insight on potential service measures and analysis methods.

Key findings from the literature review are as follows:

- The methodology for determining the safety performance of roundabouts compared to other forms of control is well established by the *Highway Safety Manual* (HSM). In addition, the combination of the HSM and the TRB *Access Management Manual* provides considerable insight on the impact of various access management techniques that can be used in a corridor of roundabouts or signals.
- The operational methods for evaluating corridors of roundabouts are lacking key methods.
- Evaluating other aspects, such as emissions, non-motorized modes, constructability, and context-sensitive design, contributes to a holistic

Evaluating the Performance of Corridors with Roundabouts

corridor evaluation. These aspects may not be able to be readily quantified in this research project, certainly not to any level of statistical significance, but anecdotal evidence from a variety of corridors can still prove useful to practitioners. As a result, the data-collection plan placed emphasis on capturing a variety of corridor contexts to gain insight on the considerations that led to the development of each corridor.

Evaluating the Performance of Corridors with Roundabouts

CHAPTER 2. RESEARCH APPROACH

The major components of the research included the following steps:

- Establish the need for the project in terms of its place within the body of existing literature and practice. The results of this are documented in Chapter 1.
- Prepare a general framework to enable the comparisons of corridors using a variety of intersection control forms, including roundabouts, traffic signals, and stop control.
- Identify roundabout corridors in the United States and key characteristics for which a breadth of useful data can be obtained.
- Prepare and execute a data collection plan to identify a set of existing roundabout corridors, conduct interviews of corridor operators, and collect operational performance data.
- Analyze the collected field data to develop predictive models for operational performance suitable for inclusion in the *Highway Capacity Manual* and to assess performance relative to hypothetical "equivalent non-roundabout corridors."

This chapter discusses the corridor comparison framework and the data collection plan, as well as background and summary information on the corridors selected for data collection.

The data analysis plan is divided into three distinct components: (1) empirical data analysis, (2) development of predictive model for roundabout corridors, and (3) development and comparison of traditional (signalized or stop controlled) alternatives. Data analysis is discussed in Chapter 3.

2.1. CORRIDOR COMPARISON DOCUMENT

A key product of this research—the foundation supporting the remainder of the research—is a framework to enable objective comparisons across various corridor treatments. The outcome of this work is called a Corridor Comparison Document (CCD). It is further discussed in Chapter 4 and presented in its entirety in Appendix A.

In general, the corridor comparison approach highlights topics practitioners use for guidance in their decision-making process when considering alternatives for a new corridor or converting a corridor from traffic signals to roundabouts. The CCD considers these issues in terms of tiers, where certain tiers have broader application to all roundabout corridors and others have lesser applicability while being useful as potential differentiators if considerations do not provide sufficient input. The classification used in the CCD is as follows:

- Tier I – critical considerations for all types of corridors (e.g., delay, travel time, constructability).

Evaluating the Performance of Corridors with Roundabouts

- Tier II – items that apply to many locations (e.g., access management, safety, pedestrian accessibility).
- Tier III – issues that may impact a smaller subset of corridors (e.g., effects on specific adjacent land uses such as schools or hospitals, familiarity of corridor drivers with roundabout operations).

Conceptually, these considerations can be separated into two broad categories: quantitative elements and qualitative elements. Quantitative elements, which promote a model-driven approach, are related to the operational models, data collection, and recommended analysis methodology. Qualitative elements include more anecdotal evidence and other considerations along the corridor that should be considered when weighing a roundabout corridor against a signalized arterial.

The CCD is intended to be an easy-to-access summary of corridor evaluation considerations and to offer practitioners easy and functional insights into the considerations and trends of the research findings. By providing users with an understanding of the evaluation consideration concepts, users will have the basis for applying the results of this research within the context of their own project environments and fully supplemented by other performance measures not investigated as part of this project. The framework has several key components:

- It includes a discussion of performance measures that may be applicable when evaluating corridors where roundabouts or signals are being considered.
- It includes guidance on how to assess performance measures. Guidance often refers readers to other documents focused on specific performance measures.

The CCD presents four examples on how to use the document. Examples present fictional corridor studies where roundabout and signal alternatives are under consideration.

2.2. STUDY SITE IDENTIFICATION

The research team identified 58 corridors as potential study sites for use in this project. The corridors were located in 18 different US states, and they included single-lane and multilane roundabouts. Per the definition of roundabout corridors in the RFP for this project, all 58 candidate corridors had at least three roundabouts in series, but several had five or more (up to ten) roundabouts in series.

These are shown in Exhibit 2-1 and listed in Exhibit 2-2, along with a summary of selected site characteristics that made them promising candidate sites.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-1: Known Roundabout Corridors, 2011 (Map)

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-2 Known Roundabout Corridors, 2011 (List)

	Road Name / Route Number	City	County	State	# Rbts	Length (mi)	Rbts / mi	Arterial Lanes	Rbt Lanes	Land Use	On-Street Parking?	Access Management	Inter-change?	Adjacent Rdbts?	Previous Control	Approx Year Built
1	AZ 179	Sedona	Coconino	AZ	6	3.2	1.9	2	1	Suburbanizing Rural	No	RIRO and some median breaks	No	No	Signals and TWSC	2008-2011
2	AZ 179	Oak Creek	Yavapai	AZ	4	1.1	3.6	2	1	Suburbanizing Rural	No	RIRO and some median breaks	No	No	Signals	2008
3	Cactus Rd	Scottsdale	Maricopa	AZ	3	1	3	2	1	Suburban - Residential	No	TWLTL, many driveways	No	Some	TWSC	2008
4	La Jolla Blvd	San Diego	San Diego	CA	4	0.6	6.7	2	1	Urban	Yes	Median, many driveways	No	Some	2 signals, 2 TWSC; TWLTL	2005 and 2008
5	O'Neill Dr	San Juan Capistrano	Orange	CA	4	0.9	4.4	2	1	Suburban - Residential	No	1 median break, not many driveways	No	Some	New	2003
6	Sienna Pkwy	San Juan Capistrano	Orange	CA	5	1	5	2	1	Suburban - Residential	No	Several median breaks for driveways	No	Some	New	2002
7	Fulton Ave	Ripon	San Joaquin	CA	3	1	3	3-4	1	Suburban	No	Frontage road, some full access driveways	No	No	New	2006
8	Manzanita Ave	Chico	Butte	CA	3	0.6	5	2	1	Suburban - Residential	No	Full access	No	No	Mixture	2008 or 2009
9	8th Ave	Chico	Butte	CA	3	0.8	3.8	2	1	Suburban - Residential	No	Full access	No	No	Probably TWSC	Between 2002 and 2005
10	Avon Rd	Avon	Eagle	CO	5	0.5	10	4	2	Suburban	No	Two side streets	Yes	No	Unknown	1997
11	Golden Road	Golden	Jefferson	CO	5	1	5	4	2	Suburban	No	Many - mix of RIRO and full access	No	No	Unknown	1998 (one added 2009)
12	William J. Post Blvd	Avon	Eagle	CO	6	1	6	4	2	Suburban	No	1 full driveway	Yes	No	New	Between 1999 and 2004
13	Lake Ave	Colorado Springs	El Paso	CO	3	0.9	3.3	2	1	Suburban	No	TWLTL, many driveways	No	Some	Unknown	1999 or earlier
14	Lowry Blvd	Denver	Denver	CO	3	0.8	3.8	4	2	Suburban	No	Some full access and RIRO driveways	No	No	Signals	1998
15	Hagen Ranch Rd	Boynton Beach	Palm Beach	FL	5	2.2	2.3	2	1	Suburban - Residential	No	A few side streets	No	No	Unknown	1998-2004
16	Morse Blvd	The Villages	Sumter	FL	6	3.3	1.8	4	2	Suburban	No	No driveways	No	Many	New	2003- 2007
17	Buena Vista Blvd (southern)	The Villages	Sumter	FL	10	4.8	2.1	4	2	Suburban	No	No driveways	No	Many	New	2003- 2007
18	Buena Vista Blvd (northern)	The Villages	Sumter	FL	4	2.7	1.5	4	2	Suburban	No	No driveways	No	Some	New	1998 - 2001
19	Spring Mill Road	Carmel	Hamilton	IN	7	4.5	1.6	2	1	Suburban	No	Many full access side streets	No	Many	Stop control	2008-2010
20	Old Meridian Street	Carmel	Hamilton	IN	4	1.3	3.1	4	2	Suburban	Limited	RIRO and some median breaks	No	Some	Signals	2007-2008
21	W Main St	Carmel	Hamilton	IN	5	2.2	2.3	2	1	Suburban - Residential	No	Many full access side streets	No	Many	Stop control	2005-present
22	W 136th St	Carmel	Hamilton	IN	4	3	1.3	2	1	Suburban - Residential	No	Many full access side streets	No	Many	Stop control	2005-present
23	Wanamaker Rd	Topeka	Shawnee	KS	3	2	1.5	4	1-2	Suburban - Residential	No	TWLTL, many driveways	No	Some	Mixture	2006 - 2007
24	Sheridan Rd	Olathe	Johnson	KS	2	0.3	6.7	3-4	1-2	Suburban	No	Many driveways	No	Some	Unknown	2000-2001
25	Renner	Lenexa	Johnson	KS	4	0.4	10	4	2	Suburban	No	Median with 1 right-in	No	No	TWSC	2007
26	Prairie Star Pkwy	Lenexa	Johnson	KS	7	1.2	5.8	4	2	Suburban	No	Median and no driveways	No	Some	New	2009
27	Scaggsville Rd (MD 216)	Scaggsville	Howard	MD	4	0.7	5.7	4	2	Suburban - Retail	No	No driveways	Yes	Some	Interchange was at-grade with signal	2002-2009
28	Hampstead Bypass (MD 30)	Hampstead	Carroll	MD	3	4.4	0.7	2	2	Rural	No	Expressway - No driveways	No	No	New	2009-2010
29	Maple Road	Farmington Hills	Oakland	MI	2	1	2	2	3	Suburban	No	Many full access side streets	No	No	Signals	2008 or 2009
30	Village Place Blvd	West Bloomfield	Livingston	MI	4	0.4	10	4	2	Suburban - Retail	No	2 full driveways	Yes	No	Signal and TWSC	2006
31	Longview Blvd	Lee's Summit	Jackson	MO	3	0.5	6	4	2	Suburban	No	Many full access side streets	No	Some	New	2005
32	Metro Parkway	Jackson	Hinds	MS	5	1.3	3.8	4	2	Becoming Urban	No	RIRO	No	Some	New	2004 - 2006
33	Shiloh Road	Billings	Yellowstone	MT	8	3.3	2.4	?	2	Suburban	No	Many full access side streets	No	Some	Mostly stop control	2009-2010
34	Hillsborough St	Raleigh	Wake	NC	2	0.5	4	4	1 and 2	Urban	No	Many full access side streets	No	Some	Signal and TWSC	2010
35	SR 67	Malta	Saratoga	NY	7	1.6	4.4	4	2	Suburban	No	Median, many RIROs	Yes	No	Mostly TWSC	2006
36	SR 85	Slingerlands	Albany	NY	4	1.2	3.3	4	2	Suburban	No	Median, several RIROs	No	No	Mostly new	2007
37	SR 590	Irondequoit	Monroe	NY	4	1.1	3.6	4	1	Suburban - Residential	No	Expressway - No driveways	No	No	Signals	2010
38	US 62	Hamburg	Erie	NY	4	1	4	2	1	Small Town	Yes	Many full access side streets	No	No	Signals	2007 and later
39	NW Crossing	Bend	Deschutes	OR	5	1.2	4.2	2	1	Suburban	No	Many	No	Many	TWSC and new	2005 - 2006

Exhibit 2-2 Con't Known Roundabout Corridors, 2011 (List)

	Road Name / Route Number	City	County	State	# Rbts	Length (mi)	Rbts / mi	Arterial Lanes	Rbt Lanes	Land Use	On-Street Parking?	Access Management	Inter-change?	Adjacent Rdbts?	Previous Control	Approx Year Built
40	Reed Market Dr	Bend	Deschutes	OR	5	1.1	4.5	4	2	Suburban	No	Median and no driveways	No	Some	New facility	2002
41	14th St	Bend	Deschutes	OR	4	1.7	2.4	2	1	Suburban	No	Many full access side streets	No	Many	Unknown	1999 - 2005
42	Maple Island Rd	Eugene	Lane	OR	3	0.2	15	2	1	Suburban Retail	No	No driveways	No	No	New development	2002
43	Via Bella	Williamsport	Lycoming	PA	3	0.3	10	2	1	Urban	No	No driveways	No	No	Signals	Between 2005 and 2008
44	Littlerock Rd	Olympia	Thurston	WA	4	1.1	3.6	2	1	Suburban	No	Many driveways	No	No	3 TWSC, 1 signal	2009 - 2010
45	Grandview Dr	University Place	Pierce	WA	5	1.2	4.2	2	1	Suburban - Residential	No	Some full access side streets	No	No	Stop control	2000
46	Borgen Blvd	Gig Harbor	Pierce	WA	4	1.4	2.9	4	2	Suburban	No	Mixture of full and RIRO	Yes	Some	Unknown	2000-2007
47	Dike Access Rd	Woodland	Columbia	WA	3	0.2	15	2?	1	Suburban	No	No driveways	Yes	No	TWSC	2010 - 2011
48	SR 539	Lynden	Whatcom	WA	4	6.5	0.6	4	2?	Suburban	No	unknown	No	No	mixture	2009 - 2010
49	SR 11/SR 20	Burlington	Skagit	WA	3	0.5	6	2	2	Suburban	No	Many full access driveways	Yes	No	Unknown	2008 - 2010
50	Valley Mall Blvd	Yakima	Yakima	WA	3	0.2	15	4?	2	Suburban	No	No driveways	Yes	No	2 signals, 1 TWSC	2010 - 2011
51	SR 145	Richfield	Washington	WI	5	0.6	8.3	2 and 2	1 and 2	Suburban	No	Some full access driveways	Yes	No	TWSC	2009
52	Sheuring Rd	Green Bay	Brown	WI	3	1	3	2?	1?	Suburban	No	Many full access driveways	No	No	stop control	2004
53	Lineville Rd	Green Bay	Brown	WI	5	1	5	2	1	Suburban	No	Some full access driveways	No	No	stop control	1999 - 2007
54	Springdale St	Mt. Horeb	Dane	WI	5	1.4	3.6	4	2	Suburban	No	Some full access driveways	No	No	unknown, some new	2004 - 2006
55	SR 42	Sheboygan	Sheboygan	WI	3	0.4	7.5	4	2	Suburban	No	No driveways	Yes	No	unknown	2007
56	CR O	Rice Lake	Barron	WI	3	0.4	7.5	2	1	Small Town/Suburban	No	1 right-in	Yes	No	stop control	2006
57	Chicago St	Green Bay	Brown	WI	3	0.5	6	2	1	Suburban	No	Some full access driveways	No	No	Unknown	2001
58	Evergreen Dr	Appleton	Outagamie	WI	3	0.5	6	2?	1?	Suburban	No	No driveways	Yes	No	2 signals, 1 TWSC	2010 - 2011

Note: RIRO = right in right out, TWSC = two-way STOP-controlled, and TWLTL = two-way left-turn lane.

Evaluating the Performance of Corridors with Roundabouts

Of these corridors, the research team prioritized the potential sites, based on the team's judgment as to which corridors appeared to have the most promise for positive research outcomes. Foremost, sites were selected if the research team believed they had sufficiently high volumes from which meaningful traffic operations results could be obtained. The research team assessed this based upon land use and, in some cases, team members' knowledge of study corridors.

The research team also considered the range of possible project catalysts that may have led to the initial corridor evaluation. These catalysts help establish a project context and influence the type of data that may be useful in conducting corridor comparisons. The range of project catalysts included:

- A new greenfield corridor
- An existing signalized corridor being evaluated because of capacity or safety performance
- An existing roundabout corridor
- A corridor with a specific access management focus
- A corridor explicitly focused on multimodal considerations
- A corridor project driven by community enhancement objectives, speed management needs, or economic development or growth opportunities

Beyond this, the site selection was guided by a variety of qualities and contexts, including:

- Saturated/unsaturated flow conditions
- Corridor land uses (commercial, residential, etc.)
- Time-of-day variations
- Roundabout density (spacing)
- Wide range of motorized and non-motorized users
- Roundabout types (single vs. multilane)
- Low vs. substantial side-street traffic
- How long the roundabouts had been operating
- Type of access management and intersection controls within the corridor
- Geographic diversity
- Efficiency of data collection (proximity to other sites to consolidate travel costs, etc.)
- Number of roundabouts
- Mixture of land use
- Range of posted/operating speeds
- Corridor length and roundabout spacing
- Presence/absence of traversable median and/or curb

Evaluating the Performance of Corridors with Roundabouts

- Presence/absence of sidewalks and/or bicycle lanes
- Presence/absence of on-street parking

With these elements in mind, the research team identified the following nine roundabout corridors from the list as preferred data collection sites:

- MD 216 in Scaggsville, Maryland
- La Jolla Boulevard in San Diego, California
- Old Meridian Street in Carmel, Indiana
- Spring Mill Road in Carmel, Indiana
- Borgen Boulevard in Gig Harbor, Washington
- SR 539 in Whatcom County, Washington
- Golden Road in Golden, Colorado
- Avon Road in Avon, Colorado
- SR 67 in Malta, New York

Full field data reports for these nine corridors are included in the NCHRP web-only document accompanying this report as Appendices B through J. Photos of the corridors taken by the research team are included in the web-only document as Appendix K. A later section of this chapter presents a summary of data from the nine corridors.

2.3. DATA COLLECTION PLAN

2.3.1. PILOT SITES

In developing the procedure to collect the field data for this project, the research team wanted to ensure the procedure was flexible enough to be effective at corridors with a variety of characteristics. As shown in Exhibit 2-2, the corridors under consideration varied widely in geographical location, number of roundabouts, corridor length, roundabout spacing, and other key variables. As a result, the team sought a data collection procedure that would capture meaningful data under varied conditions. The team conducted pilot studies at two corridors, and revised the data collection procedure for use at the remaining seven locations.

Pilot studies are commonly used in research projects to develop data collection procedures. Pilot sites were selected with the intent of including as many key site characteristics as possible. Geographical location, adjacent land use, expected vehicle speeds, corridor length, and roundabout spacing were all considered in the selection of pilot sites. Using the corridor information summarized in Exhibit 2-2, and considering the ability of the team to obtain further information (e.g., as-built plans, traffic volumes) from the appropriate road agency, the researchers looked for the corridors with the greatest potential for providing useful data as well as information on the appropriateness of the data collection procedure.

Evaluating the Performance of Corridors with Roundabouts

The research team selected two corridors to use as pilot sites: Maryland State Route 216 (MD 216) in Scaggsville, Maryland, and La Jolla Boulevard in San Diego, California. The corridor on MD 216 is located in a suburban area between Washington, D.C., and Baltimore. MD 216 is a four-lane divided roadway and has four roundabouts, two of which are at ramp terminals as part of the interchange with US 29. There are no intermediate access points between any of the roundabouts. The corridor is automobile-dominated, with little pedestrian or bicycle activity. The La Jolla Boulevard corridor is located in the Bird Rock neighborhood of San Diego. La Jolla Boulevard is a two-lane divided roadway with five roundabouts. Much of the roadway has bicycle lanes and on-street parking (either parallel or diagonal). All intermediate access points are right-in, right-out, and most are driveways to houses or parking lots with 20 or fewer spaces. The corridor has an urban character with a moderate degree of pedestrian and bicycle activity.

2.3.1.1. *Data Collection Techniques*

The objective of the team's proposed data collection plan was to emphasize flexibility. The initial pilot data collection procedure was designed so many performance measures could be collected, depending on input from the project panel. One of the key performance measures for roundabout corridors was defined as the travel time of through traffic and other key origin-destination pairs. To obtain that data, the research team designed a data collection procedure for the pilot sites that included multiple travel-time data collection techniques, which are described in the following paragraphs.

Bluetooth Technology in the form of multiple roadside units at fixed locations recorded signature identification numbers (MAC addresses) of Bluetooth-equipped cell phones and other devices of the traffic stream. It is a non-intrusive data collection reliably capturing travel times of approximately 10% of the traffic stream. Bluetooth measurements are made continuously, providing a 24-hour distribution of travel times. The technology is therefore uniquely capable of quantifying the variability of travel times throughout the day, and further provides a high sample size for statistical comparisons.

In general, several challenges exist in applying Bluetooth data to surface street corridors. First, the presence of driveways results in frequently interrupted trajectories, and these intermediate stops are not registered by Bluetooth units at the termini. Second, depending on local traffic patterns, the fraction of vehicles actually traveling the entire corridor may be limited. And third, an urban corridor is likely to have a significant portion of non-automobile users; Bluetooth devices used by bicyclists, pedestrians, or transit passengers may not be distinguishable from those in passenger vehicles, depending on congestion levels. In fact, on congested corridors a bicyclist may traverse the corridor faster than a vehicle. To help overcome these issues, the team decided to use GPS travel-time data to calibrate the Bluetooth data extraction. Using a known Bluetooth MAC address of a device in the GPS probe vehicle, defined benchmarks were created to help filter the remaining data.

Evaluating the Performance of Corridors with Roundabouts

GPS Technology, in the form of an in-vehicle data logger, continuously recorded the speed and position of the vehicle as it traveled along the corridor (in 1-second intervals). GPS travel-time trajectories provide a detailed assessment of travel characteristics, as travel-time data supplemented by speed profiles, delay estimates, and the number of stops along the traveled path. Specifically for roundabout corridors, GPS unit data readily provided an estimate of the geometric delay (relative to free-flow speed) associated with individual roundabouts.

In designing the travel-time runs, the research team proposed to have one vehicle continuously loop through a pre-defined route extending beyond the beginning and end of the corridor. The vehicle operated during peak and off-peak periods, allowing the GPS unit to collect data in both periods to calibrate the continuous Bluetooth monitoring. Another benefit of collecting data during off-peak periods was to obtain a sample of free-flow trajectories to estimate the geometric delay incurred at roundabouts.

During the first pilot study (MD 216), the research team decided to conduct GPS travel-time runs for routes involving left turns onto and off of the corridor in addition to through routes. These additional runs were conducted because the corridor was relatively short and there was time remaining after through-route data were collected. Also, to account for unforeseen issues, the team brought additional staff and vehicles to the first site.

All travel-time runs were logged on manual tally sheets, where the driver recorded the starting time, end time, and any noteworthy events for every route. This record was completed at the turnaround points to prevent any distractions during driving. A subset of GPS runs was further supplemented with in-vehicle video records of the traveled route. These recordings were made with the intent that they would be useful to present features of a particular roundabout corridor to the panel or other audience. The team also felt video recordings are useful to review certain features of the corridor after returning to the office.

Exhibit 2-3 shows the routes of the four left-turn travel-time runs conducted on MD 216, and Exhibit 2-4 shows the routes of the four left-turn travel-time runs conducted on La Jolla Boulevard. Through runs were also conducted on each corridor, but they are omitted from the exhibits below for clarity.

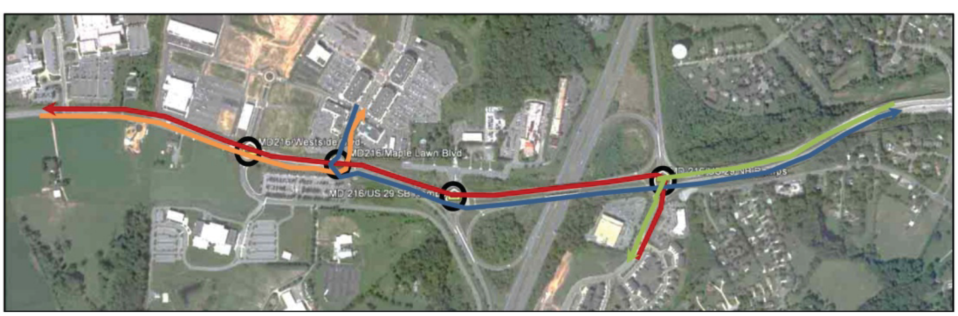


Exhibit 2-3: Schematic of Left-Turn Travel-Time Routes for MD 216

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-4: Schematic of Left-Turn Travel-Time Routes for La Jolla Boulevard

The corridor travel times, estimated through a combination of both approaches and the travel-time data, were further supplemented with other data collection technologies:

Tally Sheets: Many of the necessary data collected as a part of a roundabout corridor evaluation can be quickly gathered in the field using tally sheets from a good vantage point, including delay and queue measurements. The research team applied video to some extent to provide a permanent record of conditions during the field study and to collect data items difficult to observe in the field in real-time; however, tally sheets improved the economy of office data extraction for readily observed measures. The tally sheet data collection approach provided a fast, efficient method of documenting necessary data by the time the team left each data collection site. Tally sheets were also critical for any of the more qualitative and perception-based corridor characteristics, including access management practices, pedestrian and bicycle accommodations, and construction details. A structured field survey form assured consistency across sites and inter-observer reliability.

Video: To collect traffic volume, the research team used video recording. In addition to recording the traffic movements at each roundabout in the study corridor, video was used to create a 12-hour volume profile of the corridor. Members of the research team brought video cameras to MD 216, set them up, and later played back videos to count vehicles. The time and effort required to transport, set up, and operate video cameras proved to be substantial. The cameras and equipment for attaching them to poles were transported via airplane in an overweight piece of baggage. A ladder was required to set up the cameras at an adequate vantage point on poles or tree trunks. The cameras had to be taken down between Day 1 and Day 2 to recharge batteries, transfer video files, and prevent theft or water damage if there was rain. Transferring video files in preparation for Day 2 took much of the night.

Considering these issues, the research team employed Quality Counts, LLC, to collect video data and perform turning-movement counts at La Jolla Boulevard. The Quality Counts camera setup was capable of recording video continuously for approximately 60 hours in any weather conditions. The cost of using Quality Counts to record video data and perform turning-movement counts from the video at La Jolla Boulevard was approximately the same as the cost to use the research team's own resources at MD 216 and yielded a larger sample of data.

Lidar: The research team also wanted to directly collect a selection of speed data at key locations for comparison to the speed data obtained from the other data collection methods. To fulfill that need, the researchers used a handheld lidar gun to measure speeds, which were recorded manually on tally sheets. The researchers measured spot-speeds of traffic entering the roundabout at each end of the corridor, as well as traffic circulating within those two roundabouts. A total of 30 spot-speed measurements were recorded at each point.

Photographs and Handwritten Notes: The research team took additional notes as needed on physical measurements of key geometric features, predominant adjacent land use, and other site characteristics of note. The research team also further documented the conditions at each site with digital photographs of features of each roundabout and corridor, as well as additional noteworthy characteristics of each site.

The research team created an electronic repository of pertinent site characteristics collected at the study sites. This includes electronic files of data tables, digital copies of video recordings, and scans of handwritten notes and tally sheets. The electronic record enabled data sharing among the team and protected the data over time.

2.3.1.2. *Agency-Provided Data*

Some data elements were not readily obtained in the field, but instead from operating agencies. Local agencies provided copies of as-built construction plans and any existing traffic volume data. Researchers also interviewed state and local transportation personnel (discussed in a later section of this chapter) to obtain additional first-hand experiences with the roundabout corridors.

2.3.1.3. *Collection Schedule*

During the pilot phase, the team proposed a two-stage data collection schedule, using a combination of data-enhanced scouting trips and full-detail data collection trips. The scouting trips were scheduled for two team members to complete over 1.5 days. The purpose of these trips was to make initial field observations and determine usability for data collection. To maximize the use of project resources, the team proposed to use this initial scouting trip for some preliminary data collection. Specifically, the team conducted a sample of GPS travel-time runs, recorded some video, and conducted a structured field survey of other corridor characteristics. The intent was that these scouting trips would then be followed up by detailed data collection trips at a selected number of sites.

2.3.1.4. *Findings from Pilot Data Collection*

The research team scouted and collected data at the two pilot corridors (MD 216 and La Jolla Boulevard) in the fall of 2011. After reviewing the data obtained and discussing the experiences at those two sites, the research team determined some changes could be made to the data collection procedure. These changes were discussed with the project panel and refined based on the panel's input. As a result of the pilot data collection and discussions with the panel, the research team made the following conclusions:

Evaluating the Performance of Corridors with Roundabouts

- A two-phase data collection schedule was not necessary; there was sufficient time within a single three-day trip to collect the needed data at a study site. Extending the study period to three days provided the time needed to take pictures and compile field notes in addition to the other needed data collection. Video recorders could be installed prior to the three-day period and removed afterward to maximize the recording time.
- The equipment needed to be "efficiently portable." While all of the equipment used in the pilot data collection effort was useful to varying extents, it was not always the most efficient means of obtaining data. For example, the Bluetooth equipment provided marginally useful data on the longer corridor of MD 216 and operated efficiently once on-site, but it was expensive to transport and less useful on the shorter La Jolla Boulevard corridor with its closer roundabout spacing and on-street friction. In general, the Bluetooth equipment was useful at providing corridor-wide data, but was unable to capture more-detailed data such as geometric delay at a specific roundabout or operating speed on a specific midblock segment.
- Video recorders needed to be robust enough to record for long periods of time without external memory or power. The team made improvements in the video recording procedures between the two pilot studies, which were successful in improving efficiency. Also, they needed to be portable enough to easily transport from place to place and to mount in locations that would provide the needed point of view of each intersection. Use of Quality Counts, LLC, was superior to use of the team's own equipment and labor.
- GPS data was sufficient in providing data on travel time within the corridor. While having supplemental Bluetooth data was somewhat useful, it was deemed unnecessary in comparison to the effort and cost required to transport and install the devices.
- Other data items, while potentially informative, were determined to not be critical to the needs of the project. As a result, the research team consolidated the data items collected, which improved the efficiency of the data collection procedures and allowed for the revised three-day schedule.

Section 2.3.2 describes the revised data collection procedure in more detail.

2.3.2. REVISED DATA COLLECTION PROCEDURE

The following is a description of the data collection procedure revised to reflect the team's experience with collecting data at the two pilot sites. The team used this procedure at the seven other data collection sites. The team focused its field data collection on GPS-based travel-time, spot-speed measurements at critical locations, and a walk-through survey and photo log of the corridor. In addition, the team comprehensively deployed video equipment to capture the equivalent of (at least) one 12-hour period on video over two consecutive days for each roundabout along the corridor. The purpose of the video recordings was primarily to extract approach volume counts and intersection turning-movement counts.

One additional variation in the data collection protocol was the explicit consideration of selected left-turning movements along the corridor in the travel-time studies. As such, the team used GPS units to collect travel-time data on up to four left-turning movements into or out of each corridor. This data collection was in addition to the amount of through-movement data collection already proposed for collection, and was designed to provide supplemental data to create a more comprehensive look at the side-street performance of roundabout corridors.

The revised data collection plan required a team of two personnel over a three-day period. The following data were collected during each trip:

- <u>Travel-time data (via vehicle-mounted GPS units)</u> – AM Peak (2 hours × 2 days), PM Peak (2 hours × 2 days), Off-Peak Midday (2 hours × 2 days), Off-Peak Evening (2 hours)
    - Through-movement runs over the entire corridor
    - Two routes with left turns onto the corridor
    - Two routes with left turns off of the corridor at a roundabout

  The left-turn routes were selected based on a preliminary assessment of those movements likely to experience the most delay and/or variation throughout the day. Left-turn routes without nearby turnaround points, such as routes involving freeway on- or off-ramps, were generally avoided.

- <u>Spot-speeds (via lidar gun)</u> – Entering and circulating free-flow speed at two approaches to two different roundabouts at a sample size of 30 observations each.

- <u>Site characteristics</u> – Gathered during a walk-through in both directions, associated with a detailed photo log of the corridor and critical side streets.

- <u>Video observations</u> – Video collected for equivalent of a 12-hour period from overhead camera locations for the following data items:
    1. Turning-movement counts
    2. Midblock volumes
    3. Pedestrian and bicycle volume and operations

4. Arrival patterns (platooning)

For reasons previously discussed, the team used a vendor to collect video data and conduct counts from the video.

The extensive time spent at the corridor allowed the research team to qualitatively assess operational characteristics that were not explicitly captured by the data collection plan. For example, queue lengths were not recorded, but research team members were able to observe if queues from a roundabout spilled back to an adjacent roundabout and impacted its operations.

Use of the GPS units and speed guns, as well as the procedure for documenting site characteristics, remained largely the same as the process in the pilot study effort. The primary differences between the pilot data collection and the revised data collection were eliminating Bluetooth data collection and using a vendor for collecting and processing video data. As discussed in Section 2.3.1.4, while the Bluetooth units had potential for providing a great deal of data, the datasets were not as useful as originally envisioned, and transporting the units was cumbersome and expensive.

Exhibit 2-5 outlines how the research team ultimately used each of the data collection techniques to capture the necessary data.

Exhibit 2-5: Application of Data Collection Techniques

Data/Performance Measures	Means of Collecting Data
Corridor travel time	GPS
Operating speed and speed profiles/variability through corridor	Speed gun samples, GPS
Pedestrian and bicycle volumes	Video records
Approach delay	GPS
Travel time for side-street trips with left turn onto/off of side street	GPS
Peak period turning-movement counts	Video records
Twelve-hour corridor-volume profile	Video records
Design characteristics (median type, number of driveways, presence/absence of sidewalks, etc.)	Photographs, notes

Exhibit 2-6 displays the work schedule to collect each of these types of data with a staff of two people over a three-day period. The efficiencies obtained in the revised schedule also made it possible to sufficiently study and document a pair of sites within a five-day period, as shown in Exhibit 2-7. The schedules show the responsibilities of the two staff members (1 and 2) associated with each data item. The schedule was designed to be generous in the allocation of time to each data item, allowing flexibility in the event that rain or unforeseen complications arose. The schedule also provided time for breaks, which were critical for long data collection days that involved a great deal of driving on repeated round-trips through each site's travel-time routes.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-6: Work Schedule for Single-Site Data Collection

	Day 1						Day 2							Day 3		
Data Item	**7-9**	**9-11**	**11-1**	**1-3**	**3-5**	**5-7**	**6-7**	**7-9**	**9-11**	**11-1**	**1-3**	**3-5**	**5-7**	**7-9**	**9-11**	**11-1**
Site Characteristics (Tally)	KICKOFF MEETING / SCOUT CORRIDOR		LUNCH		1,2				1	LUNCH	1					DEBRIEFING
Corridor travel time-through (GPS)				1		1		1				1	1	1		
Corridor travel time-left turns (GPS)				2		2		2				2	2	2		
Spot Speed Measurements (Tally)					2	2			2		2					
Miscellaneous Studies (Video)	SETUP 3	V	V	V	V	V	V	V	V	V	V	V	V	V	Take-Down	

1 - Staff Person 1	E - Equipment
2 - Staff Person 2	V - Video
3- Quality Counts Staff	B# - Break + Analyst #

Exhibit 2-7: Work Schedule for Double-Site Data Collection

Day	1								2									3		
Data Item	**7-9**	**9-11**	**11-1**	**1-3**	**3-5**	**5-7**	**7-8**	**8-10**	**6-7**	**7-9**	**9-11**	**11-1**	**1-3**	**3-5**	**5-7**	**7-8**	**8-10**	**7-9**	**9-11**	**11-1**
Site Characteristics (Tally)	KICKOFF MEETING / SCOUT CORRIDOR		LUNCH		1,2		DINNER				1	LUNCH	1			DINNER				Lunch-Debrief
Corridor travel time-through (GPS)		1		1		1		1		1				1	1		Backup	1	Backup	
Corridor travel time-left turns (GPS)		2		2		2		2		2				2	2			2		
Spot Speed Measurements (Tally)					2	2					2		2		2					
Miscellaneous Studies (Video)	SETUP 3	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	

1 - Staff Person 1		E - Equipment
2 - Staff Person 2		V - Video
3- Quality Counts Staff		B# - Break + Analyst #

Day	3					4									5						
Data Item	**1-3**	**3-5**	**5-7**	**7-8**	**8-10**	**6-7**	**7-9**	**9-11**	**11-1**	**1-3**	**3-5**	**5-7**	**7-8**	**8-10**	**6-7**	**7-9**	**9-11**	**11-1**	**1-3**	**3-5**	**5-7**
Site Characteristics (Tally)		1,2		DINNER				1	LUNCH	1			DINNER					LUNCH			DEBRIEFING
Corridor travel time-through (GPS)	1		1		1		1				1	1		Backup		1	1		Backup	Backup	
Corridor travel time-left turns (GPS)	2		2		2		2				2	2				2	2				
Spot Speed Measurements (Tally)		2	2					2		2		2									
Miscellaneous Studies (Video)	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	V	Take-Down	

Evaluating the Performance of Corridors with Roundaboutsan

Evaluating the Performance of Corridors with Roundabouts

2.4. SITE CHARACTERISTICS SUMMARY

As discussed previously (Section 2.2), the research team identified 58 roundabout corridors in the United States. Two were visited during the pilot study data collection in Phase I, and an additional seven corridors were visited in Phase II. The nine corridors studied represent a diverse set of US roundabout corridors. They include the following range of attributes:

- The sites represent good geographic diversity, including east coast, midwest, mountain west, and west coast states.
- The number of roundabouts per corridor ranged from four to seven.
- The corridors were a mix of two-lane and four-lane arterials.
- The roundabouts were a mix of single-lane and multilane roundabouts.
- Speed limits ranged from 25 mph to 50 mph.
- Corridor lengths ranged from 0.5 miles to 4.5 miles.
- Corridor average roundabout spacing ranged from 650 feet to 6,465 feet.
- Land uses were primarily suburban, with one urban corridor and one rural corridor.
- Four corridors included a freeway interchange.
- Opening dates ranged from 1997 to 2011.
- Seven corridors had a non-traversable median for a portion of the corridor.
- The number of driveways ranged from 0 to 67.
- Two corridors had on-street parking.
- Eight corridors had sidewalks and crosswalks at the roundabouts.
- Two corridors had bike lanes.
- Peak-hour traffic volume and side-street traffic volume varied greatly along some corridors.
- Twelve-hour (7 a.m. to 7 p.m) arterial volumes measured near the midpoint of each corridor ranged from 9,000 to 23,000.

The research team selected corridors believed to have moderate to high traffic volume based on land use and the team's personal knowledge of the corridors because a wide range of traffic volumes is desirable when developing operational models. However, roundabouts remain relatively new in the United States, and most roundabout corridors are in the early years of their design life. As a result, traffic volumes have not grown to design-year forecasts, and capacity is available. Generally speaking, the study corridors were observed to operate below capacity with low delays during all periods of study (a.m. peak, p.m. peak, and off-peak). The field data collection approach (i.e., floating car runs along the arterial) collected only a small sample of side-street approach delays as

Evaluating the Performance of Corridors with Roundabouts

part of travel-time runs involving left turns, but based on the team's observations they were generally similar to the arterial approach delays.

Exhibits 2-8, 2-9, and 2-10 present a summary of characteristics of the nine study corridors.

Exhibit 2-8: Characteristics of Data Collection Sites

	Number of Roundabouts	Arterial # Lanes	Roundabout # Lanes	Posted Speed Limit (mph)	Corridor Length (mi)	Average Roundabout Spacing (ft)	Area Type	Includes Interchange	Construction Year
MD 216 (Scaggsville, MD)	4	4	2	45	0.7	1200	Commercial	Yes	2002, 2009
La Jolla Boulevard (San Diego, CA)	5	2	1	25	0.6	715	Urban	No	2005 to 2008
Spring Mill Road (Carmel, IN)	7	2	1	40	4.5	3950	Residential	No	2005 to 2009
Old Meridian Street (Carmel, IN)	4	4	2	40	1.3	1640	Commercial/ Residential	No	2006
Borgen Boulevard (Gig Harbor, WA)	4	4	2	35	1.4	1695	Commercial/ Residential	Yes	2000 to 2007
SR 539 (Whatcom County, WA)	4	4	2	50	3.7	6465	Rural	No	2010
Golden Road (Golden, CO)	5	4	2	25 to 35	1	1360	Commercial	No	1998 to 1999, 2004
Avon Road (Avon, CO)	5	4	2 to 3	25	0.5	650	Commercial	Yes	1997
SR 67 (Malta, NY)	7	2 to 4	1 to 2	35 to 45	1.6	1400	Commercial/ Residential	Yes	2006 to 2011

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-9: Access Management and Pedestrian/Bicycle Characteristics of Data Collection Sites

	Median	Driveways	On-Street Parking	Sidewalks	Crosswalks	Peak-Hour Pedestrian Volumes (Intersection Totals)	Bike Lanes
MD 216	Raised	0	No	Yes	Yes	Not counted	No
La Jolla Boulevard	Raised	17	Yes	Yes	Yes	10 to 60	Yes
Spring Mill Road	Mostly none	33	No	Var-ies	Yes	0 to 12	No
Old Meridian Street	Mostly raised	22	Yes	Yes	Yes	0 to 10	No
Borgen Boulevard	Varies	8	No	Yes	Yes	0 to 4	Yes
SR 539	Cable	67	No	No	Yes	0 to 8	No
Golden Road	Raised with openings	19	No	Yes	Yes	4 to 14	No
Avon Road	Raised, 1 opening	1	No	Yes	Yes	0 to 28	No
SR 67	Raised, none	22	No	Yes	Yes	0 to 24	No

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-10: Volume and Speed Observations at Data Collection Sites

	Peak-Hour Traffic Volumes (Arterial)	Peak-Hour Traffic Volumes (Side Streets)	Measured 12-Hour Arterial Volume (7 a.m. to 7 p.m.)	Roundabout Entering Speed (mph)	Roundabout Circulating Speed (mph)
MD 216	1,600 to 2,100	100 to 800	Not counted	23	21
La Jolla Boulevard	1,000 to 1,500	100 to 200	11,000	17	15
Spring Mill Road	1,100 to 1,600	200 to 1600	13,000	24	20
Old Meridian Street	500 to 1,200	80 to 1300	9,000	23	19
Borgen Boulevard	1,000 to 2,000	500 to 1400	14,000	18	15
SR 539	800	6 to 200	23,000	23	20
Golden Road	1,000 to 1,400	20 to 400	9,000	18	18
Avon Road	1,300 to 1,800	300 to 1000	13,000	N/A	16
SR 67	600 to 1,200	70 to 800	15,000	20	21

Evaluating the Performance of Corridors with Roundabouts

2.5. CORRIDOR OWNER INTERVIEWS

The research team held interviews with the owners of nine roundabout corridors at which field data was collected for this project. The interviews provide an insight into the creation and history of these roundabout corridors, agency and community goals for the corridors, and their effectivness at meeting those goals. The following were several objectives of performing the interviews:

- Obtain any corridor-specific data for use in the Work Plan.
- Gain an insight into why roundabouts were chosen for the specific corridor.
- Obtain any studies of the roundabout corridor applicable to this project as a whole and supplement the literature review.
- Engage the operators of roundabout corridors and understand what guidance and performance measures would be most useful to them when considering roundabouts for intersection control on an arterial.

Interviews with the Maryland State Highway Administration and the City of San Diego were held in person, and interviews with other agencies were conducted over the phone. Two corridors were covered in a single interview with the City of Carmel, Indiana.

The interviews reveal a variety of contexts in which roundabout corridors have come into being. Some of the corridors were designed purposefully as a complete corridor; others grew organically over time. The variety of motivations for considering roundabouts, the variety of levels of interaction with the public, and the design treatments ultimately constructed reinforce the notion that each corridor is a unique installation. The CCD developed in this project presents a process that fits well with each of these corridors, primarily because it enables case-specific comparisons and evaluations.

Summaries of each interview are provided in the following sections.

2.5.1. MD 216 – SCAGGSVILLLE, MD

Mike Niederhauser of the Maryland State Highway Administration (SHA), Office of Traffic and Safety, visited KAI's Baltimore office in November 2011. Mr. Niederhauser has served as SHA's *de facto* roundabout coordinator since the construction of Maryland's first roundabout nearly 20 years ago. He provided the project team with background information on the MD 216 roundabout corridor. One team member participated in person and another participated via video conference.

MD 216 was not envisioned by SHA as a roundabout corridor, but rather developed into one over time as roundabouts were added in proximity to other roundabouts. Planning for the US 29/MD 216 interchange began in the mid-1990s. The two roads met at an at-grade, signalized intersection at the time, and SHA was converting US 29 into an expressway as well as widening and relocating MD 216 between US 29 and I-95. The state considered a number of interchange forms and ramp terminal control options, and, ultimately, selected

Evaluating the Performance of Corridors with Roundabouts

two-lane roundabouts for the two ramp terminal intersections. SHA believed roundabouts offered a number of benefits, including reduced delay, and traffic forecasts indicated two-lane roundabouts would sufficiently serve future demand. SHA and their consultants used SIDRA to analyze traffic operations. These two roundabouts, as well as the others described below, were primarily analyzed in isolation and not as part of a series.

After the opening of the interchange in 2001, two roundabouts to the west were constructed to accommodate private developments. The first of these roundabouts was an intersection with a new road, Maple Lawn Boulevard. The developers of Maple Lawn initially considered a signalized intersection, but analysis indicated queues from the signal would spill back into the roundabout. A roundabout was not projected to have queue spillback issues and was ultimately selected for the intersection. The MD 216/Maple Lawn Boulevard roundabout opened around 2004.

The final roundabout on the corridor, at MD 216/Old Columbia Pike, opened in 2009. This intersection was initially two-way stop-controlled and improvements were required due to development. A roundabout was selected for a number of reasons including operational performance.

According to the SHA, the public and other stakeholders have generally had a positive reaction to the roundabouts, both initially and as others have been added to the corridor.

2.5.2. LA JOLLA BOULEVARD – SAN DIEGO, CA

During the visit to La Jolla Boulevard, one member of the research team met with Siavash Pazargadi, a Senior Traffic Engineer with the City of San Diego. Mr. Pazargadi discussed the history of the La Jolla Boulevard corridor and provided the team with several of the studies that led to the implementation of a road diet and the roundabouts.

The La Jolla Boulevard corridor is located within a neighborhood business district surrounded by residential areas. Before constructing the roundabouts, La Jolla Boulevard was a five-lane cross section with parallel parking. One of the five intersections ultimately converted to roundabouts was originally a signal (at Bird Rock Avenue), and one was originally an all-way stop-controlled intersection (at Forward Street). The remaining intersections were two-way stop-controlled. The corridor serves an average daily traffic volume of 22,000 to 23,000 vehicles per day.

In the late 1990s, there was considerable interest by the community to slow down traffic. Businesses in the corridor had high turnover and were unable to attract customers compared to other business districts in the area. Speeds along La Jolla Boulevard were in the range of 35 to 40 mph, which made the corridor less comfortable for bicyclists and pedestrians. The community groups in the area are among the most active in the San Diego area. An early proposal was to reduce a travel lane in each direction and add diagonal parking. However, four lanes would be needed at the all-way stop-controlled intersection at Forward Street. In addition, neighborhood groups were concerned over the potential for diversion of traffic into adjacent neighborhoods.

The City of San Diego engaged Dan Burden to conduct some design charettes to explore ways to enhance the corridor, and Michael Wallwork provided concept designs for roundabouts at each of the key intersections. The roundabout analysis conducted in SIDRA suggested that the single-lane roundabouts could accommodate approximately 27,000 vehicles per day, thus allowing a three-lane cross section to be implemented. To reduce the likelihood of diversions to adjacent streets, a number of traffic-calming measures were introduced, including neighborhood traffic circles; later data collection proved the measures were effective.

The corridor transformation was implemented over a period of seven years. The City tried to use as many existing street features as possible to minimize right-of-way acquisition, and the project was coordinated with other utility work (the water mains were replaced simultaneously). The two roundabouts on the south end were built by a developer of an adjacent 139-unit condominium complex in 2005–2006; the remaining roundabouts were built in 2007–2008. Each roundabout had a construction cost of approximately $800,000 to $900,000. For those parts not funded by the developer, funding came from the San Diego Association of Governments (SANDAG), the City's Capital Improvement Program (CIP), and a community development impact fee. A maintenance assessment district was established to pay for landscaping, with the whole community contributing based on distance from the corridor.

Public opinion within the corridor has generally been positive. Approximately 10 to 15 percent of the residents expressed no opinion throughout the project. The City has received no complaints and has operated under the principle that no news is good news. There have been occasional comments in the local paper. The businesses have expressed support for the roundabouts since their implementation, although the local economy has not been kind in recent years.

The most important lesson learned from this corridor is the need for coordination from beginning to end. Mr. Pazargadi served in this role throughout as the project passed from planning to design engineering to construction engineering. The compartmentalization that occurs in large organizations like the City of San Diego can make it difficult for a project of this magnitude to succeed as originally envisioned. Seamless coordination from a project champion and trust-based relationships throughout the project with the community and the city council helped in achieving success.

A few of the other lessons learned include the following:

- In-pavement flashers were used at crosswalks throughout the project. A less expensive brand used in the south end has had durability problems, but newer units installed on the northern end have been more reliable. The in-pavement flashers use pedestrian push buttons for activation; a passive pedestrian detection system was desired but never implemented. Pedestrian crosswalk signals were considered but rejected due to cone of vision challenges.

Evaluating the Performance of Corridors with Roundabouts

- A project like this has constant challenges. “If you think the project is good, stick with it.”
- A toolbox with factual statistics is helpful in communication, particularly when discussing issues related to pedestrian, bicyclist, and elderly issues.
- The proof of a successful project is in its implementation and use by the community. “If you do a good job, people will want you to do more.”

2.5.3. CITY OF CARMEL, IN

Two members of the project team held a conference call with Mike McBride, City Engineer with the City of Carmel, Indiana. Both the Spring Mill Road and Old Meridian Street corridors are located within one quarter mile of US 31 in Carmel. The history of both corridors was discussed with the project team during the call.

2.5.3.1. *Spring Mill Road*

Spring Mill Road is part of the county’s one-mile grid network. It was once a gravel road and was paved in the middle of the 20th century. In the 1990s, some held the belief the roadway would someday be widened to four lanes, but this expansion was not desired on the City’s part. Spring Mill Road serves as a transitional area between the commercial areas to the east along US 31 and the residential areas to the west; therefore, the City sought to preserve a narrower roadway to maintain consistency with the residential land use.

In the early 2000s, most of the intersections on Spring Mill Road were all-way stop-controlled (AWSC), and some operated poorly. While the City did not conduct a corridor study, it did study congested intersections individually. In 2005, the first roundabout on Spring Mill Road opened at 116th Street. By 2009, the last of the seven roundabouts currently on Spring Mill Road was opened. The City did not consider traffic signals at any of the intersections on Spring Mill Road, as was becoming the case citywide at the time. Eight to ten roundabouts were being constructed each year. Despite early concerns citywide, opposition to roundabouts was decreasing and the mayor was supportive of their construction. On Spring Mill Road, in particular, single-lane roundabouts offered greater capacity than signalized intersections with a similar number of lanes.

After the AWSC intersections were replaced with roundabouts, volume on Spring Mill Road increased faster than was expected. Mr. McBride believes this may be due to congestion on US 31 and the delay of a planned Indiana DOT (InDOT) improvement project that will convert US 31 to a freeway and remove at-grade intersections. In the meantime, drivers use Spring Mill Road to avoid congestion at the signalized intersections on US 31. InDOT now plans to complete the US 31 project in the mid-2010s. In the interim, the City has added lanes at some of the roundabouts on Spring Mill Road to accommodate higher-than-anticipated turning volumes. The roundabouts were constructed with 150-foot to 160-foot inscribed circle diameters (ICDs) so they could be expanded inward into double-lane roundabouts, but to date the City has only added additional lanes on some approaches.

Evaluating the Performance of Corridors with Roundabouts

The City is generally pleased with the Spring Mill Road corridor and would change relatively little if it were constructed again. Although the traffic forecasts did not accurately reflect near-term conditions, they also assumed US 31 improvements would have been in place. This assumption was consistent with the State's plans when the forecasts were created.

2.5.3.2. *Old Meridian Street*

Old Meridian Street is an old alignment of US 31 and is built on an angle across the area's grid roadway network. Prior to the construction of roundabouts, it was a two-lane roadway with a 100-foot right-of-way (ROW). This wide ROW was typical for old state-owned roadways. Most of the intersections that are now roundabouts were AWSC when the roadway was two lanes. In 1998, the State and City developed a plan to widen Old Meridian Street to a five-lane section with signalized intersections. However, Carmel had recently constructed two roundabouts on Hazel Dell Parkway, and the mayor supported constructing roundabouts on Old Meridian Street as well.

The City's long-term vision is to transform the Old Meridian Street corridor into an urban area with high-density, mixed-use development. To help facilitate this change, the City widened the roadway to four lanes and constructed four roundabouts. Federal funds were used for these improvements, which required InDOT's review and approval of the plans. InDOT preferred for the roundabouts to be relatively large to accommodate trucks. As a result, they were built with an ICD of approximately 200 feet, which is larger than the City of Carmel prefers. A signal was kept at Old Meridian Street/Carmel Drive because sufficient ROW for a roundabout was not available.

The City would make several changes to this corridor if it were constructed again, including using an offset-left design for entries and purchasing ROW so approaches were more perpendicular to one another, as well as reducing the size of the ICDs.

The roundabouts were completed in 2006. Development has occurred slowly. This is attributed, in part, to the overall slow economy nationwide. The corridor is currently lined with a mix of residential and commercial development, much of which pre-dates the roundabouts.

In the future, the following changes are foreseen along the corridor:

- Old Meridian Street/Main Street: Volumes will increase at this intersection because an interchange will be constructed at US 31/Main Street (currently a right-in, right-out configuration).
- Old Meridian Street/Grand Boulevard: A fourth leg will be added to this roundabout, and Grand Boulevard will be extended to the east. Grand Boulevard is a master-planned roadway, and this roundabout is envisioned as the center of an art and design district along Old Meridian Street.

Evaluating the Performance of Corridors with Roundabouts

- Old Meridian Street/Carmel Drive: Volumes will likely decrease at this intersection because the signal at US 31/Carmel Drive will be replaced with an overpass (without an interchange).

2.5.3.3. *Analysis Methods and Summary*

From a traffic operations perspective, the City of Carmel has generally studied roundabouts in isolation, even if multiple roundabouts were being studied on the same corridor simultaneously. Early on, the City used RODEL for operations analyses but later changed to SIDRA. The City tended to tweak early roundabout designs more than they do now and, in general, believes roundabouts should remain single-lane as long as possible because well-designed single-lane roundabouts with ICDs in the range of 150 feet to 160 feet provide greater capacity than single-lane roundabouts with poor designs or smaller ICDs.

Of the two corridors studied by the project team, the Spring Mill Road project is thought to be more similar to projects communities typically face. Old Meridian Street was somewhat of a unique project because of the extent of ROW available to the City and the coordination with InDOT.

2.5.4. BORGEN BOULEVARD – GIG HARBOR, WA

A member of the project team held a conference call with Marcos McGraw, a Project Engineer with the City of Gig Harbor, Washington. Mr. McGraw shared the history of the Borgen Boulevard corridor.

Borgen Boulevard was constructed in the 1990s by developers who wanted to create access to land in the area. The project included a new diamond interchange on SR 16, an existing expressway. An existing street—Burnham Drive—was already in place at the location where the northbound ramps were to tie into Borgen Boulevard. The City considered constructing a five-leg, signalized intersection at this location to accommodate both facilities, but eventually determined it would be expensive to construct and would operate inefficiently. The City also studied a roundabout for this intersection and determined it would be a more desirable solution than a traffic signal. As a result, the City directed the developers building Borgen Boulevard to build a roundabout at the Borgen Boulevard/SR 16 northbound ramps/Burnham Drive intersection.

Once Borgen Boulevard was built, development began to occur on adjacent land and developers began to construct new intersections. With the Borgen Boulevard/SR 16 northbound ramps/Burnham Drive roundabout in place and operating well, developers expressed a preference for roundabouts rather than traffic signals at these new intersections. The Washington State Department of Transportation (WSDOT) was also promoting the use of roundabouts at the time. For these reasons, roundabouts were added to the Borgen Boulevard corridor as additional intersections were needed.

The City's experience with the roundabouts has generally been positive. City staff members believe the roundabouts on Borgen Boulevard have fewer maintenance needs than traffic signals, and they are able to process about ten percent more traffic volume than traffic signals would. The City has had a few

Evaluating the Performance of Corridors with Roundabouts

issues with large trucks off-tracking over curbs at the 112th Street/Peacock Hill Avenue roundabout. This roundabout is smaller than others on the corridor, and was built in a location with ROW constraints.

2.5.5. SR 539 – WHATCOM COUNTY, WA

Two members of the project team held a conference call with several individuals from the WSDOT involved in the development of the SR 539 roundabout corridor. The individuals were:

- Brian Walsh, State Traffic Design and Operations Engineer
- Dina Swires, Mt. Baker Area Traffic Engineer (also a member of the 03-100 project panel)
- Dustin Terpening, Communications

WSDOT staff explained that the history of the SR 539 corridor began in 2004. This roadway is locally known as the "Guide Meridian" or simply "The Guide." SR 539 serves as a freight mobility route between I-5 and Canada.

In 2004, funds were appropriated for capital improvements to the SR 539 corridor. At the time, SR 539 was a two-lane roadway experiencing safety issues related to access points, a high percentage of truck traffic, and head-on collisions. The Whatcom County sheriff and local elected officials supported WSDOT's efforts to improve safety in the corridor. The roadway had daily traffic volumes that WSDOT considered high for a two-lane roadway, and in previous years they had considered upgrading the corridor to a freeway. There were strong feelings in WSDOT that improvements to SR 539 should include a cable median barrier on a six-mile segment of the roadway, with few breaks in the median. These desires led WSDOT staff to explore corridor and intersection treatments that would facilitate U-turns at select locations.

WSDOT staff concluded it would be desirable to create a U-turn opportunity every mile along the corridor. This provided a balance between the predominantly through traffic on the corridor and the low-volume residential and agricultural accesses. With one-mile spacing between U-turn points, a trip to or from a right-in, right-out driveway would require no more than one additional mile of trip length. WSDOT first considered jughandles to facilitate U-turns but encountered several challenges. Jughandles would have impacted property near intersections, and some would have been unsignalized. Signal warrants were not met at some intersections, including Wiser Lake Road. Furthermore, the District traffic engineer was strongly opposed to signals on high-speed roadways and encouraged project planners to find other solutions.

WSDOT staff then studied roundabouts on the corridor. Traffic operations at proposed roundabouts were analyzed with SIDRA, and traffic operations at the existing signal at SR 539/SR 544 were analyzed with SYNCHRO. Analyses indicated that two-lane roundabouts would operate acceptably on the corridor. A Paramics simulation model was used for visualization purposes, and engineers determined there would be no interaction between successive roundabouts. Staff indicated they would likely use simulation for visualization purposes today, although likely VISSIM rather than Paramics.

WSDOT staff actively communicated with stakeholders and the public throughout the project, particularly after roundabouts were selected for the corridor. WSDOT conducted strategic, in-person engagement with local elected officials, the media, local trucking companies, and the fire department. A model of a roundabout was created in a parking lot, and interested parties were able to drive through it so they might better understand how their vehicles would be accommodated. Open house meetings, a blog, and a Whatcom County newsletter were used to keep the general public informed. Mr. Terpening stated communicating with the key people in key organizations was essential to the project's success.

The roundabouts and widened roadway opened in July 2010, and WSDOT perceives the project as a great success. Nearly all respondents to an online poll are supportive of the roundabouts, whereas approximately 30 percent of those surveyed were opposed to the roundabouts prior to their construction. Mr. Terpening stated that, two years after opening, he still receives emails from the public expressing gratitude for the roundabouts. Some individuals also request that other roadways in the area be converted to roundabout corridors. The study of the corridor is ongoing, and the Insurance Institute for Highway Safety is working on a before/after study.

Travel time on the corridor has decreased. Truck drivers traveling between the US and Canada timed their trips before and after corridor improvements. Some have reported that a trip between Bellingham and Linden previously took 30 minutes, but now takes 10 to 15 minutes. Corridor residents whose travel patterns were impacted by the cable median generally remain supportive as well, and feel the capacity and safety improvements make it worth the time to replace left-turn movements with right-turn/U-turn movements.

One year before the roundabouts opened on SR 539, another four-lane section of SR 539 with traffic signals and TWSC opened between 10 Mile Road (the southernmost roundabout) and Horton Road. Planning efforts for the widening of this southern section of SR 539 began several years prior to planning efforts for the northern section with roundabouts. Some members of the public have indicated to WSDOT they prefer the section of the corridor with roundabouts over the section of the corridor with signals, and are frustrated by the need to stop at traffic signals. WSDOT was unable to change the design of the southern portion of the corridor and add roundabouts because the project was too far along by the time roundabouts were selected for the northern half of the corridor.

In conclusion, WSDOT staff stated communication was a key to the project's success. By reaching out to concerned constituents rather than ignoring them, the agency was able to build informed consent for the project. WSDOT staff members believe intersections are "traffic safety and operations decisions" and traffic engineers need to effectively communicate the benefits they offer; this was done on the SR 539 project. Finally, the combination of the cable median barrier and the roundabouts was a more effective strategy for the corridor than either would have been by itself, and WSDOT hopes to use these techniques in combination again. The District remains opposed to traffic signals on high-speed roadways; the public is pleased with the performance of SR 539 and supportive of roundabouts on high-speed roadways. WSDOT is currently planning another roundabout corridor nearby on SR 20.

WSDOT offered information on several other lessons learned from the project:

- The contractor constructed 4-inch rolled curbs rather than 6-inch rolled curbs that were typical at the time. However, based upon positive feedback from the trucking industry, 4-inch rolled curbs have become the new state standard for roundabouts.
- Signs were added after opening, instructing drivers not to travel beside trucks in the roundabout. These have also been used elsewhere across the state following their initial use on SR 539.
- WSDOT is generally pleased with the design of the roundabouts. However, if they had anticipated roundabouts prior to awarding project contracts, they would have sought out a consultant with roundabout experience.
- WSDOT did not plan for oversize/overweight trucks on this corridor but would do so today based on new policies.
- The communications plan for this project was so successful that it is now used as a model for other projects.

2.5.6. GOLDEN ROAD – GOLDEN, CO

A member of the project team held a conference call with Dan Hartman, Public Works Director with the City of Golden. Mr. Hartman discussed the history of the Golden Road corridor. Additionally, the team reviewed several papers and presentations on the corridor presented by Mr. Hartman and others at conferences.

Downtown Golden, including the Coors Brewery and the Colorado School of Mines, lies to the northwest of the corridor. I-70 and Denver lie to the southeast. Prior to the roundabouts, Golden Road was a five-lane section with two travel lanes in each direction, a two-way left-turn lane, and, in some locations, paved shoulders or right-turn lanes. The paved cross section was approximately 84 feet. The corridor was lined with suburban development such as fast-food restaurants and strip shopping centers, and there were numerous access points. Operating speed was approximately 45 miles per hour.

The impetus for corridor improvements began in the mid-to-late 1990s, with a proposal for a development anchored by a grocery store towards the northwestern end of the corridor. Residents of streets intersecting the corridor were already experiencing delay when turning left and they were concerned this delay would increase as a result of additional traffic from the development. Volumes at these intersections did not meet signal warrants.

A consultant for the City conducted traffic modeling of two scenarios on the corridor: one with traffic signal improvements and one with roundabouts. Both performed well operationally and were viable. Most attendees of public hearings on the project were skeptical of the roundabouts, but the mayor and city council were supportive.

Initial plans for the corridor called for three roundabouts, but a fourth roundabout was added at Golden Road/Utah Street at the request of several businesses on the corridor, including a fast-food restaurant. Under pre-roundabout conditions, queues formed in the parking lot of the restaurant due to the high delay of the left-turn movement out of the parking lot. The restaurant thought it would be preferable to prohibit left turns out of the parking lot and instead have customers turn right onto Golden Road and then make a U-turn at the Golden Road/Utah Street roundabout.

The first four roundabouts on the Golden Road corridor opened in 1998 and 1999. Five years later, a high school northwest of the four roundabouts was reconstructed and the access road intersection was reconfigured. At a public meeting, 70 percent of attendees preferred a roundabout over a traffic signal. A fifth roundabout was added to the corridor at this location.

The City believes the corridor has been a success. There has been a 67 percent reduction in accidents. The corridor experienced approximately ten injury accidents per year prior to the roundabouts, and it experienced two injury accidents in the ten years following construction of the roundabouts. The 85th-percentile speed on the corridor was reduced to 26 miles per hour. New businesses invested approximately $7 million in real estate along the corridor, and existing businesses invested approximately $7 million as well.

In the future, when the corridor is repaved, the City will likely replace the asphalt in the circulatory roadway with concrete. The asphalt had become rutted and required occasional maintenance. The City would change little about the corridor if constructed again. Some have noted that the mid-corridor roundabouts at Utah Street and Lunnonhaus Drive have minimal deflection on Golden Road and an ICD of 105 feet, which is considered small for a multilane roundabout. However, there were ROW limitations when the roundabouts were constructed, and there have been no operational or safety performance concerns; consequently, the City would not change the design of these roundabouts.

2.5.7. AVON ROAD – AVON, CO

Justin Hildreth, Town Engineer for the City of Avon, participated in a call with a member of the project team. He provided information on operating conditions and the changes that have taken place on the Avon Road corridor since it opened. Additionally, the project team reviewed an *ITE Journal* article on the corridor (Ourston & Hall 1997).

The Avon Road roundabouts opened in 1997, several years after the roundabouts at the nearby Vail interchange. Three intersections on Avon Road south of I-70 were previously controlled with traffic signals. The project was funded with public funds and a contribution from a local ski resort. Since then, roundabout

Evaluating the Performance of Corridors with Roundabouts

corridors have also been built nearby on William J. Post Boulevard and Edwards Access Road, and a corridor is planned nearby in Eagle, Colorado. All of these corridors include interchanges on I-70. Post Boulevard, which opened in 2002, created a new interchange on I-70. This additional link between the Interstate and Avon reduced traffic volume on Avon Road.

In 2005 or 2006, the City removed many signs associated with roundabouts from the corridor. City staff believed there was "sign clutter" on the corridor; i.e., the high number of signs was not helpful to drivers.

In 2007, the Avon Road/Benchmark Road roundabout was converted from a "teardrop" design, with no circulatory roadway in front of the south leg, to a conventional roundabout design with a complete circulatory roadway. To the south, Avon Road slopes down from the roundabout and passes under a railroad line. The original designers chose a teardrop design due to safety concerns related to the cross slope of the circulatory roadway in front of the south leg. Over time, the teardrop design created circulation challenges on the corridor, which was the impetus for its reconstruction. There have been no reported safety issues since the modification.

The City added pavement markings to the circulatory roadways in accordance with the 2009 *Manual on Uniform Traffic Control Devices* (MUTCD). Prior to this, there were no pavement markings in the circulatory roadway. Mr. Hildreth believes the pavement markings are not obeyed by drivers, and MUTCD may recommend more marking than is optimal.

The corridor currently performs well operationally and congestion only occurs during snow storms or following a crash. The City has some concerns regarding pedestrian safety on the corridor. In the spring of 2012, a police survey of the corridor found most drivers yielded to pedestrians, but in some cases it was difficult for drivers to see pedestrians and crossing areas. The City planned to improve sight distance in these areas later in 2012.

When funding becomes available, the City plans to reduce the Beaver Creek Boulevard entries to the Avon Road/Beaver Creek Boulevard roundabout from three lanes to two lanes. The City performed a future conditions traffic analysis, including all planned development for the area, and determined two-lane entries would sufficiently serve future capacity needs. One of the goals of the lane removal is to improve pedestrian comfort.

If the City were to design the corridor today, it would make several changes:

- Construct full roundabouts at the I-70 interchange.
- Potentially decrease the capacity of the corridor; the opening of Post Boulevard has decreased volumes on Avon Road.
- Use fewer pavement markings in the circulatory roadways.

2.5.8. SR 67 – MALTA, NY

One member of the project team held a conference call with several individuals from the New York State Department of Transportation (NYSDOT) involved in

Evaluating the Performance of Corridors with Roundabouts

the development and/or current operation of the SR 67 roundabout corridor. The individuals were:

- Mark Kennedy, Regional Traffic Engineer, NYSDOT Region 1
- James Boni, Assistant to the Regional Director, NYSDOT Region 1
- Howard McCulloch, Statewide Roundabout Design Specialist, NYSDOT

NYSDOT staff shared the history of the corridor and changes that have taken place since it opened.

The SR 67 corridor improvements began as a bridge replacement project. The SR 67 bridge over I-87, the Adirondack Northway, needed to be replaced for structural reasons. At the time, the bridge was three lanes wide. Ramp terminal intersections and three other intersections on SR 67 were signalized.

As part of the bridge replacement project, NYSDOT performed a traffic study for conditions 20 years into the future, which is typical for NYSDOT projects involving an interchange. Based upon traffic forecasts, an in-house NYSDOT group developed a concept for a single-point urban interchange (SPUI) at I-87/SR 67. However, NYSDOT determined the SPUI concept was too expensive and would create queue spillback with the signal at SR 67/Kelch Drive.

At the request of the NYSDOT Region 1 Director, the State developed a roundabout concept for the corridor. The project expanded in scope from two roundabouts (at the interchange) to five roundabouts:

- SR 67/State Farm Boulevard
- SR 67/I-87 southbound ramps
- SR 67/I-87 northbound ramps
- SR 67/Kelch Drive
- SR 67/US 9

NYSDOT added the additional roundabouts to the project to serve forecasted demand and reduce the likelihood of queue spillback into the interchange roundabouts.

When roundabouts were proposed for SR 67, there were no roundabouts in NYSDOT Region 1, and regional traffic engineering staff had concerns about adding the first five roundabouts in the region simultaneously on one corridor. Additionally, the traffic engineers were concerned about the operation of roundabouts in a series. At the time, NYSDOT used RODEL to analyze roundabouts, and results showed no queue spillback. Although NYSDOT believed this sufficiently addressed concerns of roundabouts in a series, NYSDOT also agreed to analyze the corridor with a simulation model and directed a consultant to do so. Paramics software was selected for simulation modeling because it modeled roundabouts in a way that was more similar to RODEL than other microsimulation models such as VISSIM. The simulation did not identify any queue spillback into the adjacent roundabouts, and it was used at public meetings for visualization purposes.

The mayor of Malta and the public were generally supportive of the concept for roundabouts on SR 67, and Region 1 staff chose to move forward with the roundabout corridor plan. The first five roundabouts opened in 2006-2007. Since then, two roundabouts have been added to the east:

- Dunning Street/Partridge Drive
- Dunning Street/Hermes Road/Plains Road

Additionally, there are now 14 roundabouts on other roadways within two and a half miles of the SR 67 corridor.

Since the corridor opened, the SR 67/US 9 roundabout has experienced a high crash frequency. Many of the crashes are by drivers 30 to 50 years of age and familiar with the corridor. Many crashes are related to a failure to yield by entering drivers or conflicts on exits. NYSDOT believes the fastest-path speeds are too high, in part because the roundabout was designed to accommodate side-by-side trucks. NYSDOT has made several improvements to address the crashes at this roundabout:

- On SR 67, exits were reduced to one lane with pavement markings, and the left lane of SR 67 entries was changed from through-left to left-only.
- Transverse pavement markings were added on the US 9 approaches, but they have not reduced speeds. It was noted that US 9 is designed as a high-speed roadway because it was the main north-south roadway in the area prior to I-87. Many speed-related crashes are on the US 9 approaches. The design of SR 67, including curbs and landscaping, is more effective at reducing speeds.

NYSDOT noted the roundabouts at the I-87 interchange have high-speed features such as overhead signs and right-turn bypasses with dedicated receiving lanes. The roundabout exits on SR 67 taper to one lane to receive the right-turn lane from the I-87 off-ramps. The public has noted this area is challenging for pedestrians.

If constructing the corridor again, NYSDOT would make few changes from a corridor perspective. In terms of individual roundabouts, the following changes would be made:

- Lower approach speeds;
- Raise the elevation of the SR 67/Kelch Drive roundabout; and
- Keep roundabouts single-lane when possible, and use nearer-term design years (single-lane roundabouts are more effective at calming traffic than double-lane roundabouts).

In closing, it was noted that it would be useful to have research on driver wayfinding in roundabout corridors. Some drivers report "getting lost" on SR 67 and missing a turn because they were unsure at which roundabout they were located. NYSDOT is considering adding sculptures or other unique decorations to the roundabouts to assist with wayfinding.

Evaluating the Performance of Corridors with Roundabouts

2.6. CONCLUSION

The research team prepared a CCD to aid practitioners with objective and comprehensive comparisons of corridor alternatives. To investigate traffic operations on roundabout corridors, the research team identified 58 roundabout corridors in the United States. Nine corridors, representing a diversity of conditions, were selected for data collection. After two "pilot" data collection trips, the research team modified the data collection procedure based on lessons learned. The revised procedure was employed at the remaining seven data collection corridors.

The research team also interviewed the owners of the nine study corridors. Several themes emerged:

- Once several roundabouts are built on a corridor, new or upgraded intersections are more likely to be roundabouts than signalized intersections. Reasons for this include good performance of the roundabouts in place, increased public and agency awareness and acceptance of roundabouts, concerns about queue spillback from signals into roundabouts, access management, and consistency within the corridor.
- Traffic analysis of roundabout corridors prior to their construction was typically conducted by analyzing each roundabout in isolation. However, several corridors were analyzed with microsimulation. It is anticipated the predictive tools for operational performance developed in this project, combined with the new tools in the HCM 2010, will provide practitioners with a simpler alternative to microsimulation.
- The safety effect of each corridor was not studied in detail in this project. However, owners of two roundabout corridors constructed as retrofits stated crash frequency decreased on the corridor following the construction of roundabouts. The consistent safety findings reported elsewhere suggest this trend is likely to continue in a corridor context.
- An agency champion was often the key to a corridor being constructed with roundabouts.

The CCD developed for this project appears to be flexible and adaptable to conditions similar to those described in the interviews.

Evaluating the Performance of Corridors with Roundabouts

CHAPTER 3. MODELING

This chapter describes the modeling framework and results for evaluating roundabout corridors in a *Highway Capacity Manual* (HCM) context. The chapter begins with a description of the modeling framework, including details on the sub-models used in the HCM framework. The chapter then presents modeling results for each sub-model, including details on the statistical model development and a discussion of implementation in the HCM. Last, comparisons to equivalent non-roundabout (signalized or stop-controlled) corridors are presented.

3.1. BACKGROUND

The objective of this chapter is to enhance the Urban Streets methodology in the HCM 2010 to integrate the evaluation of one or more roundabouts along an urban street. The methodology as currently described in Chapter 17 of the HCM 2010 is primarily geared at evaluating the performance of an urban street with a signalized boundary intersection. The chapter does currently allow for the analysis of roundabouts in an urban street, but is limited in that regard to the capabilities of the roundabout node methodology described in HCM 2010 Chapter 21. That method is primarily used to estimate the average control delay and 95th-percentile queues at roundabout approaches. However, the method does not currently allow for the estimation of segment-specific attributes in roundabout corridors, including midsegment free-flow speed or the extent of the roundabout influence area. This chapter presents a framework and models to fill that gap.

An urban street segment in the HCM 2010 context is defined as a stretch of roadway between two intersections, including the downstream boundary intersection. In other words, the total delay of an urban street segment combines the control delay at the intersection with any midsegment delays resulting from queuing, driveway friction, or simply high vehicular volumes. The level of service (LOS) of an urban street segment is defined by the metric "Percent Free-Flow Speed" (%FFS), which is calculated by dividing the average segment speed by the segment free-flow speed.

Several computational steps are necessary in the urban street chapter to arrive at the %FFS measure, which are replicated here for the case of roundabouts. These computational steps include the following:

- Step 1: Determine Traffic Demand Adjustments
- Step 2: Determine Running Time
- Step 3: Determine Proportion Arriving During Green
- Step 4: Determine Signal Phase Duration
- Step 5: Determine Through Delay
- Step 6: Determine Through Stop Rate
- Step 7: Determine Travel Speed
- Step 8: Determine Spatial Stop Rate
- Step 9: Determine LOS

Evaluating the Performance of Corridors with Roundabouts

- Step 10: Determine Automobile Perception Score

For the application to roundabout corridors, Step 1 is maintained. The Step 2 procedure for average running time is generally maintained, but certain components of that step, like the free-flow speed estimation procedure, are updated for roundabout corridor operations. Steps 3 and 4 are not applicable to roundabouts. Step 5 is replaced with a roundabout-specific delay estimation procedure. Step 6 remains as a gap in the literature, where no model for stop rates at roundabouts is available. Because this performance measure is not used in the determination of LOS for the urban street segment, it was not a focus in this research. Step 7 is maintained from Chapter 17 and uses earlier roundabout-specific interim steps. Step 8 is again a gap in the methodology for roundabouts, as no stop rate–prediction procedure is available. Step 9 is maintained for roundabouts to estimate LOS. Step 10 represents another gap in the literature, as all studies to arrive at the automobile perception score were conducted at signalized intersections.

The focus of this effort is on the calibration of Steps 2 and 5, while largely maintaining Steps 1, 7, and 9 to arrive at an urban street segment LOS for roundabout segments. Steps 3, 4, 6, 8, and 10 are either not applicable to roundabouts or do not have a roundabout-specific model available.

An example application of the methodology to the Old Meridian Road corridor in Carmel, Indiana, is presented at the end of the chapter to illustrate the computational steps. That particular corridor has four roundabouts and one signalized intersection.

3.2. MODELING FRAMEWORK

The objective of the analytical framework is to develop sub-models that can be used to estimate the performance of roundabout corridors. As a guiding principle, this framework is intended to be compatible with HCM 2010 methodologies for urban street segments (Chapter 17) and roundabouts (Chapter 21).

The team developed four different sub-models to characterize various aspects of corridor performance:

1. A <u>Roundabout Influence Area (RIA) Model</u> to estimate the spatial extents of the impact on speed of the roundabout node on the upstream and downstream urban street segments (HCM Chapter 17). This is especially important in the case of closely-spaced roundabouts that may have overlapping influence areas.
2. A <u>Geometric Delay Model</u> for travel through the roundabout node, measuring that which is incurred by *unimpeded* drivers. This model is a potential addition to HCM Chapter 21, which currently does not include geometric delay in the methodology.
3. A <u>Free-Flow Speed Prediction Model</u> for midsegment areas between two roundabout nodes under consideration of speed limit, intersection spacing, and other geometric attributes. This model is needed to

Evaluating the Performance of Corridors with Roundabouts

estimate free-flow speed in the context of HCM Chapter 17, but is also needed as an input in the geometric delay model described above.

4. An Impeded Delay Model for the roundabout node, which is an exercise to verify the existing control delay model included in HCM Chapter 21 and to include any additional delay related to the roundabout node being part of a corridor.

In addition, the team explored an Average Travel-Speed Model for the segment between two roundabouts for *prevailing traffic conditions*, which includes interaction between vehicles in heavier traffic flow conditions.

The first three models are considered *site-level* models because they are not sensitive to time-of-day variability in traffic patterns (i.e., volumes). In other words, the RIA, geometric delay, and FFS models are considered to be fixed over time for a specific roundabout approach. On the other hand, the Impeded Delay Model and Average Travel Speed Model are *operational-level* models that take traffic volumes and volume-to-capacity ratios into consideration. Conceptually, these latter two models may use one or more of the site-level models as inputs, where, for example, the average travel speed may be a function of the free-flow speed and roundabout influence area estimated in earlier models. In Chapter 17 of the HCM 2010, the average travel speed is estimated as a function of the segment running speed and the various sources of delay. The formulation of a separate travel-speed model here is based on a desire to verify the applicability of the Chapter 17 method to roundabout corridors.

Each model is described in more detail in the following sections.

3.2.1. ROUNDABOUT INFLUENCE AREA MODEL

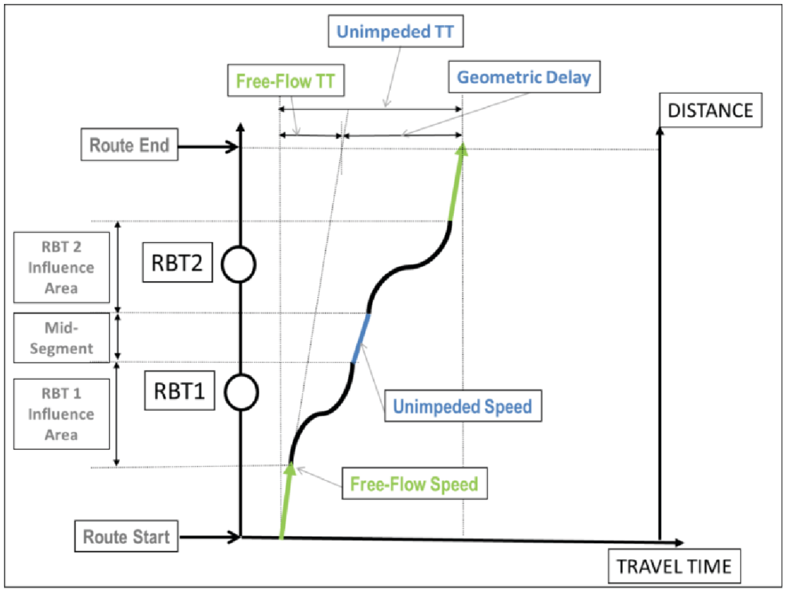
This model is used to estimate the length of the roundabout influence area (RIA). The concept of RIA assumes the roundabout corridor has a free-flow speed (FFS) corresponding to the speed drivers would travel without the presence of the roundabout or other impedances such as other vehicles. With the geometric influence of the roundabout, all drivers then have to decelerate from that FFS to traverse the roundabout. This reduced speed is referred to as *unimpeded speed* because it is constrained by geometric effects only, without any interaction or impedance from other vehicles. The speed profile of this unimpeded speed relative to the free-flow speed defines the length of the RIA. By definition, the beginning of the RIA is the point where the unimpeded speed trajectory begins to drop below the free-flow speed. The end of the RIA is the point where the unimpeded speed recovers to the free-flow speed downstream of the roundabout. Exhibit 3-1 illustrates the RIA for two adjacent roundabouts.

Exhibit 3-1 assumes a midsegment distance between the two roundabouts where vehicles travel at an unimpeded midsegment speed. However, this may not always be the case, and the team identified cases of overlapping RIAs for several of the studied corridors with closely-spaced roundabouts. The primary objective of the RIA model is to estimate whether the RIAs of two adjacent roundabouts overlap.

Evaluating the Performance of Corridors with Roundabouts

The existence of overlapping RIAs does not constitute a design flaw of the corridor. The studied roundabouts with overlapping RIAs appeared to perform normally. The overlapping RIA is merely a design aspect impacting modeling of operational performance.

Exhibit 3-1: Concept of Roundabout Influence Area and Geometric Delay (TT = travel time, RBT = roundabout)



3.2.2. GEOMETRIC DELAY MODEL

This model predicts the geometric delay through the roundabout node. It includes the deceleration and acceleration delays of an *unimpeded* vehicle traveling through the roundabout. The geometric delay is defined spatially across the RIA, which encompasses the roundabout node and any upstream and downstream distance needed for deceleration from and acceleration to free-flow speed through the corridor. The geometric delay model is independent of the vehicular volume on the corridor or at the node. A geometric delay model for roundabouts exists from research performed in the UK (Kimber 1980), as well as in Australian analysis guidance (Austroads 1993). The objective of this effort, however, was to derive an empirical model based on US roundabouts and driving conditions.

The total geometric delay for a corridor is calculated as the sum of individual node geometric delays and any midsegment geometric delays. For closely-spaced roundabouts, it is possible the RIAs of two roundabouts overlap. For larger spacing, a midsegment unimpeded speed is estimated to calculate travel times. If that midsegment unimpeded speed is equal to the roadway free-flow speed, no midsegment geometric delay is incurred. If the unimpeded speed is less than the free-flow speed, midsegment geometric delay is the product of segment length and the difference of the unimpeded trip time rate and the free-

Evaluating the Performance of Corridors with Roundabouts

flow trip time rate:

Equation 3-1

$$d_{geom} = L\left[\frac{1}{S_u} - \frac{1}{S_f}\right]$$

where

d_{geom} = geometric delay (seconds);

L = segment length (feet);

S_u = unimpeded speed (feet/second); and

S_f = free-flow speed (feet/second).

The terms used in this discussion were shown visually in Exhibit 3-1. In the HCM, geometric delay is currently not calculated explicitly in HCM Chapter 21, although other approaches exist in literature to verify the field-based measurements in this project.

For Urban Streets, Equation 17-6 of the HCM 2010 predicts the segment running time for the auto mode, and the delay associated with acceleration and deceleration is included in this equation. The equation is principally used for signalized intersection delay, but also includes a term to accommodate yield-controlled movements. Equation 17-6 is replicated below for reference.

Equation 3-2

$$t_R = \frac{6.0 - l_1}{0.0025\ L} f_x + \frac{3{,}600\ L}{5{,}280\ S_f} f_v + \sum_{i=1}^{N_{ap}} d_{ap,i} + d_{\text{other}}$$

Source: HCM 2010 Equation 17-6

with,

$$f_x = \begin{cases} 1.00 & \text{(signalized or STOP-controlled through movement)} \\ 0.00 & \text{(uncontrolled through movement)} \\ \min[v_{th}/c_{th}, 1.00] & \text{(YIELD-controlled through movement)} \end{cases}$$

where

t_R = segment running time (s);

l_1 = start-up lost time = 2.0 if signalized, 2.5 if STOP- or YIELD-controlled (s);

L = segment length (ft);

f_x = control-type adjustment factor;

v_{th} = through demand flow rate (veh/h);

c_{th} = through-movement capacity (veh/h);

$d_{ap,i}$ = delay due to left and right turns from the street into access point

Evaluating the Performance of Corridors with Roundabouts

intersection i (s/veh);

N_{ap} = number of influential access point approaches along the segment = $N_{ap,s} + p_{ap,lt} N_{ap,o}$ (points);

$N_{ap,s}$ = number of access point approaches on the right side in the subject direction of travel (points);

$N_{ap,o}$ = number of access point approaches on the right side in the opposing direction of travel (points);

$p_{ap,lt}$ = proportion of $N_{ap,o}$ that can be accessed by a left-turn from the subject direction of travel; and

d_{other} = delay due to other sources along the segment (e.g., curb parking, pedestrians, etc.) (s/veh).

The HCM Equation 17-6 does not include geometric delay within the roundabout. This geometric delay includes a difference in travel distance (i.e., the difference between traveling in the circulating lane versus travel along the center-line distance for a signalized intersection), and a difference in travel speed (i.e., deceleration from free-flow speed, travel at the geometrically-constrained circulating speed, and acceleration to free-flow speed).

The geometric delay model was derived from field data collected in this project. The explanatory variables included in the model are free-flow speed, circulating speed, and inscribed circle diameter. Other explanatory variables explored, but ultimately not incorporated into the geometric delay model, include central island diameter, lane width, median type, and other geometric components.

3.2.3. FREE-FLOW SPEED PREDICTION MODEL

Both the RIA and geometric delay models incorporate the concept of midsegment free-flow speed (FFS) between roundabouts. While the first two models use the field-measured FFS directly, the objective of this model is to predict FFS as a function of the geometry of the segment characteristics between two roundabouts. This enables use of the first two models without field measurement of FFS.

An equivalent for this approach exists in HCM Chapter 31 (for Urban Street Segments with signalized intersections), where a relationship is given between the base FFS and the *prevailing* FFS on the segment. The latter is generally lower than the (theoretical) base FFS, especially for closely-spaced intersections.

In this project, a similar FFS prediction model was developed to estimate the prevailing FFS between two roundabouts. The explanatory variables included in the model are segment length, speed limit, central island diameter, and a dummy variable for the presence/absence of overlapping RIAs. FFS computed

Evaluating the Performance of Corridors with Roundabouts

with this model will be a required input for the geometric delay model and the average travel speed model.

3.2.4. IMPEDED DELAY MODEL

This fourth model predicts the impeded delay at the roundabout, which is caused by interaction with other vehicles. The control delay methodology in HCM Chapter 21 provides an existing method for estimating the control delay for vehicles entering a roundabout. However, that methodology is limited to the upstream segments, and is therefore not applicable to the downstream segments in a roundabout corridor. For this project, the team estimated the impeded delay from prevailing conditions for both upstream and downstream segments, which includes the node control delay, as well as potential midsegment delays resulting from high vehicular flow rates.

The key difference between this and the three prior models is that the RIA, geometric delay, and FFS are all based on unimpeded trajectories. In other words, they assume the scenario of a single vehicle traversing the corridor, and the data used to develop them were collected during low-volume time periods. The impeded delay model now takes into consideration the interaction with other vehicles on the corridor. As traffic volumes increase, it is expected that the additional delay is incurred from the yield-control operations at the roundabout entry and midsegment friction effects from prevailing traffic conditions.

In particular, the control delay equation used in the roundabouts method (borrowed from two-way stop-controlled intersections) is, in its unadjusted form, based on the incremental delay term for signalized intersections (d_2) shown in Equation 3-3. This equation is sensitive to the passage time setting at the signal controller (PT) as part of the incremental delay factor (k), and an upstream filtering adjustment factor (I) as described below. The passage time directly relates to the headways between vehicles, making it sensitive to arrival patterns.

Equation 3-3

$$d_2 = 900\,T\left[(X_A - 1) + \sqrt{(X_A - 1)^2 + \frac{8kIX_A}{c_A T}}\right]$$

Source: HCM 2010 Equation 18-45

where

d_2 = incremental delay for signalized intersections (seconds);

T = analysis period duration (seconds);

X_A = average volume-to-capacity ratio;

k = incremental delay factor:

$$k = (1 - 2k_{min})(v/c_A - 0.5) + k_{min} \le 0.50$$

$$k_{min} = -0.375 + 0.354\,PT - 0.0910\,PT^2 + 0.00889\,PT^3 \ge 0.04$$

Evaluating the Performance of Corridors with Roundabouts

PT = passage time setting on signal controller (seconds);

I = upstream filtering adjustment factor:

$$I = 1.0 - 0.91\, X_u^{2.68} \geq 0.090$$

X_u = weighted v/c ratio for all upstream movements contributing to the volume in the subject movement group; and

c_A = available capacity (veh/h).

In the application to roundabouts, the equation above has been modified in HCM Chapter 21 to add an estimate of service time (3600/c), add a base delay at the yield line (5*min[X_A;1]), and set the incremental delay factor (k) and the upstream filtering adjustment factor (I) to a default value of 1.0. The equation simplifies to the following:

Equation 3-4

$$d = \frac{3{,}600}{c} + 900\,T\left[x - 1 + \sqrt{(x-1)^2 + \frac{\left(\frac{3{,}600}{c}\right)x}{450\,T}}\right] + 5 \times \min[x, 1]$$

Source: HCM 2010 Equation 21-17

where

d = average control delay (s/veh);

x = volume-to-capacity ratio of the subject lane;

c = capacity of subject lane (veh/h); and

T = time period (h) (T = 0.25 for a 15-min analysis).

For roundabouts in a corridor, it is likely the variability of arrivals and the proportion of platooned vehicles (in green) are not random, and that further adjustments to the delay equation need to be made. A simple approach to do this when developing a model is to estimate the headway distribution of arrivals and to use that proportion to calibrate the k and I factors in the delay model based on field data (presumably a combined calibration factor, α, can be defined as α=k*I). Alternatively, the focus may be on the calibration of the upstream filtering adjustment factor, I, only. This factor describes the variation in arrivals during the analysis period, and HCM Chapter 18 includes an equation that can be used if the intersection upstream of the roundabout is signalized (HCM Equation 18-3).

The team explored these various options for adapting Equation 3-4 above with consideration of these calibration factors, but while maintaining the roundabout-specific estimates of minimum service time (3600/c) and base delay at the yield line (5*min[X_A;1]). As an alternative, for estimating the total impeded delay, both the roundabout node control delay and any additional delays incurred over a segment from increases in volumes may be combined into one empirical model. Exhibit 3-2 shows the updated delay figure under consideration of control delay and other impediments to vehicles such as midsegment delay. The total impeded delay over a segment represents the sum of these two terms.

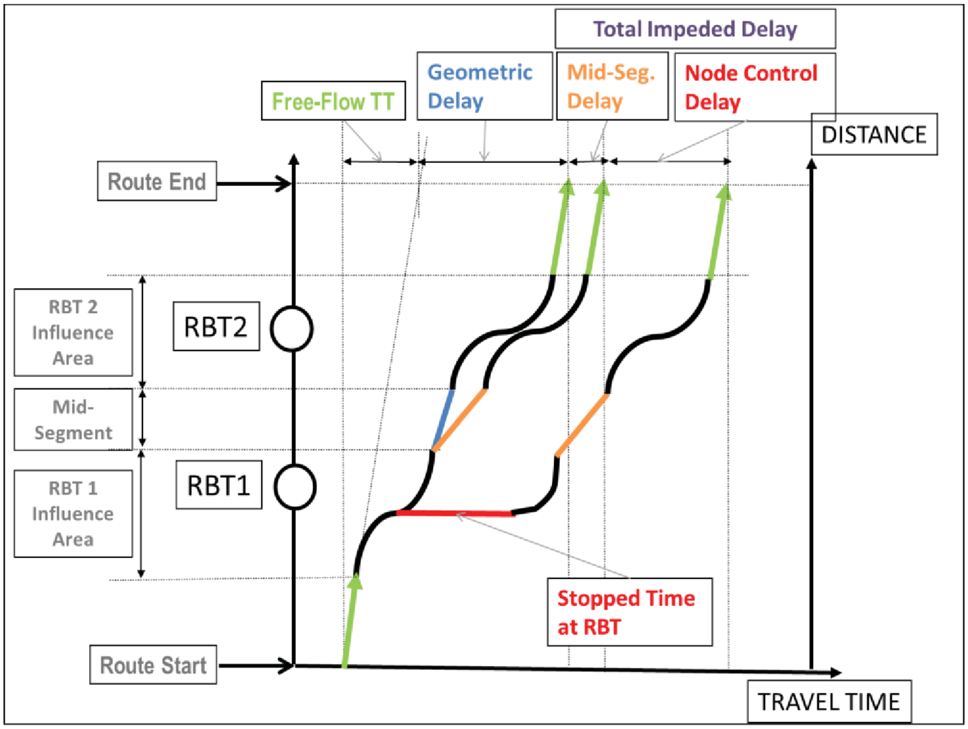


Exhibit 3-2: Impact of Control Delay on Corridor Trajectories

The team ultimately decided to derive overall empirical impeded delay models that distinguish between upstream and downstream segments. These models will then be compared to the control delay models in HCM 2010 Chapter 21 for validation.

3.2.5. AVERAGE TRAVEL-SPEED MODEL

This model predicts the average travel speed on a segment between two roundabouts. In Chapter 17 of the HCM 2010, the average travel speed is estimated as a function of the segment running speed and the various sources of delay. The formulation of a separate travel-speed model here is based on a desire to verify the applicability of the Chapter 17 method to roundabout corridors.

HCM 2010 Chapter 17 predicts average travel speed as a function of various geometric and operational variables, with a full listing of the input data shown in Exhibit 3-3 (based on HCM Exhibit 17-5). In this project, the base methodology for average travel speed is maintained from the HCM 2010, but various sub-models are replaced with roundabout-specific models developed here. In addition, a stand-alone average travel speed model is provided for comparison and validation of the HCM 2010 method.

Exhibit 3-3: Input Requirement for Urban Street Segments in HCM

Data Category	Location	Input Data Element	Basis
Traffic characteristics	Boundary intersection	Demand flow rate	Movement group
	Segment	Access point flow rate	Movement group
		Midsegment flow rate	Segment
Geometric design	Boundary intersection	Number of lanes	Movement group
		Upstream intersection width	Intersection
		Turn bay length	Segment approach
	Segment	Number of through lanes	Segment
		Number of lanes at access points	Segment approach
		Turn bay length at access points	Segment approach
		Segment length	Segment
		Restrictive median length	Segment
		Proportion of segment with curb	Segment
		Number of access point approaches	Segment
Other	Segment	Analysis period duration	Segment
		Speed limit	Segment
Performance measures	Boundary intersection	Through control delay	Through-movement group
		Through stopped vehicles	Through-movement group
		2nd- and 3rd-term back-of-queue size	Through-movement group
		Capacity	Movement group
	Segment	Midsegment delay	Segment
		Midsegment stops	Segment

Notes: Movement group = one value for each turn movement with exclusive lanes and one value for the through movement (inclusive of any turn movements in a shared lane).

Through-movement group = one value for the segment through movement at the downstream boundary intersection (inclusive of any turn movements in a shared lane).

Segment = one value or condition for each direction of travel on the segment.

Segment approach = one value or condition for each intersection approach on the subject segment.

Source: HCM 2010 Exhibit 17-5.

3.2.6. SEGMENT AND VARIABLE DEFINITIONS

Before proceeding to the individual modeling results, this section offers key definitions of analysis segments, as well as the dependent and independent variables used in model development.

3.2.6.1. *Segment Definitions*

In the evaluation of roundabout corridors in an HCM context, some key challenges emerge related to definitions of analysis segments. In HCM Chapter 17, an urban street segment is defined spatially as extending from the stop-bar of an upstream (signalized) intersection to the stop-bar of the downstream intersection. For two adjacent roundabouts, the corresponding urban street segment definition would, therefore, be defined from the upstream yield line to the downstream yield line. However, in applying HCM Chapter 21, the spatial extent of the roundabout node for the purpose of estimating geometric delay would arguably include portions of the upstream and downstream segments.

Evaluating the Performance of Corridors with Roundabouts

Assuming the roundabout analysis segment extends roughly from the midsegment point of the upstream link to the midsegment point of the downstream link, the result is that the HCM Chapter 17 segment is shifted approximately one-half block in length from the HCM Chapter 21 segment.

For analysis in this research, the team elected to use the lowest common denominator for segment definitions by applying all analysis to a segment equal to half of a link. In other words, each HCM Chapter 17 urban street segment is divided into two components at the midsegment point of the link between the two roundabouts.

Exhibit 3-4 illustrates this approach to segment definition. The exhibit shows two roundabouts (RBT1 and RBT2) separated by (Urban Street) Segment B. For the purpose of analysis, Segment B is divided into sub-segments B1 and B2, where B1 corresponds to the downstream influence of RBT1, and B2 corresponds to the upstream influence of RBT2. The upstream sub-segment of RBT1 is consequently labeled A1 (portion of Segment A associated with RBT1), and the downstream sub-segment of RBT is labeled C2 (portion of Segment C associated with RBT2).

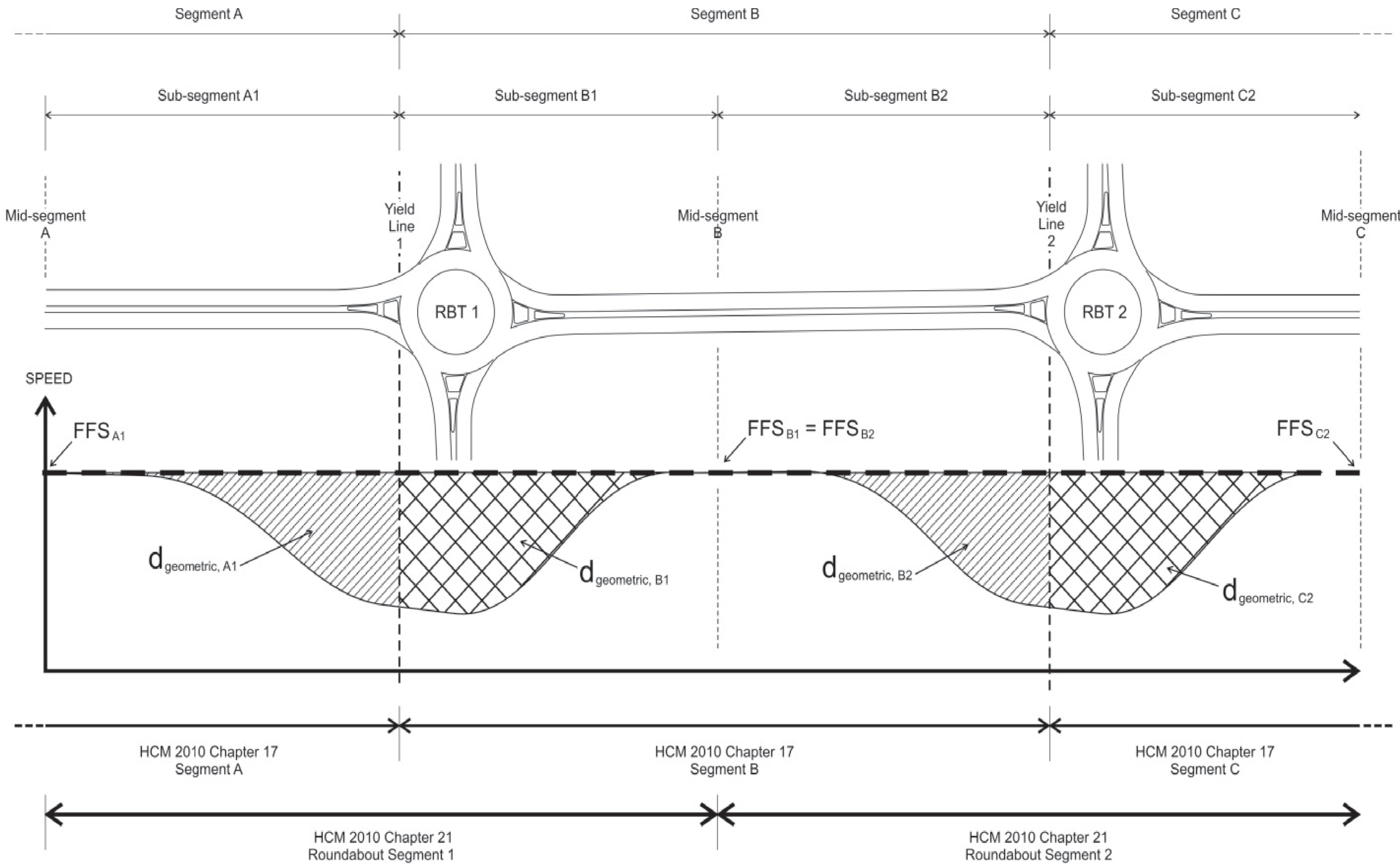
Conceptually, an *urban street segment* (HCM Chapter 17) is defined as the sum of the downstream and upstream sub-segments of two adjacent roundabouts (e.g. B1 plus B2). Similarly, a *roundabout segment* (HCM Chapter 21) is defined as the sum of the upstream and downstream segment of the same roundabout (e.g., A1 plus B1).

In the model development, the upstream and downstream sub-segments are evaluated separately for two primary reasons: (1) this allows aggregation of model results from both HCM Chapters 17 and 21, and (2) the operational effects of the two are hypothesized to be different.

To illustrate the latter point, Exhibit 3-4 shows as shaded areas the theoretical areas of geometric delay for upstream and downstream segments. Geometric delay in this case is defined as the difference between segment free-flow speed and the unimpeded trajectory speed across the sub-segment distance. The exhibit makes evident that geometric delay for the upstream roundabout sub-segment (gray lines) is arguably much less than for the downstream sub-segment (black crosshatch marks), as the latter includes significant travel at the geometrically-constrained circulating speed.

Consistent with these sub-segment definitions, all variables are defined on a sub-segment basis. This includes variables such as the free-flow speed (FFS), with each roundabout having a potentially different upstream (A1) and downstream (B1) free-flow speed. Coincidentally, the downstream FFS of RBT1 (B1) is the same as the upstream FFS of RBT2 (B2).

Exhibit 3-4: Segment Definitions for Modeling Framework



Evaluating the Performance of Corridors with Roundabouts

Evaluating the Performance of Corridors with Roundabouts

3.2.6.2. *Variable Definitions*

Based on the segment definitions above, the following variables were extracted for each upstream and downstream sub-segment (Exhibit 3-5):

Exhibit 3-5: Variable Definitions

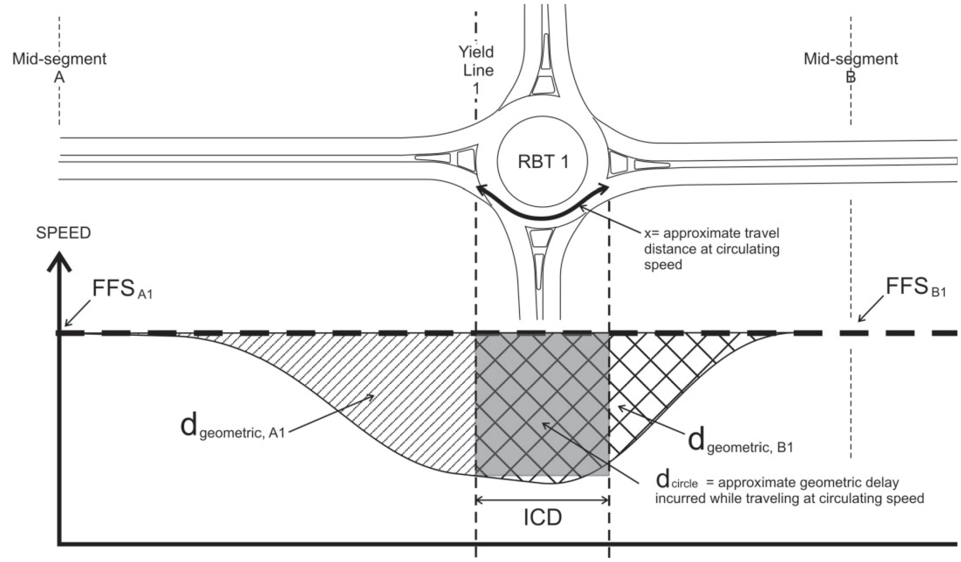
Variable Name	Symbol	Unit	Description
Roundabout Influence Area	RIA	ft	The length of the corridor along which the geometric delay due to the roundabout is incurred
Geometric Delay	$Delay_{geo}$	s	The difference between the free flow and unimpeded trajectory travel times
Total Delay	-	s	The difference between the free flow and actual travel times (which vary by time of day)
Free-Flow Speed (FFS)	S_f	mph	The speed at which vehicles desire to travel while not encumbered by geometric elements or congestion
Average Travel Speed	-	mph	The average speed across the analysis segment measured under prevailing traffic conditions
Segment Length	L	ft	The length of the roundabout corridor segment, extending from the yield bar either upstream or downstream to the nearest midsegment point
Spacing	-	ft	The distance to the nearest upstream or downstream roundabout yield bar
Access Points	-	N/A	The number of driveways or side streets along the segment encountered in the direction of travel
Curb Length	L_{curb}	ft	The total length of the segment where a curb is provided
Median Length	L_{median}	ft	The total length of the segment where a median is provided
Approach Width		ft	The width of the travel way at the yield bar
Central Island Diameter	CID	ft	The central island diameter of the roundabout (including the truck apron)
Inscribed Circle Diameter	ICD	ft	The inscribed circle diameter of the roundabout
Circulating Speed	S_c	mph	The average speed at which unimpeded vehicles traverse the interior of the roundabout
Speed Limit	SL	mph	The posted speed
Circulating Lanes	-	N/A	The number of lanes that continue through the roundabout along the approach
Midsegment Lanes	-	N/A	The number of lanes at the midsegment point of the segment
Acceleration Rate	-	ft/s^2	The rate at which vehicles decelerate into (for an upstream segment) or accelerate out of (for a downstream segment) the roundabout
Prop Curb	-	N/A	The proportion of segment with curb
Prop Median	-	N/A	The proportion of segment with restrictive median
Ratio Circulating Speed to Speed Limit	-	N/A	The ratio of circulating speed to the posted speed limit
Ratio Circulating Speed to FFS	-	N/A	The ratio of circulating speed to free-flow speed
Volume-to-Capacity Ratio	v/c	N/A	The volume-to-capacity ratio of the roundabout

In addition to the variables above, a combination variable (circulating delay) term is used to describe the geometric delay incurred while traveling within the circulatory roadway, as opposed to the deceleration and acceleration delays.

This circulating delay term is estimated by the difference between FFS and circulating speed, multiplied by the travel distance along one-third of the circle for a through movement. This concept is illustrated in Exhibit 3-6.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-6: Illustration of Circulating Delay Term



The exhibit shows the expected portion of the delay incurred while traveling around approximately one-third of the circle (distance "x" in the exhibit) at the circulating speed. That travel distance can be estimated as $x = 1/3 * Pi * ICD$. The geometric delay in seconds is then calculated as the difference between the travel time at circulating speed (x/v_{circle}) minus the travel time at free-flow speed (x/v_{ff}).This term is approximated by the following equation:

Equation 3-5

$$d_{circle} = 1/3 * Pi * ICD / s_c - 1/3 * Pi * ICD / s_f$$

$$= 1/3 * Pi * ICD\ (1/s_c - 1/s_f)$$

where

ICD = inscribed circle diameter (ft);

Pi = the number Pi (approximately 3.14);

s_f = free-flow speed (ft/s); and

s_c = circulating speed (ft/s).

For free-flow and circulating speeds given in miles per hour, the equation needs to be modified to the following:

Equation 3-6

$$d_{circle} = 1/3 * 3600/5280 * Pi * ICD\ (1/s_c - 1/s_f)$$

$$= 0.714 * ICD * (1/s_c - 1/s_f)$$

3.2.7. APPLICATION OF MODELS

Exhibit 3-7 illustrates the calculation framework for applying the models within a given segment. The framework should be applied separately for each upstream and downstream sub-segment, before eventually aggregating to the segment and facility levels. The framework is divided into computational steps A through L, with reference being made to the corresponding steps in HCM 2010 Chapter 17 for urban street segments at the appropriate time.

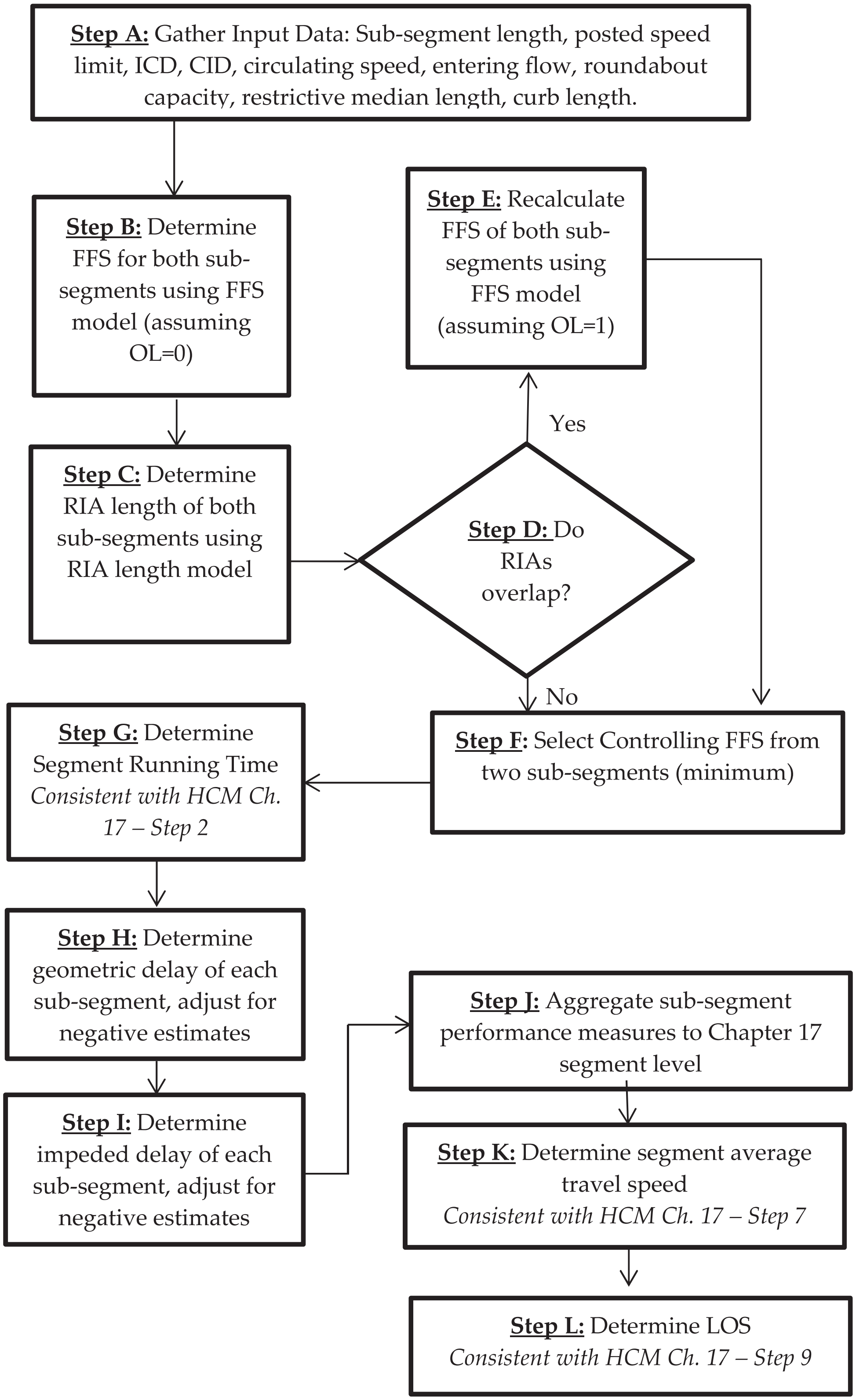
Evaluating the Performance of Corridors with Roundabouts

First, the analyst gathers input data and calculates the FFS based on the posted speed limit, segment length, and an assumption that overlapping RIAs are not present. On a portion of a roundabout corridor between two roundabouts, such as Segment B in Exhibit 3-4, the calculation is performed for the downstream sub-segment (B1 in Exhibit 3-4) and the upstream sub-segment (B2 in Exhibit 3-4). Using the model-predicted FFS and the circulating speed within the roundabout, the RIA length is calculated for both sub-segments. Then the analyst must check whether the RIA lengths overlap. If so, this necessitates a recalculation of the FFS with the overlap (OL) term set equal to 1, which will cause the predicted FFS to decrease.

With the final sub-segment FFS being determined, the analyst selects the controlling FFS for that segment. Since the FFS is defined as being measured at the segment midpoint, the same FFS has to be used for the downstream segment and the next upstream segment. For that purpose, the lower of the two FFS values is selected as the controlling factor.

From the FFS, the procedure uses HCM Chapter 17 Step 2 to estimate the segment running time, followed by roundabout-specific models to estimate geometric delay and impeded delay. From these, the performance measures are aggregated to the HCM Chapter 17 segment level (Step K), and Chapter 17 Step 7 is used to determine the average travel speed on the sub-segment. The team explored a separate, direct estimation of travel speed from the data collected in this project (between steps I and J), but for consistency use of the existing urban street segment method is preferred. From the average travel speed, the LOS is estimated for the urban street segment with roundabouts.

Exhibit 3-7: Computation Process



Note: After Step L, segments can be aggregated to facility level per HCM 2010 Chapter 16.

Evaluating the Performance of Corridors with Roundabouts

Geometric delay, impeded delay, and (optional) average travel speed are calculated separately for upstream and downstream sub-segments because the calculations use variables associated with either the upstream or downstream roundabout. Although it is not common for the FFS to vary from one sub-segment to the next (i.e., within a length of roadway between two roundabouts), geometric differences of the upstream and downstream roundabouts may result in two different predicted FFS. In these cases, the lower of the two estimates is selected as the controlling FFS.

The delays from the two sub-segments are added to produce segment geometric delay and segment impeded delay. Calculations of travel time and travel speed as a percent of base FFS are done in the manner described in Chapter 17 of the HCM 2010 and are therefore not discussed in detail here. A sample application of the methodology to the Old Meridian corridor in Carmel, Indiana, is provided later in this chapter to illustrate the methodology.

The following are the variables necessary to use the models:

- Sub-segment length, defined in Exhibit 3-5, in feet;
- Posted speed limit, in miles per hour (mph);
- Inscribed circle diameter (ICD), in feet;
- Central island diameter (CID), including the truck apron if present, in feet;
- Circulating speed, in miles per hour (mph);
- Entering flow, in vehicles per hour (vph);
- Roundabout capacity, calculated using HCM Chapter 21, in vehicles per hour (vph);
- Length of sub-segment where a restrictive median is present, in feet; and
- Length of sub-segment where a curb is present, in feet.

3.3. MODELING RESULTS

The team assembled a dataset by calculating geometric and operational parameters for each roundabout approach from the following seven roundabout corridors, leaving the two Carmel corridors out for validation purposes:

- MD 216, Scaggsville, Maryland (4 roundabouts)
- La Jolla Boulevard, San Diego, California (5 roundabouts)
- Borgen Boulevard, Gig Harbor, Washington (5 roundabouts)
- SR 539, Whatcom County, Washington (4 roundabouts)
- SR 67, Malta, New York (7 roundabouts)
- Avon Road, Avon, Colorado (5 roundabouts)
- Golden Road, Golden, Colorado (5 roundabouts)

Each approach was partitioned into an upstream segment (extending from the upstream midsegment point to the yield bar) and a downstream segment

Evaluating the Performance of Corridors with Roundabouts

(extending from the yield bar to the downstream midsegment point). The total dataset included 62 roundabout approaches, with each providing an upstream and a downstream segment. Some approaches were excluded due to (a) overlapping influence areas, or (b) a too-short upstream or downstream segment length at the end of the roundabout corridor. In the case of overlapping influence areas, these approaches were only excluded from the roundabout influence area models; these approaches were used in other models like the free-flow speed prediction model. For the too-short segments, the GPS travel-time vehicles were not able to accelerate to their desired speeds (even in free-flow conditions) because of a near-by intersection or turnaround point.

Detailed speed profiles of all seven corridors and for each direction are shown in Appendix N. The numbering and lettering conventions for all corridors are shown in Appendix M. A list of excluded approaches is as follows:

(a) Segments with Overlapping Influence Areas:

- Avon, CO: Segments D1 and D2 (Northbound and Southbound);
- Borgen, WA: Segments B1 and B2 (Eastbound and Westbound);
- MD 216, MD: Segments B1 and B2 (Eastbound and Westbound);
- SR 67, NY: Segments C2 and C3 (Eastbound and Westbound); and
- SR 67, NY: Segments D2 and D3 (Eastbound and Westbound).

(b) Segments Excluded Because of Short Segment Length:

- Avon, CO: Segment A1 (Northbound and Southbound);
- Avon, CO: Segment F5 (Northbound and Southbound);
- Avon, CO: Segment C2 (Northbound, Interchange Unimpeded);
- Avon, CO: Segment B1 (Northbound, Interchange Unimpeded);
- Avon, CO: Segment B2 (Southbound, Interchange Unimpeded);
- Borgen, WA: Segment F5 (Eastbound and Westbound);
- Borgen, WA: Segment A1 (Eastbound and Westbound);
- Golden, CO: Segment A1 (Northbound and Southbound);
- Golden, CO: Segment F5 (Northbound and Southbound);
- SR 67, NY: Segment H7 (Eastbound and Westbound); and
- SR 539, WA: Segment A1 (Northbound and Southbound).

3.3.1. DESCRIPTIVE STATISTICS

After excluding selected approaches, the remaining dataset included 49 upstream and 52 downstream roundabout "approaches." Exhibit 3-8 and Exhibit 3-9 show the descriptive statistics of the variables collected for the upstream and downstream approaches, respectively. The exhibits show the site-level descriptive statistics used for the first three models. The purpose of these exhibits is to show the range of variable values found in the dataset used to develop models. As with any models, the ones developed for this project may not be applicable for conditions beyond the range of the dataset. The descriptive statistics for the two operational-level models are shown in Appendix L.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-8: Descriptive Statistics for Upstream Segments – Site-Level Data

Variable Name	Unit	Mean	St. Dev.	Min	Max
Roundabout Influence Area	ft	311.0	184.8	72.9	897.6
Geometric Delay	sec	2.1	1.5	0.1	6.6
Free-Flow Speed (FFS)	mph	38.1	8.5	26	53
Segment Length	ft	971.8	936.9	244	3993
Spacing	ft	1930.8	1907.6	238	8004
Access Points	N/A	1.9	3.0	0	17
Curb Length	ft	453.1	382.8	0	1627
Median Length	ft	715.1	997.4	37	3993
Approach Width	ft	22.0	6.0	11	38
Central Island Diameter	ft	93.4	33.8	48	187
Inscribed Circle Diameter	ft	142.1	39.7	84	245
Circulating Speed	mph	18.0	2.6	12.4	23.6
Speed Limit	mph	34.7	9.8	25	50
Circulating Lanes	N/A	1.7	0.5	1	2
Midsegment Lanes	N/A	1.6	0.5	1	2
Acceleration Rate	ft/s^2	-0.75	0.30	-1.40	-0.30
Prop Curb	N/A	0.69	0.43	0.00	1.00
Prop Median	N/A	0.72	0.38	0.04	1.00
Ratio Circulating Speed to Speed Limit	N/A	0.5	0.1	0.37	0.94
Ratio Circulating Speed to FFS	N/A	0.5	0.1	0.32	0.78

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-9: Descriptive Statistics for Downstream Segments – Unimpeded

Variable Name	Unit	Mean	St. Dev.	Min	Max
Roundabout Influence Area	ft	617.1	296.1	235.0	1446.2
Geometric Delay	sec	3.0	1.9	0.1	9.5
Free-Flow Speed (FFS)	mph	37.3	8.4	26	53
Segment Length	ft	1049.3	910.6	270	3953
Spacing	ft	1887.7	1861.0	416	8004
Access Points	N/A	1.7	2.7	0	16
Curb Length	ft	574.5	428.1	0	2031
Median Length	ft	831.7	951.5	153	3953
Approach Width	ft	22.2	5.7	11	36
Central Island Diameter	ft	92.9	32.9	48	187
Inscribed Circle Diameter	ft	141.7	38.6	84	245
Circulating Speed	mph	18.0	2.6	11.0	23.6
Speed Limit	mph	34.1	9.7	25	50
Circulating Lanes	N/A	1.7	0.5	1	2
Midsegment Lanes	N/A	1.6	0.5	1	3
Acceleration Rate	ft/s^2	0.51	0.18	0.20	1.20
Prop Curb	N/A	0.74	0.39	0.00	1.00
Prop Median	N/A	0.79	0.31	0.11	1.00
Ratio Circulating Speed to Speed Limit	N/A	0.6	0.1	0.31	0.94
Ratio Circulating Speed to FFS	N/A	0.5	0.1	0.26	0.81

3.3.2. ROUNDABOUT INFLUENCE AREA

This section summarizes the methods used to develop a model for estimating the *roundabout influence area* (RIA), which the team defines as the length of the approach segment where geometric delay is incurred by travelers due to the presence of the roundabout. The premise of this exercise is that the factors that contribute to a larger RIA are also associated with an increase in geometric delay. A model for estimating the RIA is further useful to investigate planning decisions on spacing of adjacent intersections in a roundabout corridor, where closely-spaced roundabouts may result in *overlapping influence areas that prevent a driver from returning to FFS between adjacent roundabouts.*

The objective of the RIA model development is to predict the RIA with design and operational variables available to the analyst at the time of roundabout planning, including geometric and (design) speed parameters of the roundabout that could be obtained or reasonably approximated from concept plans. The model is restricted to the analysis of *unimpeded* vehicles, which traverse the roundabout without any interaction with other traffic. In other words, the data used for the RIA model development were limited to vehicles that experienced geometric delay only, with no interaction or impedance from other vehicles. The team hypothesized that the following factors would affect the RIA:

Evaluating the Performance of Corridors with Roundabouts

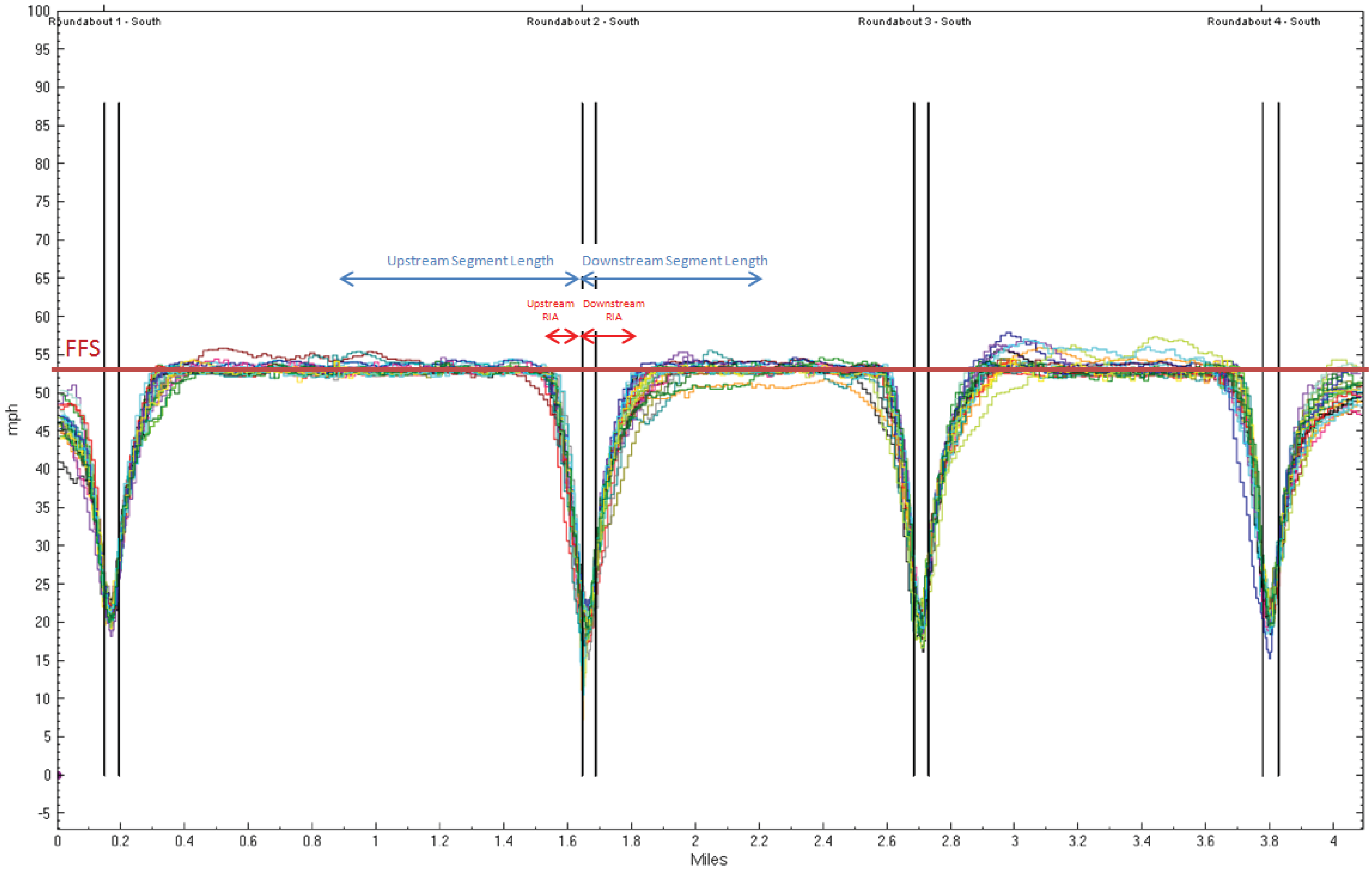
- The *inscribed circle diameter* (ICD) and/or *central island diameter* (CID) were hypothesized to be associated with RIA. A larger circle would lead to increased circulating speeds, and consequently lower geometric delay, which is associated with a shorter RIA;
- The *posted speed limit* and/or the *FFS* along the approach were hypothesized to contribute to a larger RIA, as vehicles would need a longer distance to decelerate, or to recover to a high speed than a low speed;
- The *ratio between the FFS and the circulating speed* is expected to have an impact on the RIA, with a larger ratio (posted and circulating speeds similar) resulting in a lower RIA, as the amount of necessary deceleration and acceleration is reduced. The circulating speed can be field-measured or estimated based on the radius of the R^2 fastest-path curve; and
- The RIA was hypothesized to decrease as the *number of circulating lanes* in the roundabout, the *number of midsegment lanes*, or the *approach width increases*—essentially, the team hypothesized that a wider roundabout or roundabout approach would lead to higher speeds within the roundabout due to the increased radii of fastest paths, thereby decreasing the RIA.

3.3.2.1. *Modeling Approach*

The team examined the speed profiles for each roundabout corridor to get a sense of the FFS on each approach, as well as the length of the RIA. The speed profile in Exhibit 3-10 displays an example of the team's method illustrated for one roundabout on the SR 539 corridor. The analysis steps were as follows:

1. The team used the TravTime™ software to isolate the unimpeded runs by removing the trajectories appearing to be impeded by either heavy volume or conflicting traffic at the roundabouts. The remaining data consisted of unimpeded travel-time runs from all times of day.
2. The team estimated an FFS based on the prevailing speed between roundabouts. In the case shown below, the FFS appeared to be constant (at approximately 53 mph) along the corridor, although many of the other corridors had changes in FFS between roundabouts due to changes in geometry, the posted speed, or other conditions.
3. The beginning and end points to each RIA were determined by estimating the points where the speeds started to deviate from FFS (upstream), or when the speed trajectories recovered to FFS. From these measurements, the team calculated various potential explanatory variables, as well as the dependent RIA variable as described below.

Exhibit 3-10: Roundabout Influence Area Example (Profiles for SR539 Site Northbound)



Evaluating the Performance of Corridors with Roundabouts

3.3.2.2. *Variable Correlations*

The team investigated the relationships between RIA length and the remaining variables in Exhibit 3-5 by calculating the correlation coefficient (R^2) between each pair of variables. This effort was done separately for the upstream RIA and for the downstream RIA. The correlation data for the two datasets are contained in Appendix L. A correlation coefficient close to zero corresponds to little or no correlation between variables, while a coefficient close to +1.0 or −1.0 refers to a strong positive or negative correlation, respectively.

The team made the following observations about the correlation between RIA length and the other variables:

- For the upstream and downstream datasets, RIA length was moderately positively correlated (r = +0.42 to +0.77) with segment length, spacing, and FFS.
- For the upstream and downstream datasets, RIA length was moderately negatively correlated with the ratios of circulating speed to speed limit and FFS (r = −0.63 to −0.41).
- The RIA length was *not* correlated with approach width, circulating speed, number of circulating lanes, number of midsegment lanes, or proportion of segment with median.

Additionally, many of the potential explanatory variables were highly correlated with each other, including the following pairs of variables:

- Median length and number of access points;
- Median length, speed limit, and FFS;
- Approach width, number of circulating lanes, CID, and ICD; and
- Number of midsegment lanes and number of circulating lanes.

Many of these trends are intuitive, but the team notes these relationships to avoid multicolinearity in the models. In general, if two independent variables are correlated, only one should be used in model development. The detail correlation coefficients are shown in Appendix L, with values greater than +0.5 and less than −0.5 emphasized in bold.

3.3.2.3. *Model Results*

Building on the correlational analysis, the team developed several regression models using the Statistical Analysis Software (SAS) general linear model (GLM) procedure. The team strived to avoid using highly-correlated variables in the same model. Models were developed separately for the upstream and downstream segment datasets. Appendix L shows a series of variable plots, followed by a full list of regression models considered for the RIA models. Based on the regression results, the team made the following observations:

- The models for downstream segment influence area length showed better statistical fit than the upstream length, which is indicated by the higher levels of R^2 for the downstream models (43 to 71 percent as

Evaluating the Performance of Corridors with Roundabouts

opposed to 16 to 38 percent). The R^2 can be interpreted as the portion of variability in the dependent variable that is explained by the model. A low R^2 for the upstream model, therefore, corresponds to more unexplained variability. Another way to look at this would be that driver acceleration behavior can be more consistently described than deceleration behavior at the different roundabouts.

- Several of the models indicated RIA length increases with the posted speed limit, FFS, median length, curb length, ICD, and CID.
- Several of the models indicated RIA length decreases as the approach width, number of midsegment lanes, circulating speed, and the ratio of circulating speed to FFS increase.
- Of these variables, those that had the greatest effect on RIA length were speed limit, FFS, circulating speed, and the ratio of circulating speed to FFS. This was indicated by the high Type III sum of squares and low *p*-value for these variables, as well as the relatively high R^2 of the models containing these variables.
- The models for upstream and downstream RIA are very different in parameter estimates. As a result, the two components should be estimated separately.

From the results in Appendix L, it appears that models D8 and U8 have the highest model R^2 for the downstream and upstream RIA, respectively. The RIA models are shown in Exhibit 3-11.

Exhibit 3-11: RIA Models

Model	Intercept	Free-Flow Speed (mph)	Circulating Speed (mph)	R^2
U8	165.9	13.8***	-21.1**	0.289
D8	-149.8	31.4***	-22.5**	0.714

* = p < 0.1   ** = p < 0.05   *** = p < 0.01

In equation form, Model U8 would be written as:

Equation 3-7

$$RIA_{upstream} = 165.9 + 13.8 * S_f - 21.1 * S_c$$

where

$RIA_{upstream}$ = upstream roundabout influence area (feet);

S_c = circulating speed (feet/second); and

S_f = free-flow speed (feet/second).

Some additional sensitivity analysis was performed on these two models to explore the behavior of the models. Exhibit 3-12 shows sensitivity of both models by FFS and circulating speed. The field-observed data for the upstream and downstream RIA are superimposed on the final models.

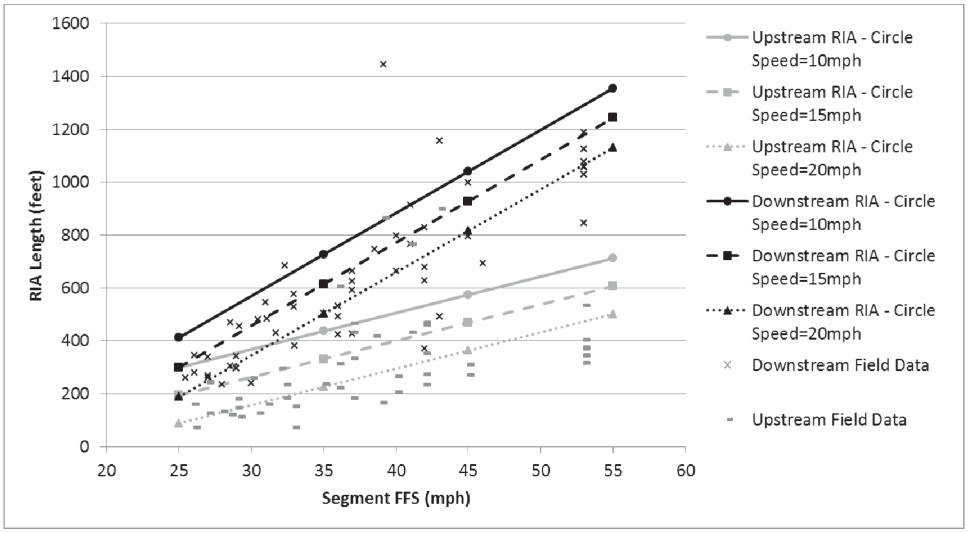


Exhibit 3-12: Sensitivity Analysis of RIA Models

The exhibit intuitively shows RIA increases with greater segment FFS (*x*-axis), and decreases with faster circulating speed. The downstream RIA is generally greater than the upstream RIA. This may be a direct artifact of the segment definitions, which divide a segment at the yield line. Consequently, the downstream segment contains travel within the roundabout, which occurs at slow speeds.

From these results, the total roundabout influence area can be estimated from the summation of upstream and downstream lengths, as follows:

Equation 3-8

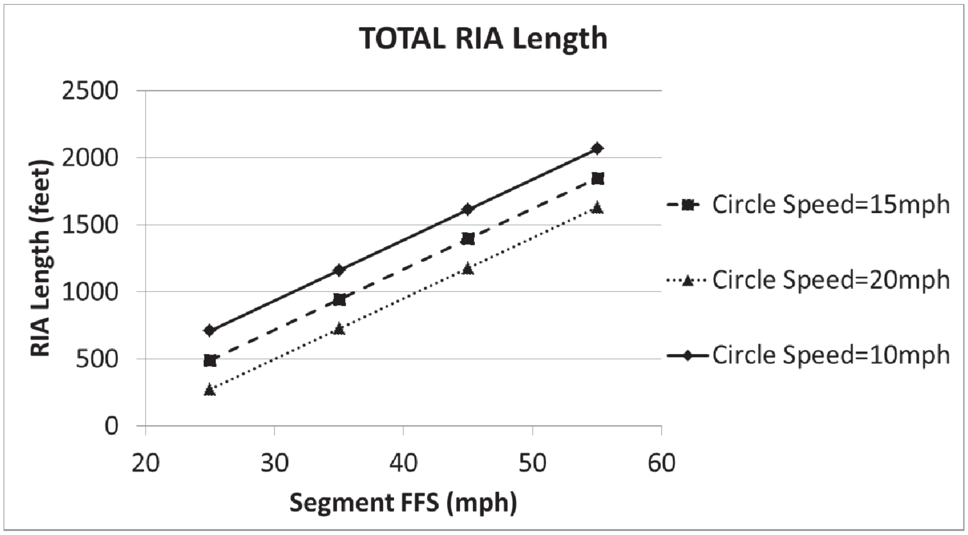
$$RIA_{Total} = RIA_{upstream} + RIA_{downstream} = \text{Model U8} + \text{Model D8}$$

$$RIA_{Total} = (165.9\text{-}149.8) + (13.8+31.4)*S_f - (21.1+22.5)*S_c$$

$$\mathbf{RIA_{Total} = 16.1 + 45.2 * S_f - 43.5 * S_c}$$

The total RIA estimation in Equation 3-8 is plotted in Exhibit 3-13.

Exhibit 3-13: Total RIA Based on Sum of Upstream and Downstream Lengths



The model in Exhibit 3-13 shows the same trends as the individual segment models, with increasing RIA with higher FFS and lower circulating speed. The graph illustrates roundabouts on roads with a high FFS that are designed with a low circulating speed can have total RIAs greater than 1,600 feet or about 1/3 of a mile.

3.3.3. GEOMETRIC DELAY

The team also modeled the geometric delay incurred to drivers at roundabouts as a function of several geometric elements. The purpose of this modeling effort is to predict the additional travel time caused by the curvature of the roundabout. A challenge in developing this model was to account for overlapping roundabout influence areas. In the case of closely-spaced roundabouts, for example, the geometric delay for each roundabout may not be easily discernible.

3.3.3.1. *Modeling Approach*

For each upstream and downstream segment in the dataset, the team estimated the unimpeded and free-flow travel times based on the corridor space-time trajectories and speed profiles, respectively. This was accomplished using methods similar to those in the first two steps described in Section 3.3.2.1. Then the geometric delay was calculated by taking the difference between the unimpeded and free-flow travel times, as shown in Exhibit 3-6. The team used a similar approach to develop the geometric delay model as was used to develop the roundabout influence area model, with the advantage that now the influence area could be used as an explanatory variable. The SAS GLM procedure was used to develop separate models for the upstream and downstream segments, and several models were developed for each of these two datasets.

Evaluating the Performance of Corridors with Roundabouts

3.3.3.2. *Variable Correlations*

The team investigated the relationships between geometric delay and the remaining variables in Exhibit 3-5 by calculating the correlation coefficient between each pair of variables. This effort was done separately for the upstream geometric delay and for the downstream geometric delay. The correlation data for the two datasets are contained in Appendix L.

The team made the following observations about the correlation between geometric delay and the other variables:

- The geometric delay was highly positively correlated (r = +0.76) with RIA length, which is expected due to the way the RIA is defined. However, it may be impractical to develop a geometric delay model based solely upon RIA length, as it is more cumbersome to determine than ordinary geometric elements of the roundabout.
- The geometric delay was moderately positively correlated (r = +0.41 to +0.46) with free-flow speed and the central island diameter, which was intuitive based on how these elements should affect travel time at roundabouts.
- The geometric delay for downstream segments was highly positively correlated (r = +0.83) with the circulating delay term, which describes the difference in travel time around 1/3 of the inscribed circle diameter if traveling at the circulating speed versus the free-flow speed.

Like the RIA modeling effort, the team found several pairs of variables that were correlated with each other, including the FFS and central island diameter (r = +0.63) and the FFS and posted speed limit (r = +0.90). Two independent variables with high correlation should not be used within the same model.

3.3.3.3. *Model Results*

Building on the correlational analysis, the team developed several regression models using the SAS GLM procedure. The team strived to avoid using highly-correlated variables in the same model. Models were developed separately for the upstream and downstream segment datasets. Appendix L shows a series of variable plots, followed by a full list of regression models considered for the geometric delay models. Based on the regression results, the team made the following observations:

- The simplest models for the upstream and downstream datasets were based solely on the roundabout influence area, which yielded an R^2 of approximately 60 percent.
- The models for downstream geometric delay showed consistently better statistical fit than the upstream length, which is indicated by the higher levels of R^2 for the downstream models (50 to 80 percent as opposed to 30 to 60 percent). Like the influence area models, another way to look at this would be that driver acceleration behavior can be more consistently described than deceleration behavior at the different roundabouts.

- Several of the models indicated geometric delay increases with FFS and number of access points but decreases with the circulating speed.
- An interaction term between the FFS, circulating speed, and inscribed circle diameter (ICD) was also used to model geometric delay, with good results for the downstream geometric delay model. This circulating delay term, as shown in Exhibit 3-14, is based on the difference between FFS and circulating speed. Intuitively, this term applies to the downstream geometric delay, as it includes the geometric delay within the circle.

From the results in Appendix L, it appears models U10 and D15 have the highest model R^2 for the upstream and downstream geometric delay, respectively. The models for geometric delay are shown in Exhibit 3-14.

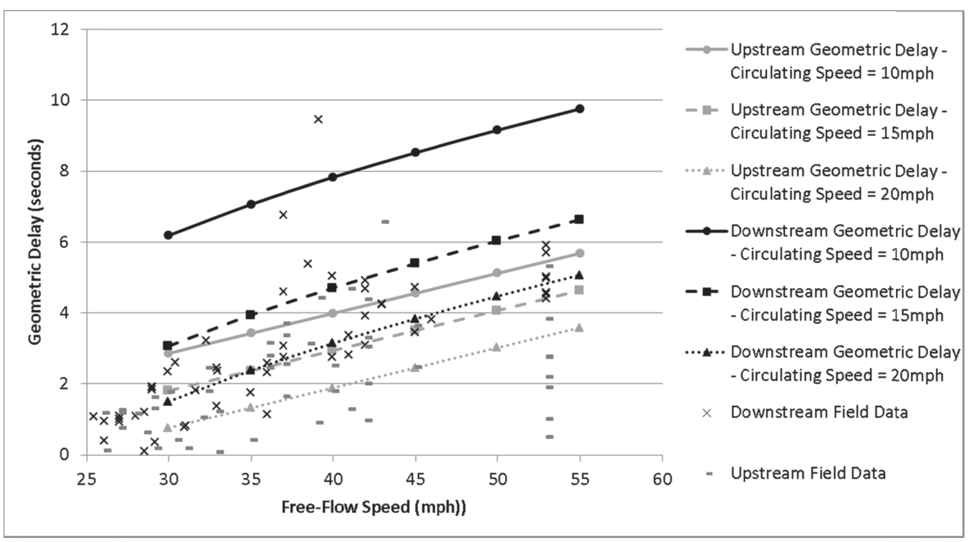
Exhibit 3-14: Geometric Delay Models

Model	Intercept	Free-Flow Speed (FFS)	Circulating Speed (mph)	Circulating Delay****	R^2
U10	1.57	0.11***	-0.21**	n/a	0.315
D15	-2.632***	0.0859***	n/a	0.7261***	0.794

* = p < 0.1 ** = p < 0.05 *** = p < 0.01
**** Circulating delay is defined in Exhibit 3-6

Some additional sensitivity analysis was performed on these two models to explore the behavior of the models similar to the previous section. Exhibit 3-15 shows sensitivity of both models by FFS and circulating speed.

Exhibit 3-15: Upstream and Downstream Geometric Delay Model Sensitivity



The exhibit shows geometric delay increases with greater segment FFS (*x*-axis), and decreases with faster circulating speed. The downstream geometric delay is generally greater than the upstream portion, which is because the downstream

Evaluating the Performance of Corridors with Roundabouts

segment contains a greater portion of travel around the circle (at a reduced speed).

The downstream geometric delay graphs are shown for a fixed inscribed circle diameter (ICD) of 150 feet. A greater ICD would result in a further increase in downstream geometric delay because of additional travel distance around the circle. In a practical application, this increase in geometric delay would be offset by a faster circulating speed (resulting from the larger circle diameter), which reduces the effect of the circulating delay term in model D15.

From these upstream and downstream results, the total geometric delay for a roundabout node can be estimated from the summation of upstream and downstream geometric delays, as follows:

Equation 3-9

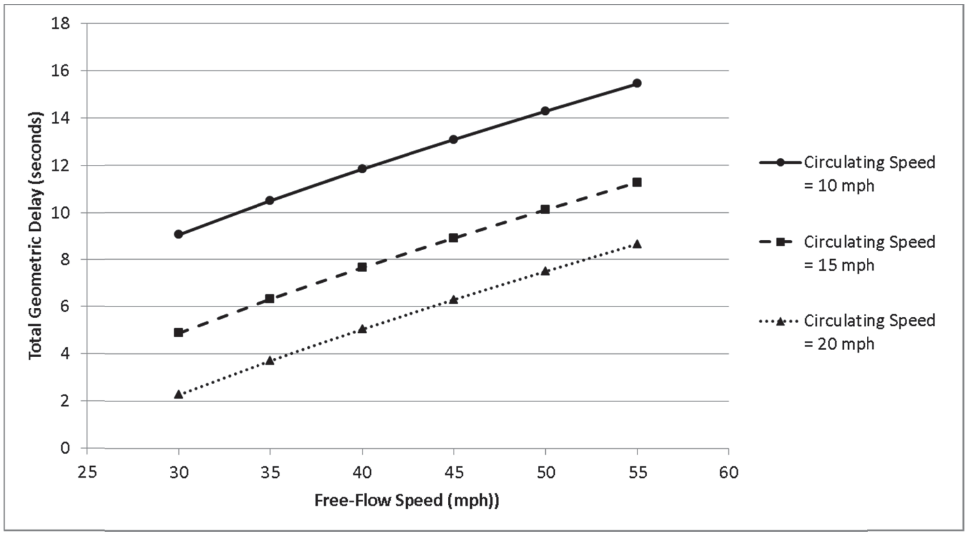
$$Delay_{geom,\ total} = Delay_{geom,\ upstream} + Delay_{geom,\ downstream} = \text{Model U10} + \text{Model D15}$$

$$Delay_{geom,\ total} = (1.57\text{-}2.63) + (0.11\text{+}0.09)*S_f - (0.21)*S_c + 0.84*0.714 * ICD * (1/S_c - 1/S_f)$$

$$\mathbf{Delay_{geom,\ total} = -1.06 + 0.20*S_f - 0.21*S_c + 0.625 * ICD * (1/S_c - 1/S_f)}$$

The resulting total geometric delay for a roundabout node is shown in Exhibit 3-16, again assuming an ICD of 150 feet. The general trend of this combined model is consistent with the earlier exhibit, as well as the shape of the roundabout influence area model. In future editions of the HCM, this model is most applicable in Chapter 21, with the individual upstream and downstream models most applicable in Chapter 17.

Exhibit 3-16: Combined Geometric Delay Model Sensitivity



3.3.4. FREE-FLOW SPEED PREDICTION

The next modeling effort concerned the FFS of each roundabout segment. The team proceeded to model FFS as a function of the geometric elements of the

Evaluating the Performance of Corridors with Roundabouts

corridor (e.g., segment length, spacing, and midsegment number of lanes), as well as the geometry of the roundabout itself (e.g., ICD). The team also hypothesized that the FFS would be highly correlated with the posted speed limit. The model allows the practitioner to estimate the FFS on an upstream or downstream roundabout segment in the absence of field data. From the modeling efforts described in previous sections, it is also useful to estimate the FFS since it emerged as a key model input to the geometric delay and roundabout influence area models.

3.3.4.1. *Modeling Approach*

For each upstream and downstream segment in the dataset, the team estimated the FFS based upon the free-flow trajectories. This was accomplished using a method similar to the second step described in Section 3.3.2.1. The segment-by-segment FFS measurements are documented in Appendix L. The FFS measurements were obtained from unimpeded trajectories through the roundabout corridors performed by members of the research team. While these unimpeded routes were intended to be free of the impact of other traffic, there was still some variability in the observed speed profiles shown in the appendix. The team initially used the average unimpeded midsegment speed as an estimate of FFS, but that definition proved challenging for multiple reasons:

1. From the trajectory data, it was unclear if all routes were truly unimpeded without interaction from other traffic;
2. The average midsegment speed in some cases was significantly lower than some of the individual unimpeded trajectories, which resulted in negative calculated travel times; and
3. The drivers collecting data were instructed to follow a "floating car" approach, which resulted in a significant portion of other drivers actually traveling faster than the data collection vehicle.

In response to these challenges, the team defined the midsegment FFS as the *maximum unimpeded trajectory speed* recorded by the travel-time vehicle. This guaranteed a positive and more realistic estimate of the geometric delay introduced by the roundabout. In an application of this method, this midsegment FFS can be interpreted as a realistic unimpeded speed of a driver familiar with the roundabout corridor but not overly aggressive in their travel behavior.

Just as before, the analysis was performed separately for the upstream and downstream segments of each roundabout. While this is consistent with the approach taken for RIA and geometric delay, the interpretation of the resulting model is quite different. While in the previous two models, a combined model could be estimated by simply adding the component sub-segment models, an additive approach is not realistic in this case. In fact, the downstream FFS for a roundabout is measured at the same point as the upstream FFS of the next roundabout along the corridor.

Consequently, the team recommends interpreting the FFS prediction as estimated FFS resulting from the impact of a roundabout on the respective upstream or downstream segment. For a midsegment FFS between two

Evaluating the Performance of Corridors with Roundabouts

roundabouts, the *prevailing FFS is defined as the minimum of the two sub-segment FFS estimated for that segment*. In other words, the analyst would calculate the downstream FFS for Roundabout 1 and the upstream FFS for Roundabout 2, and apply the smaller of the two numbers as the midsegment FFS for Segment B (see Exhibit 3-4 for numbering and lettering conventions). The FFS model is considered to be a *constraint model*, in which the characteristics of the segment and adjacent roundabout impact driver FFS at the midsegment point. Since in a corridor the downstream FFS from Roundabout 1 is defined at the same location as the upstream FFS of Roundabout 2, the lower of the two would act as the constraint for that segment. Therefore, the team recommends using the minimum of the two estimated FFS as the prevailing condition for the segment in question.

The SAS GLM procedure was used to develop separate models for the upstream and downstream segments, and several models were developed for each of these two datasets.

3.3.4.2. *Variable Correlations*

The team investigated the relationship between FFS and the remaining variables in Exhibit 3-5 by calculating the correlation coefficient between each pair of variables. This effort was done separately for the upstream FFS and for the downstream FFS. The correlation data for the two datasets are contained in Appendix L.

The team made the following observations about the correlation between FFS and the other variables:

- The FFS was highly positively correlated (r = +0.73 to +0.90) with the posted speed limit, segment length, spacing, and median length.
- The FFS was moderately positively correlated (r = +0.50 to +0.64) with the approach width, CID, ICD, and circulating speed.
- The FFS was moderately negatively correlated (r = −0.73 to −0.55) with the acceleration rate, the proportion of segment with curb, and the ratio of circulating speed to the posted speed limit.

Like the geometric delay modeling effort, the team found several pairs of variables that were correlated with each other: the segment length and spacing (r = +0.97), the CID and ICD (r = +0.94), and the segment length and median length (r = +0.91).

3.3.4.3. *Model Results*

Building on the correlational analysis, the team developed several regression models using the SAS GLM procedure. The team strived to avoid using highly-correlated variables in the same model. Models were developed separately for the upstream and downstream segment datasets. Appendix L shows a series of variable plots, followed by a full list of regression models considered for the FFS models. Based on the regression results, the team made the following observations:

Evaluating the Performance of Corridors with Roundabouts

- The simplest models for the upstream and downstream datasets were based on a combination of the segment length and posted speed limit. These models explained 80 to 90 percent of the variability in the data, which is a much better statistical fit for either the RIA or geometric delay models.
- The models could be slightly improved by accounting for segments with overlapping roundabout influence areas. These segments were identified by the research team using the corridor speed profiles. If it appeared that drivers did not have enough distance to recover to FFS between two adjacent roundabouts, then these roundabouts were defined to have overlapping influence areas. The addition of a separate intercept term for these segments improved the model R^2 by approximately 3 percent.
- Although the downstream segment models explained slightly more of the data variability, there did not appear to be as much discrepancy between the strength of the upstream and downstream models as there was in previous modeling efforts.

Exhibit 3-17 summarizes the models for upstream and downstream FFS as a function of the segment length, posted speed limit, central island diameter, and a separate intercept term (or dummy variable) for the segments with overlapping influence areas (OL).

Exhibit 3-17: FFS Prediction Models

Model	Intercept	Segment Length (ft)	Speed Limit (mph)	Central Island Diameter (ft)	OL (dummy)	R^2
U10	15.1***	0.0037***	0.43***	0.05***	-4.73***	0.901
D10	14.6***	0.0039***	0.48***	0.02**	-4.43***	0.926

* = p < 0.1 ** = p < 0.05 *** = p < 0.01

The sensitivity of these models is shown in Exhibit 3-18. The exhibit assumes a central island diameter of 100 feet, with a larger diameter resulting in a net increase in upstream and downstream FFS.

The exhibit further assumes the upper limit of the overlapping influence area occurs within a segment length of 500 feet to 1,000 feet. This assumption is generally in line with the RIA models estimated above and was used here to illustrate an upward shift in free-flow speed of about 4.4 to 4.7 mph once the RIAs no longer overlap. In a practical application of these models, the analyst should apply the RIA estimation equation above to assess whether the overlap condition is met or not.

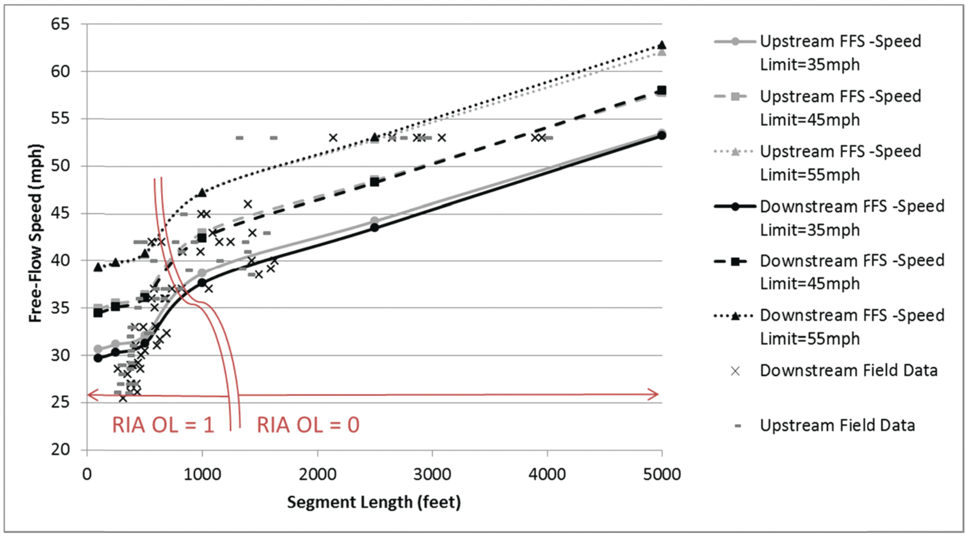


Exhibit 3-18: FFS Model Sensitivity

Note: Assumed Central Island Diameter of 100 ft

3.3.5. AVERAGE TRAVEL SPEED

This section describes the development of a model to predict the average speed for the upstream or downstream sub-segment between roundabouts along a roundabout corridor. Unlike the previous models described in this report, this model is meant to include operational data that vary by time of day, such as entering traffic volumes and circulating flows, in addition to the geometric and operational data contained in the unimpeded models above. The purpose of this modeling effort is to allow a practitioner to estimate the operational performance along a midsegment portion of a roundabout corridor based on traffic data and geometric characteristics of the roadway.

In the context of implementation in the HCM 2010 Chapter 17 Urban Street Segment procedure, this step is not needed, as the average travel speed is calculated from prior modeling steps, including the free-flow speeds and various delay terms. The average travel speed model presented here is optional and can be used to verify the predictions from the Chapter 17 methodology.

3.3.5.1. *Modeling Approach*

For each upstream and downstream segment in the dataset, the team calculated the average speed along the segment during three time-of-day periods. For the average travel speed models, the team used all approaches and segments for which roundabout volumes were available from the data collection. Unlike the data used in the roundabout influence area, geometric delay, and FFS model development, these data included trajectories that may have been impeded by circulating traffic, as well as the unimpeded trajectories. Thus, these trajectories were much more variable than the unimpeded trajectories in the previous analyses, even when the data were broken down by time of day. The FFS measurements were obtained from unimpeded trajectories through the roundabout corridors performed by members of the research team.

The team excluded a few segments that had resulted in negative total delay estimates as outliers. A negative total delay estimate may have occurred for

Evaluating the Performance of Corridors with Roundabouts

segments that were very short and where the free-flow speed estimate from the unimpeded routes was similar or somewhat less than the observed speed for other trajectories. Clearly, these negative delays were an attribute of driver behavior during field-data collection—not an adequate reflection of roundabout performance—and would have introduced inconsistencies and bias in the model estimation. The same segments were removed from the average travel speed and impeded delay models for consistency across the two models.

Like the other prediction models, this model is intended to be applied separately for upstream and downstream segment data. In the case of two adjacent roundabouts, the combined average travel speed may be estimated as the length-weighted average of estimates for the downstream segment of Roundabout 1 and the upstream segment of Roundabout 2. The SAS GLM procedure was used to develop separate models for the upstream and downstream segments, and several models were developed for each of these two datasets.

3.3.5.2. *Variable Correlations*

The team investigated the relationship between average speed and the remaining variables in Exhibit 3-5 by calculating the correlation coefficient between each pair of variables. This effort was done separately for the upstream speed and for the downstream speed. The correlation data for the two datasets are contained in Appendix L. The team made the following observations about the correlation between FFS and the other variables:

- The average speed was moderately negatively correlated with entering traffic flow, which is intuitive, as the speed should decrease as traffic congestion increases.
- The average speed was moderately negatively correlated with circulating traffic flow for the upstream segments, but uncorrelated with circulating traffic flow for the downstream segments. This is also intuitive, as congestion within a roundabout should be detrimental to speed along the upstream segment but should not affect the downstream speed.
- The average speed was also correlated with several of the same variables as the FFS, such as the posted speed limit and segment length, but these variables did not vary by time of day and are thus not included in the correlation summary.

3.3.5.3. *Model Results*

Building on the correlational analysis, the team developed several regression models using the SAS GLM procedure. The team strived to avoid using highly-correlated variables in the same model. Models were developed separately for the upstream and downstream segment datasets. Appendix L shows a series of variable plots, followed by a full list of regression models considered for the average speed models. Based on the regression results, the team made the following observations:

Evaluating the Performance of Corridors with Roundabouts

- Several models for the upstream segment data were developed as a function of the circulating flow, entering flow, posted speed limit, and segment length, which were all significant ($p < 0.021$).
- The downstream segment models were similar, although the circulating flow was not significant (see discussion above).
- The models were successful in explaining 70 (upstream) to 79 (downstream) percent of the variability in the data.

Exhibit 3-19 summarizes the models for upstream and downstream average travel running speed as a function of the FFS and the volume-to-capacity ratio of the approach. The full list of average travel speed models is shown in Appendix L.

Exhibit 3-19: Average Travel Speed Final Models

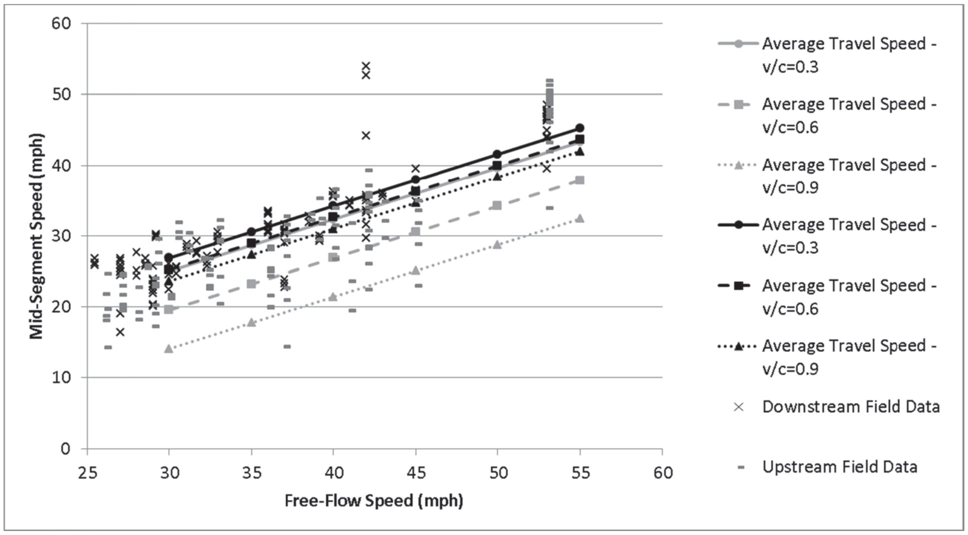
Model	Intercept	Free-Flow Speed (mph)	Volume-to-Capacity Ratio	R^2
U12	8.52**	0.73***	-18.20***	0.76
D12	6.45***	0.74***	-5.40***	0.83

* = p < 0.1 ** = p < 0.05 *** = p < 0.01

In the application of the models in Exhibit 3-19, the FFS would be calculated from one of the earlier models, as a function of segment length, speed limit, roundabout central island diameter, and a binary variable checking for overlapping roundabout influence areas. That last term is a function of the RIA estimation models, which are in turn a function of FFS and circulating speed. The volume-to-capacity ratio is calculated using entering volume, and the theoretical roundabout capacity is calculated from the equations in the HCM.

The sensitivity of these models is shown in Exhibit 3-20. The exhibit shows an increasing average travel speed with increasing FFS, and a reduction resulting from a higher volume-to-capacity ratio.

Exhibit 3-20: Average Travel Speed Sensitivity Analysis Plots



3.3.6. IMPEDED DELAY

The team's final modeling effort included predicting the impeded delay at each roundabout due to the interaction among vehicles. This differs slightly from the control delay methodology in HCM Chapter 21 because the field data used to calibrate the models presented here was collected from roundabout corridors rather than isolated roundabouts and is based on empirical regression rather than analytical derivation. It further includes some corridor impedances resulting from curbs, medians, and the prevailing traffic volume. However, in a validation and verification exercise, the team compared the resulting impeded delay models to the current control delay models in the HCM 2010.

3.3.6.1. *Modeling Approach*

The team began the model calibration effort by calculating the geometric and impeded delay for each upstream and downstream roundabout segment over each time of day. The geometric delay was computed using the same methodology described in Section 3.3.2.1., and the impeded delay was calculated by taking the difference between the average travel time and the free-flow travel time for each segment. Segments with incomplete data (e.g., a short segment on the end of a corridor of roundabouts between the last roundabout and where the driver collecting the data turned around) were excluded from the dataset. The time-of-day dependent variables such as entering and circulating flow were defined using the same methods described in Section 3.3.5.1. Additionally, the team calculated the capacity and volume-to-capacity ratio of the roundabout nearest to each segment (i.e. downstream of an upstream segment and upstream of downstream segment) using the roundabout capacity equations contained in the HCM Chapter 21. As with the other models, the team used the SAS GLM procedure to develop several models for the upstream and downstream segment datasets.

3.3.6.2. *Variable Correlations*

The team performed a correlation analysis similar to the analysis presented in Section 3.3.5.2., but using impeded delay as the dependent variable. The full correlation analysis results are presented in Appendix L. These *operational* data tables include time-of-day specific volumes, and are thus different from the *site-level* correlation tables in Appendix L, which were used for the RIA, FFS, and geometric delay data.

The team made the following observations about the data:

- The impeded delay for the upstream segments was positively correlated (r = +0.66) with the conflicting flow within the roundabout and negatively correlated (r = −0.66) with the capacity of the roundabout.
- The impeded delay for the downstream segments was positively correlated with the posted speed limit (r = +0.51), free-flow speed (r = +0.64), and segment length (r = +0.55) but was not strongly negatively correlated with any of the variables.

Additionally, several pairs of the possible explanatory variables appeared to be highly correlated with each other, including entry flow and volume-to-capacity ratio, posted speed limit and FFS, CID and ICD, and median length and segment length. During model development the team avoided using highly-correlated explanatory variables within the same model.

3.3.6.3. *Model Results*

The team then used the observations from the correlation analysis to develop several models for the upstream and downstream segment datasets under the SAS GLM procedure. Appendix L displays a series of variable plots, followed by a full list of regression models considered for the impeded delay models. Based on the regression results, the team made the following observations:

- The upstream and downstream delays were best modeled as a function of FFS and the volume-to-capacity ratio.
- The downstream delay was also related to the segment length, median length, and curb length. Although these three variables may be correlated to each other, the presence of these variables in the HCM Chapter 17 models motivated the team to still consider these variables.
- The models with the best predictive ability in either case explained 55 to 70 percent of the variability in the data.

Exhibit 3-21 summarizes the models for upstream and downstream delay as a function of these elements. The upstream impeded delay is a function of FFS, volume-to-capacity ratio, and entering flow rate. The downstream impeded delay is a function of FFS, volume-to-capacity ratio, segment length, median length, and curb length across the segment.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-21: Impeded Delay Final Models

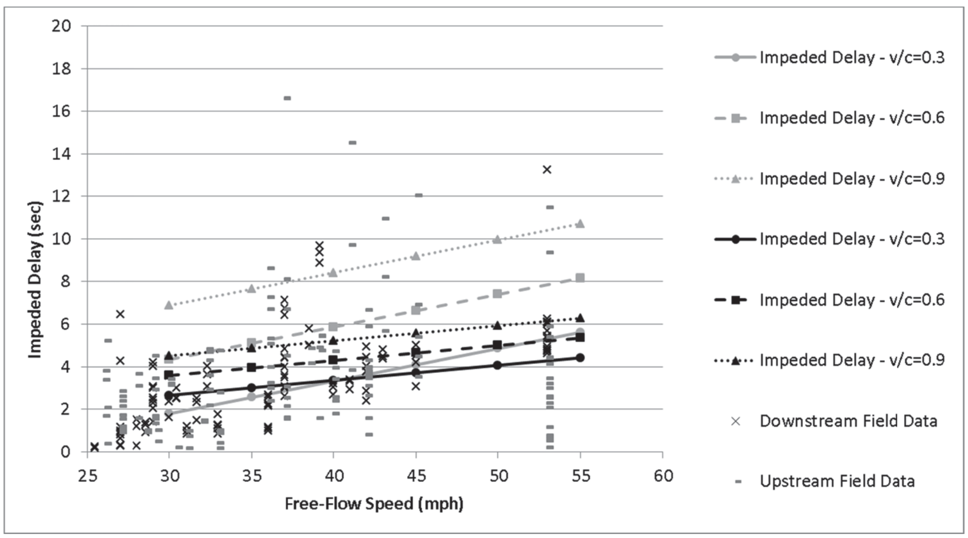
Model	Int.	Free-Flow Speed (mph)	Volume-to-Capacity Ratio	Entering Flow (veh)	R^2
U7	-5.35***	0.15***	42.50***	-0.03***	0.67

Model	Int.	Free-Flow Speed (mph)	Volume-to-Capacity Ratio	Segment Length (ft)	Median Length (ft)	Curb Length (ft)	R^2
D6	-2.65**	0.07*	3.10***	0.0020**	-0.0010*	0.0014**	0.56

* = p < 0.1 ** = p < 0.05 *** = p < 0.01

The full list of impeded delay models is shown in Appendix L. The sensitivity of these models is shown in Exhibit 3-22. The exhibit shows increasing delay with greater FFS, as well as increasing volume-to-capacity ratio. The entering flow rate for Model U7 was set at 1,000 vehicles times the modeled volume-to-capacity ratio, resulting in 300, 600, and 900 vehicles/hour for the three volume-to-capacity ratios used in the graph. For Model D6, the three segment lengths were fixed at 1,000 feet for total length and 500 feet each for median and curb length (corresponding to 50 percent of segment with median and curb).

Exhibit 3-22: Impeded Delay Sensitivity Analysis Plots



Evaluating the Performance of Corridors with Roundabouts

3.4. MODEL VALIDATION AND APPLICATION

This section presents a model validation and application exercise in two parts. First, the results of the various sub-models will be validated internally against the field-observed data. Specifically, the team compares model predictions to field-observed performance data for FFS, roundabout influence area, geometric delay, impeded delay, and average travel speed. Second, the team presents an external validation of the methodology through application to the Old Meridian Road and Spring Mill Road corridors in Carmel, Indiana. Both corridors were excluded from model development, and thus represent a true validation of the methodology. The Old Meridian Road corridor is further presented in a step-by-step application of the HCM 2010 Chapter 17 Urban Street Segments method, integrated with modifications for the roundabout nodes. Because the Old Meridian corridor contains one signalized intersection, the application exercise further illustrates how the method can be applied to mixed corridors of signals and roundabouts.

3.4.1. INTERNAL MODEL VALIDATION

To validate the various sub-models developed in this research, this section compares the model predictions to the field-observed performance data for FFS, roundabout influence area, geometric delay, impeded delay, and average travel speed. The following sections use the data from the seven corridors used in model development, excluding the two Carmel, Indiana, corridors (which are evaluated separately in Section 3.4.2).

3.4.1.1. *Free-Flow Speed*

The FFS prediction model is a function of the speed limit (mph), segment length (ft), central island diameter (ft), and a binary variable checking for overlapping roundabout influence areas. Plots for field-measured versus predicted FFS are shown in Exhibit 3-23 for upstream and downstream segments. The plots show a wide observed range of free-flow speeds from approximately 20 to 55 mph, and that the same range is predicted from the model. Further, the upstream and downstream models show an overall model R^2 fit of 0.90 and 0.93, respectively, with intercept terms close to zero and a slope close to 1.0. Overall, these results point to a very good model fit to the field-observed data.

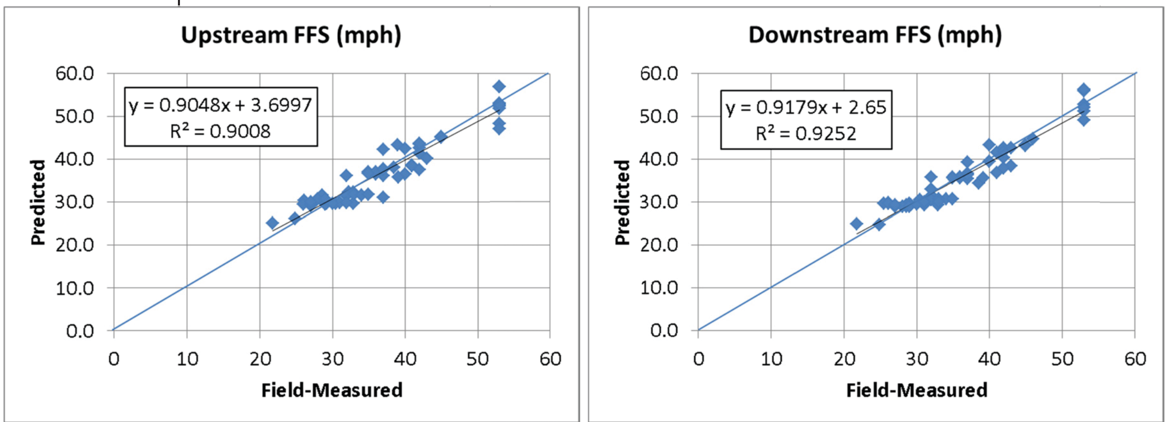


Exhibit 3-23: Internal Validation – Free-Flow Speed Models

3.4.1.2. *Roundabout Influence Area*

The roundabout influence area (RIA) model predicts the spatial extent of the speed-reducing effects of the roundabout measured upstream and downstream from the yield line. The RIA is predicted as a function of the segment free-flow speed and the circulating speed.

Data plots for field-measured versus predicted free-flow speed are shown in Exhibit 3-24 for upstream and downstream segments. The plots show a model R^2 fit of 0.24 and 0.57 for upstream and downstream RIA, respectively. Therefore, the resulting models do not fit as well as the free-flow speed models, which is directly attributable to the high variability of RIAs across the corridors. Nonetheless, the RIA models generally capture the range of field-observed data.

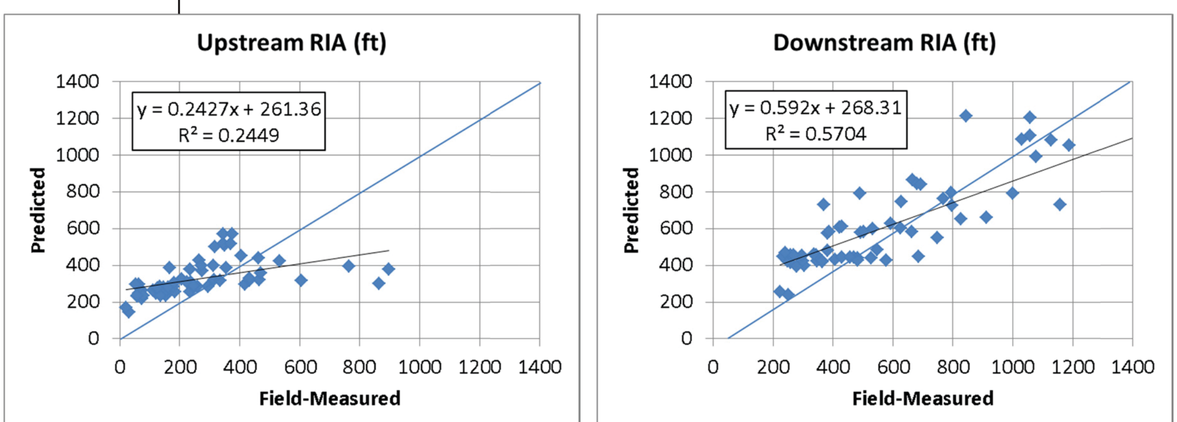


Exhibit 3-24: Internal Validation – Roundabout Influence Area Models

3.4.1.3. *Geometric Delay*

The geometric delay upstream of the roundabout is predicted as a function of the free-flow speed and the circulating speed, just as the roundabout influence area. The downstream geometric delay further includes an effect of the inscribed circle diameter, which is combined with FFS and circulating speed into a circulating delay term.

In Exhibit 3-25, the plots of field-observed data versus model prediction generally show a better fit for the downstream models (R^2 = 0.50) than the upstream models (R^2 = 0.24). Both models suggest having a somewhat "flat" slope, with the models over-predicting some low geometric delays, but underestimating high delays.

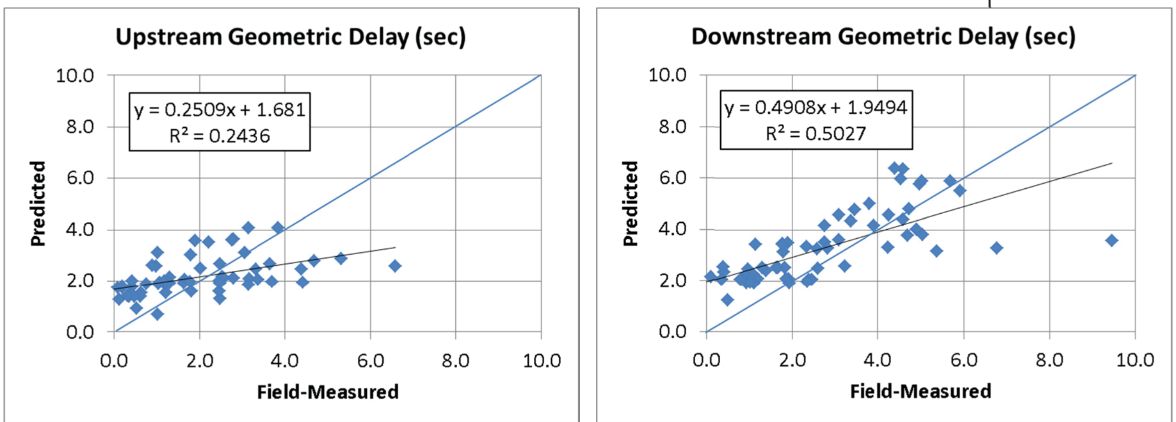


Exhibit 3-25: Internal Validation – Geometric Delay Models

3.4.1.4. *Impeded Delay*

The impeded delay model is the first model taking into account prevailing conditions and variations in volumes. The upstream impeded delay is predicted as a function of the FFS, the volume-to-capacity ratio, and the entering flow rate. The downstream impeded delay is predicted as a function of the FFS, the volume-to-capacity ratio, and three geometric terms (segment length, median length, and curb length).

In Exhibit 3-26, the plots of field-measured versus predicted data show a better fit for the downstream impeded delay (R^2 = 0.43) than the upstream model (R^2 = 0.11). The model predictions generally follow an increasing trend relative to the field data, but are further subject to much scatter. The higher variability of delays is expected, as the data are representative of a much wider range of operating conditions than the free-flow speed models.

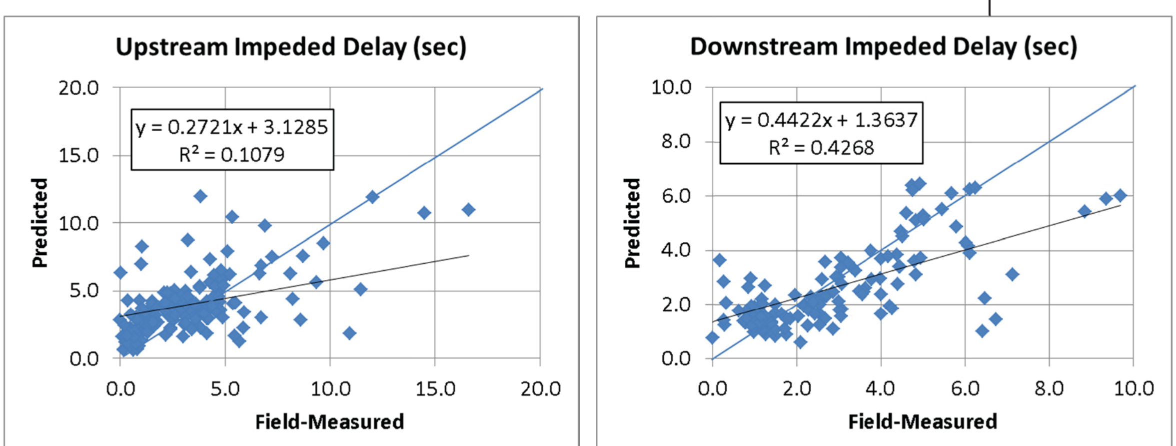


Exhibit 3-26: Internal Validation – Impeded Delay Models

3.4.1.5. *Average Travel Speed*

As the final model, the team predicted the average travel speed on the segment as a function of the FFS and the volume-to-capacity ratio. Similar to the impeded delay model, the average travel speed model takes into account the prevailing conditions on the corridor, with the speed reducing with increasing volume (increasing volume-to-capacity ratio).

In Exhibit 3-27, the data plots suggest a good fit with the field-observed values with R^2 statistics of 0.55 and 0.75 for upstream and downstream segments, respectively.

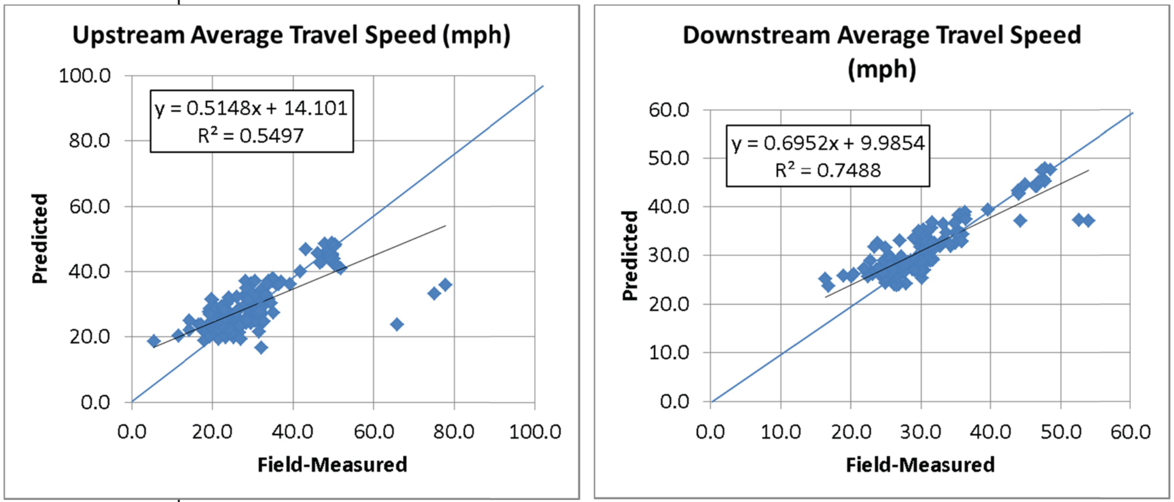


Exhibit 3-27: Internal Validation – Average Travel Speed Models

3.4.2. EXTERNAL MODEL VALIDATION AND APPLICATION

The previous section compared the estimates of the various sub-models with the field-observed data for seven of the roundabout corridors. The team collected additional data at two corridors that were not used in model development. In this section, the full methodology will be applied to these corridors in an effort to validate the overall methodology. The roundabout corridor methodology will be illustrated using the p.m. peak period in the northbound direction, which is the period with the highest travel time. Results for the a.m. peak period and the southbound a.m. and p.m. peaks will also be shown. The discussion presents the Old Meridian Street corridor in detail, followed by a presentation of the validation results for the Spring Mill Road corridor.

3.4.2.1. *Application to Old Meridian Corridor*

The roundabout corridor on Old Meridian Street consists of five intersections with four roundabouts and one signalized intersection over a distance of approximately 1.25 miles. Old Meridian Street runs roughly southwest-to-northeast, though it will be referred to within this document as south to north. Exhibit 3-28 describes the five intersections from south to north.

Evaluating the Performance of Corridors with Roundabouts

Number	Cross Street	# Legs	Control	ICD (ft)
1	Pennsylvania St.	4	Roundabout	220′
2	Carmel Dr.	4	Signal	n/a
3	Grand Blvd.	3	Roundabout	191′
4	Main St.	4	Roundabout	211′
5	Guilford Rd.	4	Roundabout	216′

Exhibit 3-28: Old Meridian Validation – Facility Summary

All four roundabouts have two lanes along Old Meridian Street. The signalized intersection at Carmel Dr. has two through lanes (one with a shared right) and an exclusive left-turn lane for both northbound and southbound approaches.

STEP A: GATHER INPUT DATA

The first step in the corridor analysis is to gather the necessary input data for the methodology. A summary of input needs for the roundabout corridor methodology by sub-model is shown in Exhibit 3-29. The data below are collected for each upstream and downstream sub-segment. The sub-segment length is defined from the midpoint of each segment to the yield line (upstream), and then from the yield line to the next segment midpoint (downstream).

Exhibit 3-29: Old Meridian Validation – Data Input Summary

Data Element	Unit	Sub-Model					
		Step B, E, F	Step C, D	Step G	Step H	Step I	Step L
		Free-Flow Speed	Roundabout Influence Area	Segment Running Time	Geometric Delay	Impeded Delay	Average Travel Speed
Speed Limit	mph	X					
Free-Flow Speed (FFS)	mph		X	X	X	X	
Circulating Speed (1)	mph		X		X		
Central Island Diameter	ft	X					
Inscribed Circle Diameter	ft				X		
Entering Volume	veh						
Roundabout Capacity (2)	veh						
Volume/Capacity Ratio	-					X	
Segment Length	ft	X		X		X	X
Median Length	ft					X	
Curb Length	ft					X	
Start-Up Lost Time (3)	sec			X			
Other	-	(4)		(5), (6), (7)			(8), (9)

(1) – Can be estimated from Inscribed Circle Diameter
(2) – Estimated from HCM 2010 Chapter 21 Roundabout Method
(3) – Defaulted to 2.5 seconds from HCM Chapter 17
(4) – Binary Check for Overlapping Roundabout Influence Areas
(5) – Proximity Adjustment Factor (f_v) from HCM Chapter 17
(6) – Delay due to turns at Access Points from HCM Chapter 17
(7) – Other delay on segment from HCM Chapter 17
(8) – Segment Running Time (from Step G)
(9) – Total Delay (from Steps H and I)

STEP B: DETERMINE FFS FOR SUB-SEGMENT

With all data collected, the FFS (S_f) is estimated for each upstream (US) and downstream (DS) segment from the speed limit (SL), central island diameter (CID), and segment length (L), using the following equation. In this initial step, it is assumed that the two adjacent ramp influence areas do not overlap (OL=0).

$$S_{f,US} = 15.1 + 0.0037*L + 0.43 * SL + 0.05 * CID - 4.73 * OL$$

$$S_{f,DS} = 14.6 + 0.0039*L + 0.48 * SL + 0.02 * CID - 4.43 * OL$$

For the signalized intersection at Old Meridian Road/Carmel Drive, the FFS estimation procedure from HCM 2010 Chapter 17 is used (Equation 17-2), as a function of a speed constant (S_0), a cross-section adjustment (f_{CS}), and an adjustment factor for access points (f_A).

Equation 3-10

$$S_{f0} = S_0 + f_{CS} + f_A$$

Source: HCM 2010 Equation 17-2

Using the equations above, the FFS for each sub-segment is estimated as shown in Exhibit 3-30.

Exhibit 3-30: Old Meridian Validation – Step B Results

Seg. #	Int. #	Type	US/DS	Speed Limit (mph)	Segment Length (ft)	CID (ft)	FFS (mph)
A	1	RBT	US	40	184	137	39.8
B	1	RBT	DS	40	763	137	39.5
B	2	Signal	US	40	763	n/a	42.2
C	2	Signal	DS	40	628	n/a	42.2
C	3	RBT	US	40	628	115	40.3
D	3	RBT	DS	40	968	115	39.9
D	4	RBT	US	40	1015	143	43.2
E	4	RBT	DS	40	1075	143	40.9
E	5	RBT	US	40	967	140	42.9
F	5	RBT	DS	40	581	140	38.9

STEP C: DETERMINE ROUNDABOUT INFLUENCE AREA LENGTH

In Step C, the roundabout influence area is estimated for each upstream and downstream segment from the FFS (S_f) and the circulating speed (S_c).

$$RIA_{US} = 165.9 + 13.8* S_f - 21.1*S_c$$

$$RIA_{DS} = -149.8 + 31.4* S_f - 22.5*S_c$$

In the equations above, the circulating speed can be approximated using the following equation as a function of the inscribed circle diameter (ICD).

$$S_c = 3.4614*(ICD/2)^{0.3673}$$

The RIA for each sub-segment is estimated as shown in Exhibit 3-31.

Exhibit 3-31: Old Meridian Validation – Step C Results

Seg #	Int. #	Type	US/ DS	FFS (mph)	ICD (ft)	Circle Speed (mph)	RIA (ft)
A	1	RBT	US	39.8	220	19.5	303.1
B	1	RBT	DS	39.5	220	19.5	653.2
B	2	Signal	US	42.2	n/a	n/a	n/a
C	2	Signal	DS	42.2	n/a	n/a	n/a
C	3	RBT	US	40.3	191	18.5	331.0
D	3	RBT	DS	39.9	191	18.5	686.7
D	4	RBT	US	43.2	211	19.2	355.9
E	4	RBT	DS	40.9	211	19.2	701.9
E	5	RBT	US	42.9	216	19.3	347.9
F	5	RBT	DS	38.9	216	19.3	635.8

STEP D: CHECK FOR OVERLAPPING INFLUENCE AREA

After the downstream and upstream influence areas of two adjacent roundabouts have been calculated, this step checks whether the sum of the two influence areas is greater than the segment length between roundabouts, as shown in Exhibit 3-32. For the first and last segment, the RIA is simply compared to the sub-segment length. If these conditions are met, the RIAs overlap and the FFS has to be re-calculated with OL=1.

Exhibit 3-32: Old Meridian Validation – Step D Results

Seg. #	Int. #	Type	US/DS	Segment Length (ft)	RIA (ft)	Overlap?
A	1	RBT	US	184	303.1	YES
B	1	RBT	DS	763	653.2	NO
B	2	Signal	US	763	n/a	NO
C	2	Signal	DS	628	n/a	NO
C	3	RBT	US	628	331.0	NO
D	3	RBT	DS	968	686.7	NO
D	4	RBT	US	1015	355.9	NO
E	4	RBT	DS	1075	701.9	NO
E	5	RBT	US	967	347.9	NO
F	5	RBT	DS	581	635.8	YES

STEP E: RECALCULATE FFS FOR OVERLAPPING INFLUENCE AREAS

In this step, the FFS equation from Step B is reapplied with OL=1 for roundabout segments with overlapping influence areas. On this corridor, the only “overlapping” influence areas are on the two sub-segments that are external to the corridor, where the roundabout influence areas are longer than the sub-segments, as shown in Exhibit 3-33.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-33: Old Meridian Validation – Step E Results

Seg. #	Int. #	Type	US/DS	FFS (mph) - Initial	Overlap?	FFS (mph) - Adjusted
A	1	RBT	US	39.8	YES	35.1
B	1	RBT	DS	39.5	NO	39.5
B	2	Signal	US	42.2	NO	42.2
C	2	Signal	DS	42.2	NO	42.2
C	3	RBT	US	40.3	NO	40.3
D	3	RBT	DS	39.9	NO	39.9
D	4	RBT	US	43.2	NO	43.2
E	4	RBT	DS	40.9	NO	40.9
E	5	RBT	US	42.9	NO	42.9
F	5	RBT	DS	38.9	YES	34.4

STEP F: SELECT CONTROLLING FFS

In this project, the FFS has been defined and measured at the midpoint between two roundabouts; therefore, one FFS is defined for a downstream segment and the next upstream segment. Consequently, the lower of these two separate estimates is selected as the controlling free-flow speed for both sub-segments, as shown in Exhibit 3-34.

Exhibit 3-34: Old Meridian Validation – Step F Results

Seg. #	Int. #	Type	US/DS	FFS (mph) - Adjusted	FFS (mph) - Controlling
A	1	RBT	US	35.1	35.1
B	1	RBT	DS	39.5	39.5
B	2	Signal	US	42.2	
C	2	Signal	DS	42.2	40.3
C	3	RBT	US	40.3	
D	3	RBT	DS	39.9	39.9
D	4	RBT	US	43.2	
E	4	RBT	DS	40.9	40.9
E	5	RBT	US	42.9	
F	5	RBT	DS	34.4	34.4

STEP G: DETERMINE SEGMENT RUNNING TIME

Using Equation 17-6 in HCM 2010, the segment running time is calculated, as shown in Exhibit 3-35. The equation uses the start-up lost time (l_1=2.5 for yield-control), segment length (L), start-up adjustment factor (f_x), free-flow speed (S_f), a proximity adjustment factor (f_v), additional delay from turns (d_{ap}), and other delay (d_{other}).

$$t_R = \frac{6.0 - l_1}{0.0025\ L} f_x + \frac{3{,}600\ L}{5{,}280\ S_f} f_v + \sum_{i=1}^{N_{ap}} d_{ap,i} + d_{\text{other}}$$

Equation 3-11

Source: HCM 2010 Equation 17-6

with,

$$f_x = \begin{cases} 1.00 & \text{(signalized or STOP-controlled through movement)} \\ 0.00 & \text{(uncontrolled through movement)} \\ \min[v_{th}/c_{th}, 1.00] & \text{(YIELD-controlled through movement)} \end{cases}$$

For the Old Meridian corridor, the additional delay from turns (d_{ap}) and other delay (d_{other}) were assumed to be zero because of minimal midsegment driveway activity and no other delay sources.

Exhibit 3-35: Old Meridian Validation – Step G Results

Seg. #	Int. #	Type	US/DS	Segment Length (ft)	FFS (mph) – Controlling	Running Time (sec)
A	1	RBT	US	184	35.1	6.3
B	1	RBT	DS	763	39.5	14.1
B	2	Signal	US	763	39.5	15.4
C	2	Signal	DS	628	40.3	13.3
C	3	RBT	US	628	40.3	11.5
D	3	RBT	DS	968	39.9	17.3
D	4	RBT	US	1015	39.9	18.2
E	4	RBT	DS	1075	40.9	18.8
E	5	RBT	US	967	40.9	16.9
F	5	RBT	DS	581	34.4	12.5

STEP H: DETERMINE GEOMETRIC DELAY

Next, the geometric delay is estimated for each upstream and downstream segment as a function of FFS (S_f), circulating speed (S_c), and inscribed circle diameter (*ICD*), as shown in Exhibit 3-36. The circulating speed can be estimated from the central island diameter as discussed above.

$Delay_{geom,US} = 1.57 + 0.11 * S_f - 0.21 * S_c$

$Delay_{geom,DS} = -2.63 + 0.09 * S_f + 0.625 * ICD * (1/S_c - 1/S_f)$

There is generally no geometric delay for a through movement at a signalized intersection; therefore, following guidance in HCM 2010 Chapter 17, the geometric delay is set to zero for Intersection 2.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-36: Old Meridian Validation – Step H Results

Seg. #	Int. #	Type	US/DS	FFS (mph) – Controlling	Circle Speed (mph)	ICD (ft)	Geom. Delay (sec)
A	1	RBT	US	35.1	19.5	220	1.3
B	1	RBT	DS	39.5	19.5	220	4.3
B	2	Signal	US	39.5	n/a	n/a	0.0
C	2	Signal	DS	40.3	n/a	n/a	0.0
C	3	RBT	US	40.3	18.5	191	2.0
D	3	RBT	DS	39.9	18.5	191	4.3
D	4	RBT	US	39.9	19.2	211	1.9
E	4	RBT	DS	40.9	19.2	211	4.5
E	5	RBT	US	40.9	19.3	216	2.0
F	5	RBT	DS	34.4	19.3	216	3.4

STEP I: DETERMINE IMPEDED DELAY

The next step is to estimate the impeded delay for each upstream and downstream roundabout segment under consideration of prevailing traffic conditions, as shown in Exhibit 3-37. The model is a function of free-flow speed (S_f), volume-to-capacity ratio (x), entering flow rate ($v_{entering}$), segment length (L), median length (L_{median}), and curb length (L_{curb}).

$$Delay_{imp,\ US} = -5.35 + 0.15*S_f + 42.50*x - 0.03 * v_{entering}$$

$$Delay_{imp,\ DS} = -2.65 + 0.07*S_f + 3.10*x + 0.0020 *L - 0.0010 *L_{median} + 0.0014 * L_{curb}$$

For the signalized intersection approach, the team used the HCM 2010 Chapter 17 methodology to estimate the control delay at the approach to the signalized intersection (upstream). In this case, Intersection 2 was assumed to operate under fixed-time control with random arrivals from the upstream and downstream roundabouts. Signal timing parameters were obtained from the City of Carmel, Indiana, to complete this analysis.

Exhibit 3-37: Old Meridian Validation – Step I Results

Seg. #	Int. #	Type	US/ DS	FFS (mph)	Vol. (veh)	Seg. Length (ft)	Median Length (ft)	Curb Length (ft)	Imp. Delay (sec)
A	1	RBT	US	35.1	906	184	184	50	0.8
B	1	RBT	DS	39.5	906	763	763	467	0.2
B	2	Sig.	US	39.5	n/a	n/a	n/a	n/a	26.3
C	2	Sig.	DS	40.3	n/a	n/a	n/a	n/a	0.0
C	3	RBT	US	40.3	826	628	628	111	0.0
D	3	RBT	DS	39.9	826	968	968	576	0.0
D	4	RBT	US	39.9	872	1015	1015	965	2.0
E	4	RBT	DS	40.9	872	1075	1075	845	0.0
E	5	RBT	US	40.9	736	967	967	761	2.7
F	5	RBT	DS	34.4	736	581	45	581	0.3

Evaluating the Performance of Corridors with Roundabouts

STEP J: AGGREGATE PERFORMANCE MEASURES

Up to this step, all calculations have been performed on the sub-segment level, where upstream and downstream sub-segments used different equations to estimate the various performance measures. At this stage, these performance measures need to be aggregated to the HCM 2010 Chapter 17 Urban Street Segments method. That methodology defines an urban street segment as the distance between two stop bars (or yield lines). In this project, that corresponds to the sum of one downstream and the next upstream segment.

Aggregation should be performed as the sum of the sub-segments for segment running time, geometric delay, and impeded delay, as shown in Exhibit 3-38. For the free-flow speed, the controlling FFS has already been selected to apply for the entire segment.

Exhibit 3-38: Old Meridian Validation – Step J Results

Seg. #	Int. #	Type	US/ DS	FFS (mph) – Controlling	Running Time (sec)	Geom. Delay (sec)	Imp. Delay (sec)
A	1	RBT	US	35.1	6.3	1.3	0.8
B	1	RBT	DS	39.5	29.5	4.3	26.5
B	2	Signal	US				
C	2	Signal	DS	40.3	24.8	2.0	0.0
C	3	RBT	US				
D	3	RBT	DS	39.9	35.5	6.2	2.0
D	4	RBT	US				
E	4	RBT	DS	40.9	35.7	6.5	2.7
E	5	RBT	US				
F	5	RBT	DS	34.4	12.5	3.4	3.4

STEP K: DETERMINE SEGMENT AVERAGE TRAVEL SPEED

The average travel speed for each segment is calculated from HCM 2010 Equation 17-14 below, as a function of segment length (L), running time (t_R), and total delay (d_t), as shown in Exhibit 3-39. The total delay is calculated as the sum of geometric and impeded delay.

Equation 3-12

$$S_{T,seg} = \frac{3{,}600\ L}{5{,}280\ (t_R + d_t)}$$

Source: HCM 2010 Equation 17-14

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-39: Old Meridian Validation – Step K Results

Seg. #	Int. #	Type	US/ DS	FFS (mph)	Run Time (sec)	Geom. Delay (sec)	Imp. Delay (sec)	Avg. Travel Speed (mph)
A	1	RBT	US	35.1	6.3	1.3	0.8	14.9
B	1	RBT	DS	39.5	29.5	4.3	26.5	17.3
B	2	Signal	US					
C	2	Signal	DS	40.3	24.8	2	0	32.0
C	3	RBT	US					
D	3	RBT	DS	39.9	35.5	6.2	2	30.9
D	4	RBT	US					
E	4	RBT	DS	40.9	35.7	6.5	2.7	31.0
E	5	RBT	US					
F	5	RBT	DS	34.4	12.5	3.4	3.4	20.5

STEP L: DETERMINE LEVELS OF SERVICE (LOS)

The LOS for an urban street segment is defined based on travel speed as a percent of free-flow speed, which is calculated by dividing the segment travel speed from Step K by the controlling segment free-flow speed from Step F. The LOS is then obtained from the thresholds given in HCM 2010 Exhibit 17-2, which is shown here in Exhibit 3-40.

Exhibit 3-40: Urban Street LOS Table (HCM 2010 Exhibit 17-2)

Travel Speed as a Percentage of Base Free-Flow Speed (%)	LOS by Volume-to-Capacity Ratio[a] ≤1.0	> 1.0
>85	A	F
>67–85	B	F
>50–67	C	F
>40–50	D	F
>30–40	E	F
≤30	F	F

Note: (a) Volume-to-capacity ratio of through movement at downstream boundary intersection.

Evaluating the Performance of Corridors with Roundabouts

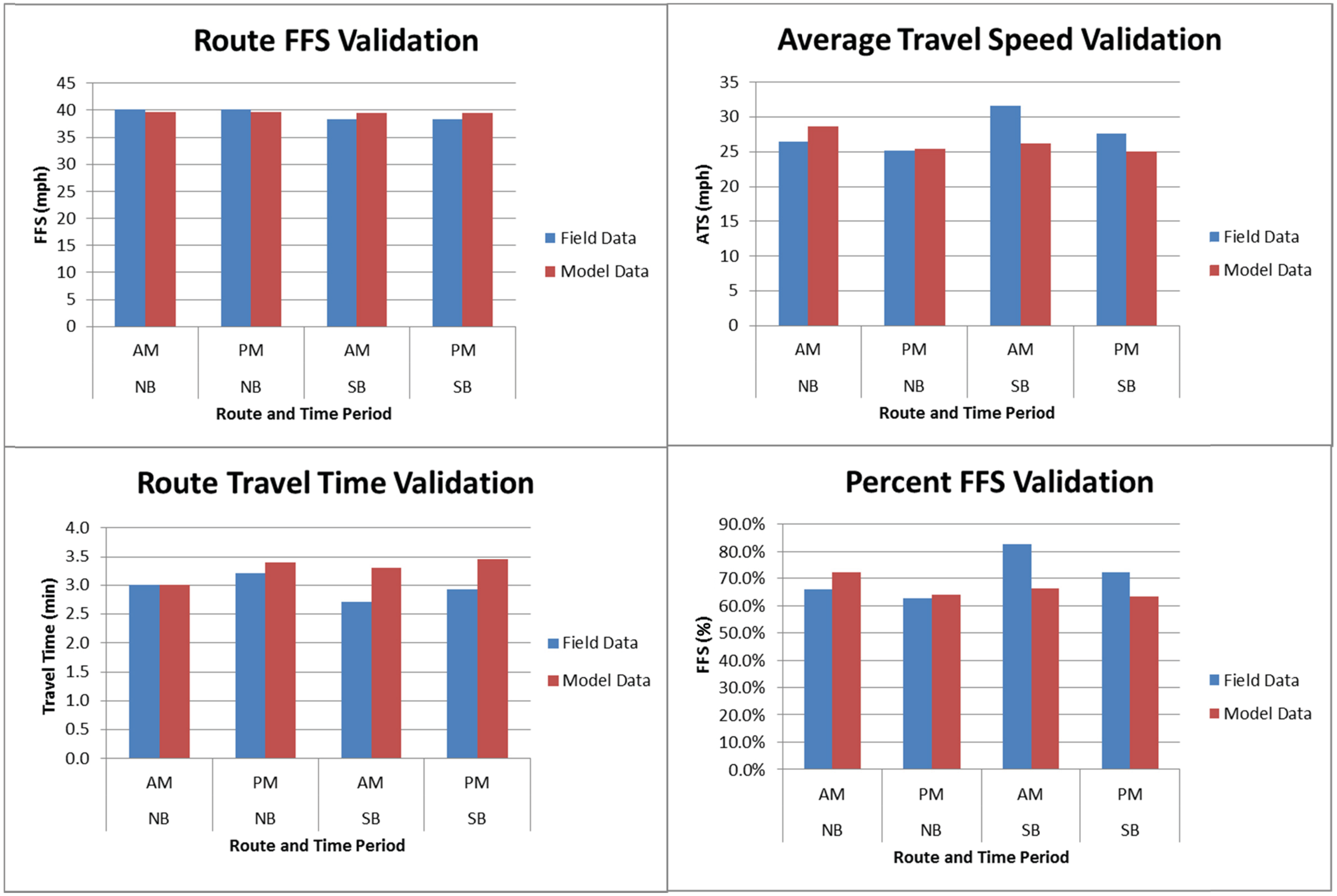
Exhibit 3-41: Old Meridian Validation – Step L Results

Seg. #	Int. #	Type	US/ DS	FFS (mph)	Avg. Travel Speed (mph)	Percent FFS	LOS
A	1	RBT	US	35.1	14.9	42.6%	D
B	1	RBT	DS	39.5	17.3	43.7%	D
B	2	Signal	US				
C	2	Signal	DS	40.3	32.0	79.3%	B
C	3	RBT	US				
D	3	RBT	DS	39.9	30.9	77.5%	B
D	4	RBT	US				
E	4	RBT	DS	40.9	31.0	75.8%	B
E	5	RBT	US				
F	5	RBT	DS	34.4	20.5	59.7%	C

3.4.3. OLD MERIDIAN ROUTE VALIDATION

The validation results above can further be aggregated to the roundabout corridor level, and those estimates compared to the field-observed data. Exhibit 3-42 shows the facility-average FFS, the total travel time, the average travel speed, and the percent FFS for the entire corridor compared to the field estimates. In addition to the results of the northbound p.m. peak data, the exhibit shows the northbound a.m. peak results, as well as the results for northbound a.m. and p.m. peaks.

Exhibit 3-42: Old Meridian Validation – Summary Results



Evaluating the Performance of Corridors with Roundabouts

The validation results suggest a close match of the predicted FFS for both northbound and southbound, with an error of 0.6 mph and 1.2 mph, respectively. For the average travel speed estimation, the northbound results for the a.m. and p.m. peaks match the field-observed data with an error of 2.1 mph (8.0%) and 0.2 mph (0.8%), respectively. For the southbound, the average travel speed estimates are lower than the field-observed data by 5.4 mph (17.0%) and 2.5 mph (9.2%) for the a.m. and p.m. peaks, respectively. This difference in average travel speed translates to a difference in travel times of 0.0 and 0.2 minutes for northbound a.m. and p.m. peaks, and 0.6 and 0.5 minutes for the southbound routes.

The final validation results are shown in Exhibit 3-43 in terms of the estimates of percent FFS and the corridor LOS. The results show the method predicted the correct LOS within one letter grade for all four tested routes. The validation generally performed better for the northbound route than the southbound route.

Exhibit 3-43: Old Meridian Validation – Route Validation Results

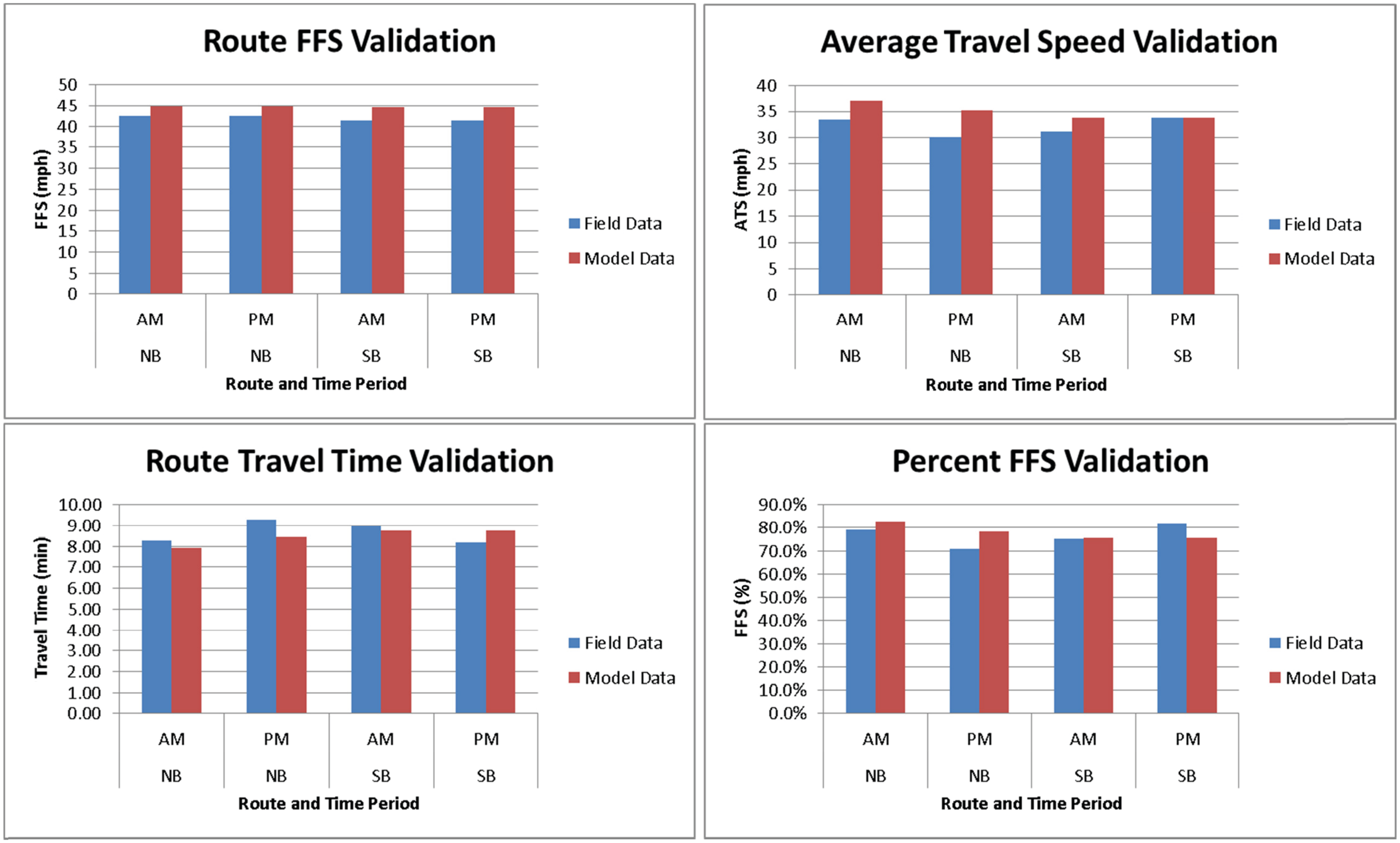
Route	Time	Field Data		Model Data	
		%FFS	LOS	%FFS	LOS
NB	AM	66%	C	72%	B
NB	PM	63%	C	64%	C
SB	AM	83%	B	66%	C
SB	PM	72%	B	63%	C

3.4.4. SPRING MILL ROUTE VALIDATION

Similar to the Old Meridian corridor, the team also performed the overall route validation for the Spring Mill corridor in Carmel, Indiana. The details are not shown here, as the steps are largely the same as described above. Further, the Spring Mill corridor does not contain any interim signals, which simplifies the analysis.

The validation results have been aggregated to the roundabout corridor level, and those estimates compared to the field-observed data. Exhibit 3-44 shows the facility-average FFS, the total travel time, the average travel speed, and the percent FFS for the entire corridor compared to the field estimates.

Exhibit 3-44: Spring Mill Route Validation – Summary Results



Evaluating the Performance of Corridors with Roundabouts

The Spring Mill corridor validation shows a close match between the model and field estimates. The model slightly overestimated the FFS and the average travel speed by about 2 to 5 mph across the four analyzed routes. This translated to a slight underestimation of travel time for three of the routes by 0.2 to 0.8 minutes. However, the resulting percent FFS measure proved to be within 10 percent of the field-observed data for all routes, which is explained because the error in FFS and average travel speed was in the same direction.

The final route validation results are shown in Exhibit 3-45. It is evident the method correctly predicted the route LOS for all four cases.

Exhibit 3-45: Spring Mill Validation – Route Validation Results

Route	Time	Field Data		Model Data	
		%FFS	LOS	%FFS	LOS
NB	AM	79%	B	83%	B
NB	PM	71%	B	78%	B
SB	AM	75%	B	76%	B
SB	PM	82%	B	76%	B

3.5. EQUIVALENT NON-ROUNDABOUT CORRIDORS

A typical corridor operations evaluation involves comparing various control treatments at key intersections, typically roundabout versus signalized intersections. To gain insight into typical comparisons, the research team developed a non-roundabout alternative for each of the study corridors. In most cases, the non-roundabout alternative has signals in place of roundabouts. In a few cases noted in Section 3.5.2, stop-controlled intersections were used in place of roundabouts. These comparisons are intended to provide insight from an operational perspective on the potential strengths and weaknesses that roundabouts and signalized intersections bring to a corridor.

Because the corridors under study have already been built into a roundabout configuration, it is possible to use field-based measurements of operational performance of the roundabout configuration and compare them to a predicted performance of the equivalent signalized corridor. These types of comparisons involve assumptions regarding the configuration of the equivalent non-roundabout corridor (e.g., control, phasing, lane configurations) and the modeling tools employed (e.g., HCM-based analytical analysis or simulation).

The comparison presented here, like the data collected in the field for this project, is operations-focused. A comprehensive evaluation of corridor alternatives, such as the process described in the Corridor Comparision Document (Appendix A), would incorporate many other elements, including safety. The *Highway Safety Manual* (HSM) contains safety performance functions (SPFs) for signalized and two-way stop-controlled intersections (AASHTO 2010). SPFs estimate the number of crashes expected in future years based on annual average daily traffic (AADT) and the basic geometric configuration. The HSM and a more recent study by Gross et al. (2012) provide crash modification

Evaluating the Performance of Corridors with Roundabouts

factors (CMFs) for converting signalized intersections and stop-controlled intersections into roundabouts. Through the use of SPFs and CMFs, the safety performance of non-roundabout and roundabout alternatives can be compared on a planned corridor or on an existing corridor where a retrofit is being considered. Generally speaking, CMFs indicate that replacing a signalized or two-way stop-controlled intersection with a roundabout will reduce total and injury crashes.

3.5.1. METHODOLOGY

The methodology for the operations comparison to equivalent signalized corridors consisted of the following steps:

1. Create an equivalent non-roundabout corridor configuration.
2. Use a simplified simulation process to estimate the performance of the equivalent non-roundabout corridor for the routes that were field-measured for the roundabout corridor.
3. Compare the estimated performance of the selected routes between the equivalent non-roundabout corridor and the roundabout corridor.

3.5.2. CORRIDOR CONFIGURATIONS

Lane configurations, signal phasing, and coordinated signal timing parameters were developed to produce an equivalent non-roundabout alternative that the research team believes could have been realistically considered in an alternatives analysis. Details for each equivalent non-roundabout corridor configuration are provided in Appendix O. For maps of the corridors, refer to the site reports in Appendices B through J.

In general, each equivalent non-roundabout corridor was developed by replacing each roundabout with a signalized intersection so a pure roundabout-only versus signal-only comparison could be made. However, several corridor-specific exceptions were made to adapt to local conditions:

- La Jolla Boulevard: The equivalent non-roundabout corridor assumes three two-way stop-controlled intersections (Colima Street, Midway Street, and Camino de la Costa) and two signalized intersections (Bird Rock Avenue and Forward Street). Based on the data obtained by the research team, the three unsignalized intersections would function acceptably as stop-controlled intersections and would not meet MUTCD signal warrants.
- Old Meridian Street: The existing roundabout corridor contains a signalized intersection (Carmel Drive) within the series of roundabouts. Therefore, the field-measured roundabout performance includes the performance of a signalized intersection.
- SR 539: For the equivalent non-roundabout corridor, only the central two intersections were assumed to be signalized. The intersections on each end were assumed to be two-way stop-controlled due to not meeting MUTCD signal warrants.
- Golden Road: For the equivalent non-roundabout corridor, three intersections (Utah Street, Lunnonhaus Drive, and Jackson Street/Ford

Street) were modeled as two-way stop-controlled intersections. Utah Street and Lunnonhaus Drive were previously two-way stop-controlled intersections, and Jackson Street and Ford Street form a one-way couplet extension of Golden Road. The remaining intersections (Ulysses Street and Johnson Road) were modeled as signalized intersections.

3.5.3. CORRIDOR TRAVEL-TIME EXTRACTION

Each corridor was analyzing using six routes:

- Two routes conducted end to end.
- Four routes involving left-turn movements at a selected intersection within the corridor. These routes capture the effect of major-street left turns from the arterial and minor-street left turns onto the arterial.

A simplified method was employed to conduct the analysis of the equivalent non-roundabout corridor. To generate travel time from the signalized/stop-controlled intersection model that could be compared against the field data under roundabout control, the project team aggregated the intersection through-movement average travel times reported in SimTraffic for each intersection along the corridor in each direction. This process was necessitated by the fact that SimTraffic does not provide a feature to collect the travel time for a segment or series of segments with a user-specified origin/start point and destination/end point (such as the routes including a left turn onto or off the corridor). As SimTraffic is a stochastic simulation model, five runs were performed and the results were averaged to derive measures of effectiveness (MOEs) for the corridor under signalized/stop control.

As a result of applying the performance measure aggregation methodology, the computed route travel time is similar to but not the same as taking the travel time of only those vehicles that travel the entire route. In other words, the travel time reported by SimTraffic is an average value for all vehicles traveling on the segment, including vehicles that are not traveling the route being studied. The methodology used here assumes the difference between the travel times of vehicles following and not following the studied route is negligible for the purpose of the comparison.

Exhibit 3-46 provides a summary of travel time and intersection delay time comparisons for selected routes for each corridor. The table is sorted by travel-time difference. The first route had the largest percentage decrease in travel time with the roundabouts. These values are extracted from the detailed analysis contained in Appendix O.

Exhibit 3-46: Summary of Travel Time and Intersection Approach Delay Comparisons, Sorted by Travel Time Difference

Corridor	Route Route beginning with: - 1 and 2 are through routes -3 and 5 are left turns departing the corridor -4 and 6 are left turns entering the corridor	Equivalent Signalized Route Travel Time (s)	Roundabout Travel Time (s)	**Travel-Time Difference**	Equivalent Signalized Route Int. Approach Delay (s)	Roundabout Route Int. Approach Delay (s)	Intersection Approach Delay Difference	Equivalent Signalized Route Non-Intersection Time (s)	Roundabout Route Non-Intersection Time (s)	Non-Intersection Time Difference
MD 216	3. East-South, left turn at #4 NB US-29	103.7	48.0	**-54%**	72.3	12.0	-83%	31.4	36.0	15%
Spring Mill (AM)	5. North-East, left turn at #5 131st	295.1	166.8	**-43%**	189.1	61.2	-68%	106.0	105.6	0%
Spring Mill (PM)	4. West-North, left turn from #6 136th	139.1	81.0	**-42%**	76.9	23.4	-70%	62.2	57.6	-7%
MD 216	4. South-West, left turn from #4 NB US-29	191.4	114.0	**-40%**	101.2	24.0	-76%	90.2	90.0	0%
MD 216	5. West-North, left turn at #2 Maple Lawn	77.5	48.0	**-38%**	38.2	12.0	-69%	39.3	36.0	-8%
Avon Road	3. South-West, left turn at #3 Beaver Creek	104.0	68.4	**-34%**	55.5	20.4	-63%	48.5	48.0	-1%
Old Meridian (PM)	4. West-North, left turn from #3 Main	104.0	68.4	**-34%**	52.6	21.0	-60%	51.4	47.4	-8%
Spring Mill (AM)	4. West-North, left turn from #6 136th	135.5	90.0	**-34%**	77.4	32.4	-58%	58.1	57.6	-1%
Spring Mill (PM)	6. East-South, left turn from #5 131st	577.6	390.0	**-32%**	248.4	69.0	-72%	329.2	321.0	-2%
Old Meridian (AM)	4. West-North, left turn from #3 Main	92.6	64.8	**-30%**	52.6	17.4	-67%	40.0	47.4	19%
Avon Road	6. East-South, left turn from #4 Benchmark	88.3	62.4	**-29%**	48.9	25.8	-47%	39.4	36.6	-7%
Avon Road	4. West-North, left turn from #3 Beaver Creek	99.4	72.0	**-28%**	63.9	32.4	-49%	35.5	39.6	12%
SR 67	5. West-North, left turn at #5 US 9	175.9	127.8	**-27%**	75.1	33.6	-55%	100.8	94.2	-7%
MD 216	6. North-East, left turn from #2 Maple Lawn	188.2	138.0	**-27%**	103.8	42.0	-60%	84.4	96.0	14%
Golden Road	4. West-North, left turn from #4 Johnson	122.4	91.2	**-25%**	30.1	9.6	-68%	92.3	81.6	-12%
Spring Mill (PM)	5. North-East, left turn at #5 131st	193.5	145.2	**-25%**	88.7	39.6	-55%	104.8	105.6	1%
MD 216	2. East-West	189.8	144.0	**-24%**	86.1	30.0	-65%	103.7	114.0	10%
Spring Mill (AM)	2. North-South	707.9	538.2	**-24%**	304.9	139.8	-54%	403.0	398.4	-1%
Avon Road	5. North-East, left turn at #4 Benchmark	100.5	79.2	**-21%**	53.2	35.4	-33%	47.3	43.8	-7%
Old Meridian (AM)	6. East-South, left turn from #2 Grand	119.5	94.8	**-21%**	56.0	37.2	-34%	63.5	57.6	-9%
Spring Mill (AM)	6. East-South, left turn from #5 131st	490.8	394.2	**-20%**	174.8	73.2	-58%	316.0	321.0	2%
SR 67	4. South-West, left turn from #5 US 9	189.2	153.0	**-19%**	81.4	51.0	-37%	107.8	102.0	-5%
SR 67	6. North-East, left turn from #5 US 9	151.1	124.2	**-18%**	72.1	18.0	-75%	79.0	106.2	34%
SR 539	6. East-South, left turn from #3 Wiser Lake	273.7	228.0	**-17%**	66.7	46.8	-30%	207.0	181.2	-12%
MD 216	1. West-East	186.5	156.0	**-16%**	81.8	42.0	-49%	104.7	114.0	9%
SR 539	4. West-North, left turn from #2 Pole	245.8	207.0	**-16%**	54.6	52.2	-4%	191.2	154.8	-19%
Avon Road	2. North-South	125.9	107.4	**-15%**	56.3	40.8	-28%	69.6	66.6	-4%
Old Meridian (AM)	5. North-East, left turn at #2 Grand	117.2	100.8	**-14%**	20.0	18.0	-10%	97.2	82.8	-15%

Exhibit 3-46 Continued

Corridor	Route Route beginning with: - 1 and 2 are through routes -3 and 5 are left turns departing the corridor -4 and 6 are left turns entering the corridor	Equivalent Signalized Route Travel Time (s)	Roundabout Travel Time (s)	**Travel-Time Difference**	Equivalent Signalized Route Int. Approach Delay (s)	Roundabout Route Int. Approach Delay (s)	Intersection Approach Delay Difference	Equivalent Signalized Route Non-Intersection Time (s)	Roundabout Route Non-Intersection Time (s)	Non-Intersection Time Difference
Spring Mill (PM)	2. North-South	558.4	490.2	**-12%**	154.8	91.8	-41%	403.6	398.4	-1%
Spring Mill (AM)	3. South-West, left turn at #6 136th	484.6	435.0	**-10%**	139.6	78.6	-44%	345.0	356.4	3%
Old Meridian (PM)	5. North-East, left turn at #2 Grand	111.6	100.2	**-10%**	21.6	17.4	-19%	90.0	82.8	-8%
Golden Road	6. East-South, left turn from #4 Johnson	110.2	99.0	**-10%**	33.2	31.2	-6%	77.0	67.8	-12%
Avon Road	1. South-North	133.8	120.6	**-10%**	64.1	55.2	-14%	69.7	65.4	-6%
Spring Mill (PM)	3. South-West, left turn at #6 136th	558.6	508.8	**-9%**	191.7	152.4	-21%	366.9	356.4	-3%
Spring Mill (PM)	1. South-North	602.4	556.8	**-8%**	203.0	166.2	-18%	399.4	390.6	-2%
SR 539	3. South-West, left turn at #2 Pole	153.6	142.8	**-7%**	12.7	32.4	155%	140.9	110.4	-22%
La Jolla Boulevard	5. West-North, left turn from #4 Bird Rock	75.0	70.0	**-7%**	29.5	6.0	-80%	45.5	64.0	41%
Golden Road	5. North-East, left turn at #4 Johnson	107.1	100.8	**-6%**	9.2	12.6	37%	97.9	88.2	-10%
Old Meridian (PM)	1. South-North	202.5	192.0	**-5%**	75.3	74.4	-1%	127.2	117.6	-8%
SR 67	1. West-East	246.1	233.4	**-5%**	85.2	50.0	-41%	160.9	183.4	14%
Old Meridian (AM)	2. North-South	164.4	162.6	**-1%**	37.3	38.4	3%	127.1	124.2	-2%
Old Meridian (AM)	3. South-West, left turn at #3 Main	139.5	138.0	**-1%**	40.0	45.6	14%	99.5	92.4	-7%
SR 539	5. North-East, left turn at #3 Wiser Lake	122.5	121.2	**-1%**	11.1	25.8	132%	111.4	95.4	-14%
SR 539	1. South-North	327.9	327.0	**0%**	30.2	74.4	146%	297.7	252.6	-15%
SR 539	2. North-South	321.8	322.8	**0%**	23.4	63.6	172%	298.4	259.2	-13%
Golden Road	1. South-North	161.7	163.2	**1%**	19.9	28.2	42%	141.8	135.0	-5%
Spring Mill (AM)	1. South-North	492.6	497.4	**1%**	108.5	106.8	-2%	384.1	390.6	2%
Old Meridian (PM)	3. South-West, left turn at #3 Main	153.7	156.0	**1%**	51.1	63.6	24%	102.6	92.4	-10%
Golden Road	2. North-South	161.1	167.4	**4%**	22.5	30.0	33%	138.6	137.4	-1%
Old Meridian (AM)	1. South-North	170.7	180.6	**6%**	52.4	63.0	20%	118.3	117.6	-1%
Golden Road	3. South-West, left turn at #4 Johnson	91.9	98.4	**7%**	24.2	27.6	14%	67.7	70.8	5%
Old Meridian (PM)	6. East-South, left turn from #2 Grand	97.8	105.0	**7%**	50.5	47.4	-6%	47.3	57.6	22%
SR 67	2. East-West	234.7	253.2	**8%**	79.4	69.6	-12%	155.3	183.6	18%
SR 67	3. East-South, left turn at #5 US 9	121.6	138.6	**14%**	40.7	22.8	-44%	80.9	115.8	43%
Old Meridian (PM)	2. North-South	142.7	175.8	**23%**	28.0	51.6	84%	114.7	124.2	8%
La Jolla Boulevard	3. East-South, left turn from #1 Colima	41.4	54.0	**30%**	11.8	6.0	-49%	29.6	48.0	62%
La Jolla Boulevard	4. South-West, left turn at #4 Bird Rock	88.9	126.0	**42%**	32.8	6.0	-82%	56.1	120.0	114%

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-46 Continued

Corridor	Route Route beginning with: - 1 and 2 are through routes -3 and 5 are left turns departing the corridor -4 and 6 are left turns entering the corridor	Equivalent Signalized Route Travel Time (s)	Roundabout Travel Time (s)	**Travel-Time Difference**	Equivalent Signalized Route Int. Approach Delay (s)	Roundabout Route Int. Approach Delay (s)	Intersection Approach Delay Difference	Equivalent Signalized Route Non-Intersection Time (s)	Roundabout Route Non-Intersection Time (s)	Non-Intersection Time Difference
La Jolla Boulevard	2. North-South	110.7	162.0	**46%**	21.8	24.0	10%	88.9	138.0	55%
La Jolla Boulevard	1. South-North	110.2	162.0	**47%**	21.4	24.0	12%	88.8	138.0	55%
La Jolla Boulevard	6. North-East, left turn at #1 Colima	106.7	192.0	**80%**	24.7	36.0	46%	82.0	156.0	90%

Int. = Intersection

From this diverse set of corridors, the research team made a number of observations:

- Neither roundabout nor signalized/stop-controlled corridor configurations consistently result in reduced travel time or intersection delays. Of the 20 *through* route combinations analyzed, approximately half resulted in lower travel time under a roundabout configuration and approximately half resulted in lower travel time under non-roundabout configuration.
- The corridors having better travel times under a roundabout configuration (MD 216, Spring Mill Road, Avon Road) also are notable for irregular intersection spacing.
- Corridors that can use two-way stop-controlled intersections in the place of roundabouts or signals generally produce better end-to-end travel times, even if intersection delays are lower under a roundabout configuration. The corridor having the lowest travel times under an equivalent non-roundabout configuration (La Jolla Boulevard) was designed with mixed signalized/unsignalized control. As a result, end-to-end travel times are more favorable under an equivalent non-roundabout configuration, even though the roundabout configuration resulted in lower intersection delays.
- The corridor analyzed with large intersection spacing and higher speeds (SR 539) showed virtually no difference in travel time between the two alternatives, despite the observation that the intersection delay increases in a roundabout configuration.
- The analysis becomes more illuminating when including the travel time runs involving left turns to and from the arterial street. Of the 60 total corridor-route combinations analyzed, approximately three-quarters (44) of the corridor-route combinations resulted in lower travel time under a roundabout configuration than under an equivalent signalized configuration. Approximately one-third (21) of the analyzed combinations resulted in reduced travel time of 20 percent or more. In addition, approximately three-quarters (44) of the corridor-route combinations resulted in lower intersection approach delay for the routes studied under a roundabout configuration.

Evaluating the Performance of Corridors with Roundabouts

- Anecdotal observations about specific corridors or groups of corridors may explain some of the variability in the results:
    - Approach delay was lower with roundabouts for all intersections in both major-street directions except for SR 539.
    - Through-route travel time (average of both directions) increased with roundabouts on La Jolla Boulevard, Old Meridian Street, and Golden Road; decreased with roundabouts on MD 216, Spring Mill Road, Avon Road, and SR 67; and remained virtually unchanged on SR 539. The corridors on which through travel times decreased with roundabouts have irregular intersection spacing, changes in land use, and an interchange. They also have the highest peak hour major-street and side-street volumes among the corridors studied. SR 539 has the longest intersection spacing and highest speed limit; changes in intersection performance have a lower effect on overall corridor operations on SR 539 than on other corridors.
    - Travel time for routes with a left turn off the major street (average of both directions) increased with roundabouts on La Jolla Boulevard; decreased on MD 216, Old Meridian Street, Spring Mill Road, SR 539, Avon Road, and SR 67; and remained virtually unchanged on Golden Road. It is noted that Golden Road has the lowest 12-hour (7 a.m. to 7 p.m.) volume of all the corridors studied.
    - Travel time for routes with a left turn onto the major street (average of both directions) increased with roundabouts on La Jolla Boulevard and decreased on MD 216, Old Meridian Street, Spring Mill Road, SR 539, Golden Road, Avon Road, and SR 67.
    - The La Jolla Boulevard corridor performs quite differently from the other corridors studied in this project. It is the most urban of the corridors studied, with considerable pedestrian, bicycle, and on-street parking activity. As a result, through vehicular traffic experiences more friction than was observed for other corridors. La Jolla Boulevard has the second-shortest average intersection spacing (Avon Road has the shortest spacing), the lowest speed limit (La Jolla Boulevard and Avon Road are both 25 mph), and the highest pedestrian volume. It is the only corridor with angled on-street parking. Over half of the length of the corridor lies within roundabout influence areas, or the areas upstream and downstream of roundabouts where drivers are accelerating or decelerating. Three of the five roundabouts on La Jolla Boulevard were modeled as two-way stop-control in the equivalent corridor because they did not meet signal warrants.

Based on these observations, the research team believes a case-by-case evaluation is necessary to determine what is preferred operationally for a given corridor. The evidence suggests a roundabout corridor has a good likelihood of improving travel-time performance, but site-specific conditions may favor

Evaluating the Performance of Corridors with Roundabouts

signalized (or stop-controlled) operation. The Corridor Comparison Document in Appendix A provides a framework for operations evaluations such as this in the context of a comprehensive corridor alternative analysis.

3.6. SUMMARY

This chapter highlighted the key methods, observations, and conclusions of the modeling effort for this project. In this section, the five modeling results are summarized for roundabout influence area, geometric delay, free-flow speed, average running speed, and impeded delay.

The team began the study of roundabout corridors by developing a framework to investigate the effects of corridor design and operation on the speeds and delay incurred at the individual roundabouts. The first three models (RIA, FFS, and geometric delay) used a dataset with only geometric and speed data that did not change with the time of day, while a separate dataset containing time-of-day data was used for the last two types of models (average travel speed and impeded delay).

3.6.1. ROUNDABOUT INFLUENCE AREA MODEL

The team investigated the spatial extent of each roundabout's influence on the adjacent segments by determining the length of the segment along which geometric delay due to the presence of the roundabout is incurred. This length was denoted as the *Roundabout Influence Area Length*. The RIA was calculated using the *unimpeded* speed profile of each roundabout corridor and determining where the speed fell below free-flow speed. Segments that the team suspected contained overlapping RIAs were excluded from the dataset.

The team developed several regression models for the upstream and downstream portions of the RIA and found that it is primarily related to the free-flow speed along the segment as well as the circulating speed (i.e., the minimum unimpeded speed) within the roundabout. For a given segment, as the free-flow speed increases, the RIA length predicted by the model also increases, but this length decreases as the circulating flow in the roundabout adjacent to the segment increases. This is intuitive because it suggests roundabouts necessitating a greater decrease in speed (as indicated by a large difference between the free-flow speed and circulating speed) should be associated with a longer RIA. While the downstream RIA models could explain up to 71 percent of the variability in the data, the upstream data were considerably more variable, and the preferred models only explained 29 to 38 percent of the variability in the data.

3.6.2. GEOMETRIC DELAY MODEL

The HCM Chapter 21 methodology for determining roundabout delay only considers control delay and does not currently estimate geometric delay, which is incurred due to the presence of the roundabout and affects all vehicles regardless of the level of congestion. The team calculated the geometric delay for each upstream and downstream segment by taking the difference between

Evaluating the Performance of Corridors with Roundabouts

unimpeded and free-flow travel times along each segment. Again, segments with overlapping RIAs were excluded from the dataset.

Although the data were highly variable, the team found the geometric delay could be partially explained by the free-flow speed along the segment and the circulating speed within the roundabout. These relationships were similar to those in the RIA models in that a higher free-flow speed along a segment would cause the model to predict a higher geometric delay, but a higher circulating speed within the adjacent roundabout would lower the predicted geometric delay. This is also intuitive in that drivers should experience more geometric delay at a roundabout necessitating a greater decrease in speed. While the downstream geometric delay models could explain up to 66 percent of the variability in the data, the upstream data were considerably more variable; these models could only account for 30 to 40 percent of the variability in the data.

3.6.3. FREE-FLOW SPEED MODEL

The third model aimed to predict the free-flow speed along each roundabout segment based on planning and geometric data, before any traffic or operational data were considered. The team estimated the free-flow speed within each segment by using the unimpeded speed profiles and examining the speeds in-between the roundabouts. Several of the roundabouts were so closely spaced that the vehicles may not have been able to accelerate back to free-flow speed (due to overlapping RIAs), so segments with this quality were denoted *overlapping segments*.

The team found the free-flow speed was primarily related to the segment length, the posted speed limit along the segment, and the central island diameter of the adjacent roundabout, as well as whether the segment was an overlapping segment. Specifically, the free-flow speed was found to increase with the posted speed, segment length, and central island diameter, but the models assigned a significant penalty (minus 4.5 to 5.0 mph) to the free-flow speed of these overlapping segments. The free-flow speed models had the strongest fit of any of the models here; they explained more than 90 percent of the variability in the data.

3.6.4. IMPEDED DELAY MODEL

The team investigated the average impeded delay incurred to each driver at each roundabout by calibrating a delay model for the time-of-day dataset. The team determined the impeded delay for each segment by taking the difference between the average travel time and the free-flow travel time.

The resulting models indicated that, for the upstream segments, the impeded delay is primarily related to the free-flow speed along the segment as well as the entering flow and volume-to-capacity ratio for the downstream roundabout. The upstream models suggested drivers will experience more delay at a roundabout with a high level of congestion (indicated by a high volume-to-capacity ratio), but a roundabout with a higher level of entering flow will incur a lower amount of delay. This latter relationship may seem counterintuitive but

Evaluating the Performance of Corridors with Roundabouts

should be interpreted along with the volume-to-capacity ratio term. Like the geometric delay model, a segment with a higher free-flow speed is associated with greater impeded delay. For the downstream segments, the free-flow speed, segment length, curb length, and volume-to-capacity ratio all increased the delay, but an increase in median length led to a decrease in the model-predicted delay. Again, this last relationship may be due to the sample size or range of median lengths. The resulting impeded delay models explained 55 to 70 percent of the variability in the data.

3.6.5. AVERAGE TRAVEL SPEED MODEL

Finally, the team investigated the average speed along the sub-segments of a roundabout corridor, under consideration of traffic characteristics such as the level of congestion. The average travel speed along each sub-segment was calculated for the upstream and downstream segments to calibrate several models. This model is an optional step in the implementation of these models in HCM 2010 Chapter 17, as that procedure already contains a step to estimate the average travel speed under consideration of free-flow speed, delays, and other factors. But the model developed here may be useful as a roundabout-specific verification of the Chapter 17 model, which was calibrated from signalized intersection data.

The team found the average travel speed was chiefly related to the free-flow speed and volume-to-capacity ratio of the downstream roundabout. Thus, a segment with a higher free-flow speed would experience a higher average speed, but an increase in volume-to-capacity ratio would lower the average speed within the segment. These models explained 75 to 85 percent of the variability in the data.

3.6.6. VALIDATION

After completing the model development of all predictive models, the team performed two types of validation to the field-observed data. The validation exercise was intended to document how well the various sub-models match the field data, as well as verify that the proposed methodology results in satisfactory performance results.

First, the results of the various sub-models were validated internally against the field-observed data for the seven roundabout corridors used in model development. A comparison of the model predictions to field-observed performance data generally showed that the free-flow speed model explained over 90 percent of the variability in the data for both upstream and downstream segments. The roundabout influence area models predicted 57 percent of the downstream variability, but only 24 percent of the upstream variability, which is attributed to high variability and some outliers in the field-observed data. A similar model fit was observed for the geometric delay models, with downstream prediction ability being higher than upstream geometric delays. This may be explained by more consistent circulating and acceleration behavior across sites, with more variable deceleration profiles. This trend is similarly replicated for the impeded delay and average travel speed models. While not

Evaluating the Performance of Corridors with Roundabouts

used in the proposed methodology, the direct estimation of average travel speed is a viable model alternative to the HCM 2010 Chapter 17 method, explaining 55 percent and 75 percent of the upstream and downstream variability in average travel speed, respectively.

Second, the team presented an external validation of the methodology through application to the Old Meridian Street and Spring Mill Road corridors in Carmel, Indiana. Both corridors were excluded from model development, and thus represent a true validation of the methodology. The Old Meridian Street corridor is further presented in a step-by-step application of the HCM 2010 Chapter 17 Urban Street Segments method, integrated with modifications for the roundabout nodes. Since the Old Meridian Street corridor contains one signalized intersection, the application exercise further illustrated how the method can be applied to mixed corridors of signals and roundabouts.

The Carmel, Indiana, validation exercise showed the developed corridor methodology correctly predicted the LOS for all four analysis routes for the Spring Mill Road corridor, and it predicted LOS within one letter grade for the Old Meridian Street corridor. The resulting percent free-flow speed estimates matched the field-observed data within 10 percent for Spring Mill Road (all four routes) and for the northbound routes on Old Meridian Street. The Old Meridian Street southbound routes matched within a 20 percent difference.

3.6.7. EQUIVALENT NON-ROUNDABOUT CORRIDORS

From this diverse set of corridors, neither roundabout nor signalized/stop-controlled corridor configurations consistently result in reduced travel time or intersection delays. Of the 20 *through* route combinations analyzed, approximately half resulted in lower travel time under a roundabout configuration and approximately half resulted in lower travel time under a non-roundabout configuration. The corridors having lower travel times under a roundabout configuration tend to have irregular intersection spacing, while those having two-way stop-controlled intersections tend to have lower travel times in the non-roundabout alternative.

The analysis becomes more illuminating when including the travel-time runs involving left turns to and from the arterial street. For all routes analyzed, approximately three-quarters resulted in lower travel time under a roundabout configuration than under an equivalent signalized configuration. Approximately one-third of the analyzed combinations resulted in reduced travel time of 20 percent or more. Evaluating non-operational elements such as safety would also help to differentiate more comprehensively between roundabout corridors and equivalent signalized corridors.

CHAPTER 4. APPLICATIONS

This chapter presents the two major applications developed under this project: the Corridor Comparison Document (CCD) and a predictive methodology for travel speed on a roundabout corridor. Section 4.1 presents an overview of the CCD. The entire CCD and four examples illustrating its use can be found in Appendix A. Section 4.2 is a step-by-step presentation of the predictive model for travel speed on a roundabout corridor, including equations used in the models and sample calculations. Details on the development of individual models used in the predictive method can be found in Chapter 3 of this report.

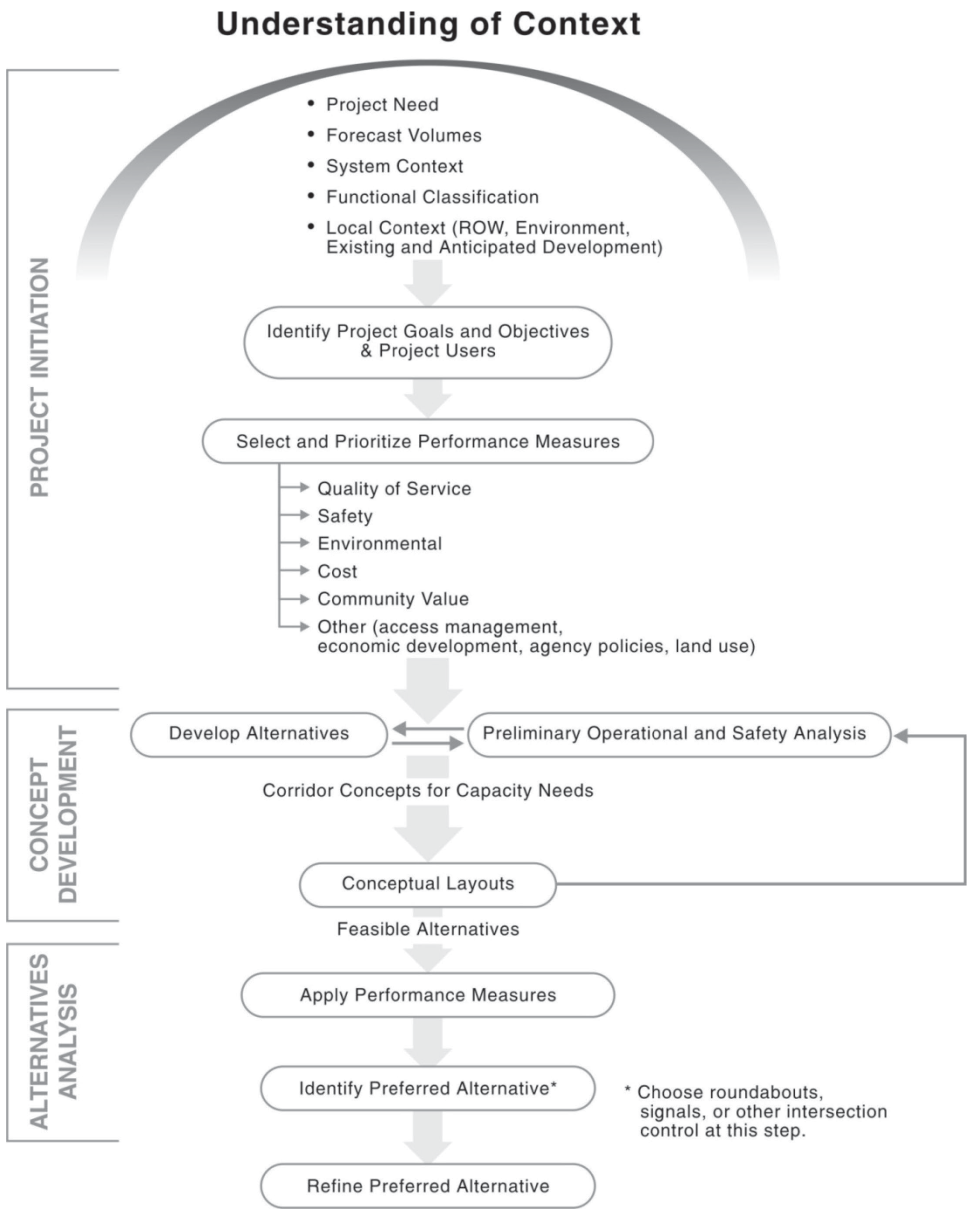
4.1. CORRIDOR COMPARISON DOCUMENT

Data collection and modeling conducted as part of this NCHRP project were focused on traffic operational performance such as travel time and speed. However, there are many other performance measures to consider when assessing corridor alternatives and choosing intersection control. The CCD presented in Appendix A provides an overall framework for users to compare alternative corridor configurations and to objectively inform project decisions based on the unique context of each corridor.

- Chapter 1 of the CCD provides the purpose and scope of the document. It also identifies the document's intended users and its relationship to other resource documents.
- Chapter 2 of the CCD provides information on different users of arterials. Users include passenger cars, buses, pedestrians, bicycles, trucks, and emergency vehicles. Chapter 2 is focused on differences between signals and roundabouts, and how these may affect the experience of users.
- Chapter 3 of the CCD discusses project-planning processes, and is written from the perspective of a practitioner evaluating alternatives for reconstructing an existing corridor or constructing a new roadway where the alignment has already been determined. In other words, it is focused on intersection control and cross-section decisions, not roadway alignment decisions. A typical project-planning process is presented in Chapter 3 and shown here as Exhibit 4-1.

The process shown in Exhibit 4-1 has three primary stages: project initiation, concept development, and alternatives evaluation. The CCD emphasizes the involvement of community stakeholders throughout the planning process.

Exhibit 4-1: Corridor Planning Process



Project initiation begins with gaining an understanding of context. What is the roadway location? Who will it serve? What type of roadway and place are stakeholders looking to create? Often, knowledge of a project's catalyst will help answer these questions. Some typical project catalysts include:

- A new greenfield corridor;
- An existing signalized corridor being evaluated because of capacity or safety performance;
- An existing roundabout corridor;
- A corridor with a specific access-management focus;
- A corridor that is explicitly focused on multimodal considerations;

Evaluating the Performance of Corridors with Roundabouts

- A corridor project driven by community enhancement objectives, speed-management needs, or economic development or growth opportunities; and,
- A hybrid corridor containing roundabouts, traffic signals, and stop-controlled intersections.

The degree to which the users identifed in Chapter 2 are present also provides practitioners with an understanding of context.

The CCD recommends choosing performance measures at an early stage of the project-planning process, when a practitioner has gained an understanding of a project's context but has not started development of alternatives. In the CCD, performance measures are grouped into six categories:

- Quality of Service Measures: Examples include delay and travel time for all modes.
- Safety Measures: Examples include the predicted number of fatal/injury crashes or expected relative difference in crash frequency.
- Environmental Measures: Examples include effects on public facilities, impacts to wetlands, and fuel consumption.
- Cost Measures: Examples include economic benefits associated with a project, the capital cost of a project, and the economic cost of crashes.
- Community Values: Examples include livability, place-making, and community acceptance.
- Others: Examples include policy choices such as "roundabouts first," tort and other legal issues, access management, economic development, speed management, and community acceptance.

All projects are unique, and key performance measures will differ from project to project. Chapter 4 of the CCD provides additional information on performance measures.

Concept development is the second primary phase of the project-planning process shown in Exhibit 4-1. Developing concept alternatives should be an iterative process. Some alternatives, while found to be infeasible, may have certain feasible and desirable features that can be incorporated into other alternatives. Examples of design elements of arterials that may differ between alternatives are listed below:

- Control at major intersections (traffic signal, roundabout, stop-control, or uncontrolled)
- Median type
- Number of lanes
- Presence of bike lanes
- Access/control at driveways and side streets
- Access management

Evaluating the Performance of Corridors with Roundabouts

- Roadway cross-section
- Right-of-way
- Design speed
- Intersection spacing

Alternatives analysis is the third primary stage of the project-planning process shown in Exhibit 4-1. Practitioners apply selected performance measures to the developed alternatives, and identify a preferred alternative.

Chapter 4 of the CCD provides guidance on assessment techniques for a variety of performance measures shown in Exhibit 4-2. The CCD lists common performance measures that are relevent to many arterial projects. In some cases, additional performance measures not listed in Exhibit 4-2 are relevent and should be considered; no list of performance measures could ever include all possible options.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-2: Performance Measures

PERFORMANCE MEASURES

Quality of Service

- Corridor travel time – all modes
- Intersection delay – all modes
- Operating speeds and speed profiles – auto mode
- Queues – auto mode
- Intersection capacity – auto mode
- Arterial capacity – auto mode
- Critical headways for permitted movements – auto mode
- Auto traffic impacts on pedestrians and bicyclists – pedestrian and bicycle modes
- Pedestrian and bicycle impacts on auto traffic – auto mode
- Bus operations – bus mode
- Delay to left-turn movements – auto mode
- Lane utilization – auto mode
- Side-street/driveway traffic performance at TWSC intersections – auto and pedestrian modes
  - Delay
  - Travel time for direct and indirect left turns (autos only)
  - Percent time side street is blocked by queue
  - Roadway crossing difficulty factor from HCM 2010 Urban Streets procedure (pedestrians only)
- Side-street traffic performance at major (signal/roundabout) intersections – auto mode
- Number of stops – all modes
- Additional bicycle- and pedestrian-only performance measures
- Emergency vehicle-only performance measures
- Truck performance measures

Safety

- Predicted crash frequency and severity (estimated with safety performance functions)
- Changes in crash frequency and severity (estimated with crash modification factors)
- Surrogate measures such as:
  - Presence of vehicles in crosswalk when in use by pedestrian
  - Speed differential between bicycles and autos
  - Number of close passes of bicyclists by autos
  - Vehicles parking in bicycle lanes
  - Conflict points (vehicle, pedestrian, bike)

Environmental

- Impacts to sensitive features, such as:
  - Wetlands
  - Historic properties
  - Cultural features
  - Habitats of protected species
  - Public facilities (parks, schools, etc)
  - Other sensitive or protected areas
- Impacts to private property, including access
- Specific state and local agency environmental performance measures
- Emissions and fuel consumption
- Noise analysis
- Light pollution
- Amount of impervious surface

Costs

- Construction costs
  - Pre-construction costs such as design
  - Right-of-way acquisition cost
  - Capital construction cost
- Maintenance costs
  - Annual maintenance cost
  - Electrical consumption cost
  - Future expansion cost
- User costs and benefits
  - Lost or gained productivity due to delay and travel-time impacts
  - Cost of injuries and delay due to crashes
  - Fuel consumption
- Community costs and benefits
  - Emissions and air quality
  - Development and impact of a facility on businesses
  - Access and mobility

Community Values

- Livability
- Walkability
- Property Value
- Aesthetics
- Placemaking
- Community acceptance
- Health
- Social equality
- Access

Other

- Land-use considerations
- Access-management considerations
- Economic-development/tax base considerations
- Agency policies, including “Roundabouts First” policies
- Private investment and public-private partnerships
- Legal issues (including tort)
- Public education

In addition to identifying assessment techniques for performance measures, Chapter 4 of the CCD also notes cost-benefit analysis and scoring as two techniques for comparing the results of an alternatives analysis and for identifying a preferred alternative.

Chapter 5 of the CCD presents four fictional example applications that illustrate its use:

- Example Application #1 is a new suburban arterial being built in a greenfield to create access to undeveloped land and to provide increased connectivity. The alternatives considered are a signalized arterial with a two-way left-turn lane, a signalized arterial with a median, and a roundabout arterial. The roundabout alternative is selected primarily because of predicted safety performance.
- Example Application #2 is a community enhancement project on an existing urban arterial. Alternatives considered are a road diet with traditional intersections (signals and two-way stop-control), a one-way couplet with traditional intersections, and a road diet with roundabouts. The roundabout alternative is selected primarily because of its traffic calming benefits and potental to enhance the image of the corridor.
- Example Application #3 is an existing two-lane highway in a rural, context-sensitive environment that is beginning to experience suburban-style development as it transforms into a vacation and second-home community. Alternatives considered are addition of a two-way left-turn lane and signals, addition of roundabouts (cross-section varies between median and two-way left-turn lane), and a bypass of the area where development is occurring. The roundabout alternative is selected based on place-making and aesthetic improvement opportunities.
- Example Application #4 is an existing suburban corridor being evaluated for safety and operational improvements due to changing context and a need for pavement rehabilitation. Alternatives considered are rebuilding the existing six-lane arterial, reducing the arterial to four lanes and maintaining signals at major intersections, and reducing the arterial to four lanes and replacing signals with roundabouts. The four-lane signal alternative is selected primarily because it offers pedestrian and bicycle improvements at a substantially lower cost than the roundabout alternative. However, one signal was replaced with a roundabout at an intersection with unusual geometry and past crash history.

4.2. PREDICTIVE OPERATIONS METHODOLOGY

Automobile travel speed is one performance measure for an arterial. On a signalized arterial, "Percent Free-Flow Speed" (%FFS), which is calculated by dividing the average segment speed by the segment free-flow speed, is the performance measure that determines automobile level of service per the Urban Street chapters (16 and 17) of the *Highway Capacity Manual* 2010 (HCM 2010).

The objective of modeling conducted as part of this project is to enhance the Urban Streets methodology in the HCM 2010 to accommodate one or more roundabouts along an urban street. The Urban Streets chapter currently allows

Evaluating the Performance of Corridors with Roundabouts

for the analysis of roundabouts in an urban street but does not provide a complete set of roundabout-specific equations for doing so. The Urban Street Segments chapter refers users to the roundabout control delay equation in Chapter 21 in place of a signal control delay equation. However, no equation is provided for geometric delay, and there is no adjustment to running speed or free-flow speed to account for the area near a roundabout where drivers accelerate and decelerate because of the roundabout's speed-limiting geometry. This project developed a framework and models to fill that gap. A summary of the framework, new equations recommended for inclusion into the next edition of the HCM, and sample calculations are presented below. A full discussion of the modeling research efforts is provided in Chapter 3.

4.2.1. FRAMEWORK

An urban street segment in HCM 2010 Chapter 17 is defined as a stretch of roadway between two intersections, including the downstream boundary intersection. In other words, the total delay of an urban street segment combines the control delay at the intersection with any midsegment delays resulting from queuing, driveway friction, high vehicular volumes, or other sources. Several computational steps are necessary in the Urban Street Segments chapter to arrive at the %FFS measure. These computational steps are:

- Step 1: Determine Traffic Demand Adjustments
- Step 2: Determine Running Time
- Step 3: Determine Proportion Arriving During Green
- Step 4: Determine Signal Phase Duration
- Step 5: Determine Through Delay
- Step 6: Determine Through Stop Rate
- Step 7: Determine Travel Speed
- Step 8: Determine Spatial Stop Rate
- Step 9: Determine Levels of Service
- Step 10: Determine Automobile Perception Score

For the application to roundabout corridors, Step 1 is maintained. The Step 2 procedure for average running time is generally maintained, but certain components of that step, like the free-flow speed estimation procedure, are updated for roundabout corridor operations. Steps 3 and 4 are not applicable to roundabouts. Step 5 is replaced with a roundabout-specific delay estimation procedure. Step 6 remains as a gap in the literature, where no model for stop rate at roundabouts is available. Because this performance measure is not used in the determination of level of service (LOS) for the urban street segment, it was not a focus in this research. Step 7 is maintained from Chapter 17 and uses earlier roundabout-specific interim steps. Step 8 is again a gap in the methodology for roundabouts, as no stop rate prediction procedure is available. Step 9 is maintained for roundabouts to estimate LOS. Step 10 represents another gap in the literature, as all studies to arrive at the automobile perception score were conducted at signalized intersections.

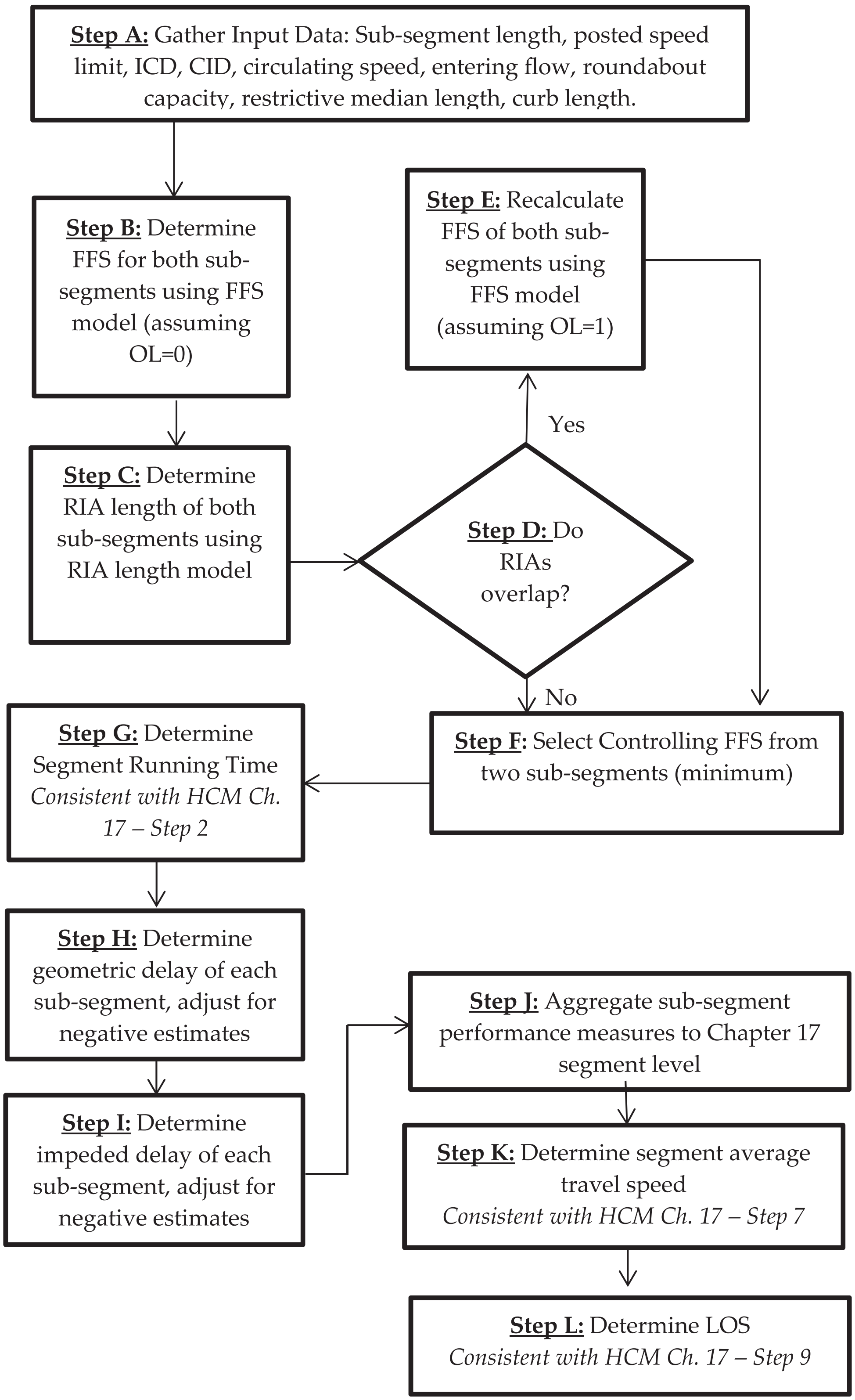
In HCM 2010 Chapter 17, an urban street segment begins at the stop bar of an upstream signalized intersection, extends through the intersection and the

Evaluating the Performance of Corridors with Roundabouts

section of roadway that follows, and ends at the stop bar of a downstream signalized intersection. For reasons discussed in Chapter 3 of this document, this segment definition is problematic. In general, it is problematic for roundabouts because it would include a portion of the impacts (geometric delay, free-flow speed decrease) associated with the upstream roundabout and the downstream roundabout. Therefore, the roundabout version of the procedure divides each segment into an upstream and downstream sub-segment, and, for some steps, separate equations are used for upstream and downstream sub-segments. Sub-segment definitions are illustrated in Exhibit 3-4 in Chapter 3.

Exhibit 4-3 illustrates the calculation framework for applying the models within a given segment. The framework should be applied separately for each upstream and downstream sub-segment before eventually aggregating to the segment level. Further aggregation to the facility level can be performed using the Urban Street Facility procedure of HCM 2010 Chapter 16. The framework is divided into computational steps A through L, with reference being made to the corresponding steps in HCM 2010 Chapter 17 for Urban Street Segments at the appropriate time. Steps for the roundabout-specific Urban Streets procedure are lettered rather than numbered to avoid confusion with the existing, signal-focused Chapter 17 procedure.

Exhibit 4-3: Computation Process



Note: After Step L, segments can be aggregated to facility level per HCM 2010 Chapter 16.

Evaluating the Performance of Corridors with Roundabouts

First, the analyst gathers input data and FFS is calculated based on the posted speed limit, segment length, central island diameter of the roundabout, and an assumption that overlapping roundabout influence areas (RIA) are not present. On a portion of a roundabout corridor between two roundabouts (i.e., not a first or last segment), the calculation is performed for a downstream sub-segment and the upstream sub-segment. Using the model-predicted FFS and the circulating speed within the roundabout, the RIA length is calculated for both sub-segments. RIA is the length of a sub-segment over which speeds are reduced due to the impact of the roundabout. The analyst must check whether the RIA lengths of the sub-segments overlap. If so, this necessitates a recalculation of the FFS with the overlap (OL) term set equal to 1, which will cause the predicted FFS to decrease.

With the final sub-segment FFS being determined, the analyst selects the controlling FFS for that segment. Because the FFS is defined as being measured at the segment mid-point, the same FFS has to be used for a downstream sub-segment and the next upstream sub-segment. In this procedure, the lower of the two FFS values is selected as the controlling FFS value for the entire segment.

Next, this procedure uses HCM Chapter 17 Step 2 to estimate the segment running time, followed by roundabout-specific models to estimate geometric delay and impeded delay. Impeded delay consists of control delay at the node and other delay associated with traffic volume (not geometry). Next, the sub-segment performance measures are aggregated to the HCM Chapter 17 segment level (Step J) and Chapter 17 Step 7 is used to determine the average travel speed on the segment. From the average travel speed, the level of service is estimated for the urban street segment with roundabouts.

4.2.2. MODELS

The procedure discussed in Section 4.2.1 includes four new models. They are presented here.

The FFS (S_f using HCM terminology) speed models for upstream and downstream sub-segments used in Step B are:

$$S_{f,US} = 15.1 + 0.0037L + 0.43SL + 0.05CID - 4.73OL$$

$$S_{f,DS} = 14.6 + 0.0039L + 0.48SL + 0.02CID - 4.43OL$$

where

$S_{f,US}$ = upstream free-flow speed (mph);

$S_{f,DS}$ = downstream free-flow speed (mph);

L = sub-segment length (feet);

SL = posted speed limit (mph);

CID = central island diameter (feet); and

OL = binary variable equal to one when overlapping influence areas are present on the sub-segment, zero otherwise.

The RIA models for upstream and downstream sub-segments used in Step C are:

$$RIA_{US} = 165.9 + 13.8S_f - 21.1S_c$$

$$RIA_{DS} = -149.8 + 31.4S_f - 22.5S_c$$

where

RIA_{US} = upstream roundabout influence area length (feet);

RIA_{DS} = downstream roundabout influence area length (feet); and

S_c = circulating speed (mph).

The geometric delay models for upstream and downstream sub-segments used in Step H are:

$$Delay_{geom,US} = 1.57 + 0.11S_f - 0.21S_c$$

$$Delay_{geom,DS} = -2.63 + 0.09S_f + 0.73 \cdot ICD \cdot \left(\frac{1}{S_c} - \frac{1}{S_f}\right)$$

where

$Delay_{geom,\ US}$ = upstream geometric delay (seconds); and

$Delay_{geom,\ DS}$ = downstream geometric delay (seconds).

The impeded delay models for upstream and downstream sub-segments used in Step I are:

$$Delay_{imp,US} = -5.35 + 0.15S_f + 42.50x - 0.03v_{entering}$$

$$Delay_{imp,DS} = -2.65 + 0.07S_f + 3.10x + 0.0020L - 0.0010L_{median} + 0.0014L_{curb}$$

where

x = volume-to-capacity ratio;

$v_{entering}$ = entering flow (vph);

L_{median} = length of sub-segment with restrictive median (feet); and

L_{curb} = length of sub-segment with curb (feet).

4.2.3. SAMPLE PROBLEM

This section presents a fictional sample problem. The same sample problem is presented in Example Application #1 of the CCD.

Evaluating the Performance of Corridors with Roundabouts

STEP A: GATHER INPUT DATA

Beechmont Avenue is a planned arterial facility. It will have seven roundabouts and no traffic signals. Exhibit 4-4 lists basic data about the facility. Because this project is at the planning stage, some values are approximated and assumed to be the same for all roundabouts on the corridor.

Exhibit 4-4: Data for Analysis

Variable	Value	Unit	Notes
Posted speed limit	45	mph	Planned speed limit for Beechmont Avenue
Intersection volume-to-capacity (v/c) ratio	0.78	none	The average of the range of v/c based on preliminary traffic analysis
Circulating speed	20	mph	Typical R^2 fastest path speed for a double-lane roundabout
Peak hour directional entry flow	1,000	vph	The average of the range of flow in the corridor traffic projections
Inscribed circle diameter (ICD)	160	ft	Typical value for a roundabout with two circulating lanes
Central island diameter (CID)	100	ft	Typical value for a roundabout with two circulating lanes

The sub-segment lengths for Beechmont Avenue are shown in Exhibit 4-5:

Exhibit 4-5: Segment Lengths

Roundabout	Sub-segment	Length (ft)
1	US	800
1	DS	1,140
2	US	1,000
2	DS	940
3	US	800
3	DS	890
4	US	750
4	DS	940
5	US	800
5	DS	1,140
6	US	1,000
6	DS	1,140
7	US	1,000
7	DS	290

"US" sub-segments are upstream of a roundabout, and "DS" sub-segments are downstream of a roundabout.

STEP B: DETERMINE FREE-FLOW SPEED

Temporarily assuming that the roundabout influence area of each roundabout does not overlap, the free-flow speed over each segment can be estimated using the free-flow speed models:

$$S_{f,US} = 15.1 + 0.0037L + 0.43SL + 0.05CID - 4.73OL$$

$$S_{f,DS} = 14.6 + 0.0039L + 0.48SL + 0.02CID - 4.43OL$$

where

$S_{f,US}$ = upstream free-flow speed (mph);

$S_{f,DS}$ = downstream free-flow speed (mph);

L = sub-segment length (feet);

SL = posted speed limit (mph);

CID = central island diameter (feet); and

OL = binary variable equal to one when overlapping influence areas are present on the sub-segment, zero otherwise.

The results are shown in Exhibit 4-6:

Exhibit 4-6: Free-Flow Speed Results

Roundabout	Sub-segment	Free-Flow Speed (mph)
1	US	42.4
1	DS	42.6
2	US	43.2
2	DS	41.9
3	US	42.4
3	DS	41.7
4	US	42.2
4	DS	41.9
5	US	42.4
5	DS	42.6
6	US	43.2
6	DS	42.6
7	US	43.2
7	DS	39.3

For example, the free-flow speed for sub-segment 1US can be computed using the free-flow speed model for an upstream sub-segment:

$$S_{f,US} = 15.1 + 0.0037(800) + 0.43(45) + 0.05(100) - 4.73(0) = 42.41 \text{ mph}$$

Using the downstream sub-segment free-flow speed model, the estimated FFS for sub-segment 1DS follows as:

$$S_{f,DS} = 14.6 + 0.0039(1{,}140) + 0.48(45) + 0.02(100) - 4.43(0) = 42.646 \text{ mph}$$

The above values are shown rounded in Exhibit 4-6. However, unrounded values for these and other intermediate calculations should be used for subsequent calculations.

STEP C: DETERMINE ROUNDABOUT INFLUENCE AREA LENGTH

The length of each roundabout influence area can be estimated using the roundabout influence area models:

$$RIA_{US} = 165.9 + 13.8 S_f - 21.1 S_c$$

$$RIA_{DS} = -149.8 + 31.4S_f - 22.5S_c$$

where

RIA_{US} = upstream roundabout influence area length (feet);

RIA_{DS} = downstream roundabout influence area length (feet); and

S_c = circulating speed (mph).

Exhibit 4-7: Roundabout Influence Area Results

The resulting lengths are shown in Exhibit 4-7:

Roundabout	Sub-segment	Roundabout Influence Area Length (feet)
1	US	329
1	DS	739
2	US	339
2	DS	715
3	US	329
3	DS	709
4	US	327
4	DS	715
5	US	329
5	DS	739
6	US	339
6	DS	739
7	US	339
7	DS	496

For example, the roundabout influence area of sub-segment 1US can be calculated using the roundabout influence area model for an upstream sub-segment:

$$RIA_{US} = 165.9 + 13.8(42.41)\ 21.1 - (20) = 329 \text{ ft}$$

The roundabout influence area of sub-segment 1DS can be calculated using the roundabout influence area model for a downstream sub-segment:

$$RIA_{DS} = -149.8 + 31.4(42.646) - 22.5(20) = 739 \text{ ft}$$

STEP D: CHECK FOR OVERLAPPING ROUNDABOUT INFLUENCE AREAS

In Step B it was assumed that roundabout influence areas did not overlap. To check this assumption, the roundabout sub-segment lengths listed in Exhibit 4-6 are compared to the roundabout influence areas calculated in Step C and listed in Exhibit 4-7. All sub-segments, except for one (7DS), are longer than their respective roundabout influence areas and do not overlap. The one overlapping sub-segment (7DS) is not a true example of two sub-segments having overlapping influence areas because it lies beyond the last roundabout on the corridor. However, because the roundabout influence area is still longer than the

Evaluating the Performance of Corridors with Roundabouts

sub-segment, it is considered to "overlap" and free-flow speed is recalculated in the next step.

STEP E: RECALCULATE FREE-FLOW SPEED OF SEGMENTS WITH OVERLAPPING ROUNDABOUT INFLUENCE AREAS

Treating sub-segment 7DS with $OL = 1$, the free-flow speed is now 34.9 mph.

STEP F: SELECT CONTROLLING FREE-FLOW SPEED FROM EACH PAIR OF SUB-SEGMENTS

This step takes the minimum free-flow speed within each pair of sub-segments for use in future calculations. For example, sub-segment 1DS has a free-flow speed of 42.6 mph, and sub-segment 2US has a free-flow speed of 43.2 mph, so the controlling free-flow speed for segment 1DS/2US is 42.6 mph.

STEP G: DETERMINE SEGMENT RUNNING TIME

Referring to Equation 17-6 from the HCM 2010 (Step 2 of Chapter 17), Exhibit 4-8 shows the running times calculated for each sub-segment:

Exhibit 4-8: Segment Running Time Results

Roundabout	Sub-segment	Proximity Adjustment Factor	Sub-segment Running Time (s)
1	US	1.027	14.6
1	DS	1.026	19.7
2	US	1.026	17.5
2	DS	1.027	16.9
3	US	1.027	14.7
3	DS	1.027	16.2
4	US	1.027	14.1
4	DS	1.027	16.9
5	US	1.027	14.7
5	DS	1.026	19.7
6	US	1.026	17.5
6	DS	1.026	19.7
7	US	1.026	17.5
7	DS	1.033	9.6

Note that this process also requires the computation of the proximity adjustment factor (HCM 2010 Equation 17-5). Due to the access-management policy

Evaluating the Performance of Corridors with Roundabouts

associated with the context of the site development, all midsegment access-point delays on Beechmont Avenue were assumed to be zero.

STEP H: DETERMINE GEOMETRIC DELAY OF EACH SUB-SEGMENT

Using these controlling free-flow speeds, the geometric delay incurred over the roundabout influence area can be estimated for each segment using the following model:

$$Delay_{geom,US} = 1.57 + 0.11S_f - 0.21S_c$$

$$Delay_{geom,DS} = -2.63 + 0.09S_f + 0.73 \cdot ICD \cdot \left(\frac{1}{S_c} - \frac{1}{S_f}\right)$$

where

$Delay_{geom,US}$ = upstream geometric delay (seconds); and

$Delay_{geom,DS}$ = downstream geometric delay (seconds).

The resulting geometric delays are shown in Exhibit 4-9:

Exhibit 4-9: Geometric Delay Results

Roundabout	Sub-segment	Geometric Delay (s)
1	US	2.0
1	DS	4.2
2	US	2.1
2	DS	4.1
3	US	2.0
3	DS	4.1
4	US	2.0
4	DS	4.1
5	US	2.0
5	DS	4.2
6	US	2.1
6	DS	4.2
7	US	2.1
7	DS	2.9

For example, the geometric delay of sub-segment 1US can be calculated using the geometric delay model for an upstream sub-segment:

$$Delay_{geom,US} = 1.57 + 0.11(42.4) - 0.21(20) = 2.0\text{ s}$$

Evaluating the Performance of Corridors with Roundabouts

The geometric delay of sub-segment 1DS can be calculated using the geometric delay model for a downstream sub-segment:

$$Delay_{geom,DS} = -2.63 + 0.11(42.6) - 0.21(20) + 0.73\left(\frac{1}{20} - \frac{1}{42.6}\right) = 4.2 \text{ s}$$

STEP I: DETERMINE IMPEDED DELAY OF EACH SUB-SEGMENT

Using the controlling free-flow speeds and traffic characteristics, impeded delay (i.e., the delay incurred due to traffic conditions and not geometric constraints) of each sub-segment is now calculated. The following are the impeded delay models:

$$Delay_{imp,US} = -5.35 + 0.15S_f + 42.50x - 0.03v_{entering}$$

$$Delay_{imp,DS} = -2.65 + 0.07S_f + 3.10x + 0.0020L - 0.0010L_{median} + 0.0014L_{curb}$$

where

x = volume-to-capacity ratio;

$v_{entering}$ = entering flow (vph);

L_{median} = length of sub-segment with restrictive median (feet); and

L_{curb} = length of sub-segment with curb (feet).

The results are shown in Exhibit 4-10:

Exhibit 4-10: Impeded Delay Results

Roundabout	Sub-segment	Impeded Delay (s)
1	US	4.2
1	DS	5.5
2	US	4.1
2	DS	5.0
3	US	4.1
3	DS	4.8
4	US	4.1
4	DS	5.0
5	US	4.2
5	DS	5.5
6	US	4.2
6	DS	5.5
7	US	3.0
7	DS	2.9

For example, the impeded delay of sub-segment 1US can be calculated using the impeded delay model for an upstream sub-segment:

$$Delay_{imp,US} = -5.35 + 0.15(42.4) + 42.5(0.78) - 0.03(1{,}000) = 4.2 \text{ s}$$

Evaluating the Performance of Corridors with Roundabouts

The impeded delay of sub-segment 1DS can be calculated using the impeded delay model for a downstream sub-segment:

$$Delay_{imp,DS} = -2.65 + 0.07(42.6) + 3.10(0.78) + 0.0020(1{,}140) - 0.0010(1{,}140)$$
$$+ 0.0014(1{,}140) = 5.5 \text{ s}$$

STEP J: AGGREGATE SUB-SEGMENT PERFORMANCE MEASURES TO CHAPTER 17 SEGMENT LEVEL

The average travel time over each segment is calculated by adding the following elements of each (non-overlapping) sub-segment:

1. The sub-segment running time,
2. The geometric delay, and
3. The impeded delay.

Exhibit 4-11 displays the average travel time for each segment, as well as a list of the sub-segments that comprise each segment:

Exhibit 4-11: Average Travel Time for Each Segment

Segment	Sub-segments Aggregated to Comprise Segment		Average Travel Time (s)
	Downstream	Upstream	
A	N/A	1US	20.8
B	1DS	2US	53.2
C	2DS	3US	46.7
D	3DS	4US	45.2
E	4DS	5US	46.7
F	5DS	6US	53.2
G	6DS	7US	53.2
H	7DS	N/A	15.5

For example, the average travel time of Segment A is 14.6 seconds (sub-segment 1US running time) + 2.0 seconds (sub-segment 1US geometric delay) + 4.2 seconds (sub-segment 1US impeded delay) = 20.8 s.

STEP K: DETERMINE SEGMENT AVERAGE TRAVEL SPEED

After the travel times are computed, the average segment travel speed is computed by dividing each segment length by the respective average travel time. This performance measure is consistent with the methodology in HCM Chapter 17. The results are shown in Exhibit 4-12:

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-12: Average Travel Speed for Each Segment

Segment	Average Travel Time (s)	Segment Length (ft)	Average Travel Speed (mph)
A	20.8	800	26.2
B	53.2	2,140	27.4
C	46.7	1,740	25.4
D	45.2	1,640	24.7
E	46.7	1,740	25.4
F	53.2	2,140	27.4
G	53.2	2,140	27.4
H	15.5	290	12.8

For example, the average travel speed (ATS) of Segment A is computed using the segment length (800 feet):

$$ATS = \frac{800\text{ ft}}{20.8\text{ s}} \times \frac{3{,}600\frac{\text{s}}{\text{hr}}}{5{,}280\frac{\text{ft}}{\text{mi}}} = 26.2\text{ mph}$$

STEP L: DETERMINE SEGMENT LEVEL OF SERVICE

Referring to Exhibit 17-2 in the HCM, the level of service can then be computed for each segment using the percentage of the base FFS at which the segment operates. The results are shown in Exhibit 4-13:

Exhibit 4-13: Level of Service for Each Segment

Segment	Average Travel Speed (mph)	Base Free-Flow Speed (mph)	Travel Speed as a Percentage of Base Free-Flow Speed	LOS
A	26.2	42.4	61.8	C
B	27.4	42.6	64.4	C
C	25.4	41.9	60.6	C
D	24.7	41.7	59.3	C
E	25.4	41.9	60.6	C
F	27.4	42.6	64.4	C
G	27.4	42.6	64.4	C
H	12.8	39.3	32.5	E

The results indicate that all but one segment (the short Segment H at the end of the route) operate at LOS C. The final segment, Segment H, operates at LOS E, likely because the entire segment lies within the influence area of Roundabout 7; i.e., vehicles are accelerating or decelerating over most of the segment.

Evaluating the Performance of Corridors with Roundabouts

FACILITY LEVEL OF SERVICE

To aggregate the travel times over the entire facility, HCM Chapter 16 is used directly. The facility travel speed is the aggregation of all segment travel speeds. The facility base FFS is the aggregation of all segment FFS.

For Beechmont Avenue, the travel speed is 25.8 mph and the facility base FFS is 42.4 mph. Per Exhibit 16-4 of the HCM 2010, the facility operates at LOS C.

Roundabout-specific models to analyze the performance of a roundabout corridor – that can be included in the Urban Streets procedure of the *Highway Capacity Manual* – were developed, as well as a framework for comprehensively comparing corridor alternatives and identifying a preferred alternative.

CHAPTER 5. CONCLUSIONS AND SUGGESTED RESEARCH

Corridors of roundabouts have developed as roundabouts have become more common and more distributed across the United States; 58 corridors were identifed at the start of this project. Based upon an in-depth examination of nine of those corridors, the use of roundabouts in series appears to have been successful in a wide variety of contexts throughout the United States. The following sections summarize the major conclusions from this study, and further research is recommended in a number of areas.

5.1. CONCLUSIONS

It is clear that roundabouts in series can be successful in a variety of contexts, although success can be measured in a variety of ways, including improvement in safety, improvement in operations, improvement in pedestrian and bicycle access, and community acceptance. The authors believe a case-by-case evaluation is the preferred approach for evaluating the performance of corridors, and the Corridor Comparison Document (CCD) developed in this project is intended to facilitate this process.

The CCD (Appendix A) provides a framework for comparing alternative corridor configurations and should objectively inform project decisions based on the unique context of each corridor. The following elements are included:

- Information on different users of arterials, including passenger cars, buses, pedestrians, bicycles, trucks, and emergency vehicles.
- An overview of the project planning process written from the perspective of a practitioner evaluating alternatives for reconstructing an existing corridor or constructing a new roadway where the alignment has already been determined.
- Typical performance measures, assessment techniques for performance measures, and methods for selecting and prioritizing performance measures. The performance measures presented in the CCD are grouped in the broad categories of quality of service, safety, environmental, costs, community values, and others.
- Four fictional example applications illustrating use of the CCD on the following corridors:
  - A new suburban arterial being buit in a greenfield to create access to undeveloped land and to provide increased connectivity;
  - A community enhancement project on an existing urban arterial;
  - An existing two-lane highway in a rural, context-sensitive environment that is beginning to experience suburban-style development as it transforms into a vacation and second-home community; and

   - An existing suburban corridor being evaluated for safety and operational improvements due to changing context and a need for pavement rehabilitation.

The use of the CCD for these four examples results in roundabout control being selected for three of the corridors, and signal control being selected for the fourth.

Field data collection for this project was focused on travel time, in part because this project's literature review identified a lack of methods of corridor-level operational evaluations of roundabouts. Vehicles equipped with GPS probes drove through and left-turn routes at different times of the day on nine roundabout corridors. Data from seven of the nine corridors were used to develop the following models:

- Free-flow speed models for roadway sub-segments upstream and downstream of roundabouts.
- Roundabout influence area (RIA) models for roadway sub-segments upstream and downstream of roundabouts. The RIA is the distance upstream and downstream of a roundabout in which unimpeded speeds are below free-flow speed due to acceleration into and deceleration out of the roundabout. Field data indicate that upstream RIA lengths generally vary more from site to site than downstream RIA lengths.
- Geometric delay (upstream and downstream of the roundabout). Field data indicate upstream geometric delay generally varies more than downstream geometric delay from site to site.
- Impeded delay (upstream and downstream of the roundabout).

These models were incorporated into the framework of the Urban Streets procedure in Chapter 17 of the HCM 2010 and can be used to estimate travel time and facility level of service on a roundabout corridor.

Field travel times were generally recorded during the a.m. peak, off-peak, and p.m. peak periods. Aggregating data within these times periods together for each corridor indicates the following:

- Study corridors operated at LOS A through C based on travel speed as a percent of free-flow speed (the HCM 2010 performance measure for Urban Streets).
- Most routes had the same LOS for the three time periods. Some changed by one letter grade.
- Travel speed and LOS for through routes and left-turn routes were generally similar, and there was no pattern of one performing better than the other.

Data from the two Carmel, Indiana, corridors were reserved for validation and not used in the development of the models. The validation exercise showed the developed corridor methodology correctly predicted the LOS for all four analysis routes for the Spring Mill corridor, and the LOS was within one letter grade for the Old Meridian corridor. The resulting percent free-flow speed estimates (the metric used to determine LOS) matched the field-observed data within 10 percent for Spring Mill Road (all four routes) and for the northbound routes on Old Meridian Road. The Old Meridian southbound routes matched within a 20 percent difference.

Comparisons of field-measured vehicle travel times and simulated "equivalent" corridors with signalized or two-way stop-control indicated the following:

- Neither roundabout nor signalized/stop-controlled corridor configurations consistently result in reduced travel time or intersection delays for through routes. Evidence suggests roundabout corridors have a good likelihood of improving travel time performance, but site-specific operational conditions may favor signalization or stop control. This finding reinforces the need for a case-by-case evaluation.
- Corridors with irregular intersection spacing (MD 216, Spring Mill Road, Avon Road) show a higher likelihood for having better travel times under a roundabout configuration rather than a signalized configuration.
- Corridors that can use two-way stop-controlled intersections in the place of roundabouts or signals generally produce better end-to-end travel times, even if intersection delays are lower under a roundabout configuration. The corridor having the lowest travel times under an equivalent non-roundabout configuration (La Jolla Boulevard) was designed with mixed signalized and unsignalized control. As a result, end-to-end travel times are more favorable under an equivalent non-roundabout configuration, even though the roundabout configuration resulted in lower intersection delays.
- The corridor analyzed with large intersection spacing and higher speeds (SR 539) showed virtually no difference in travel time between the two alternatives, despite the observation that the intersection delay increases in a roundabout configuration.
- For corridors where turning movements entering or departing the corridor are of similar or greater importance than end-to-end travel times, roundabout corridors appear more likely to improve those travel times. This may be due to the fact that cycle lengths and offsets for signalized corridors are typically prioritized for movement of through traffic over left turns and side-street movements. This is, of course, a variable that should be evaluated when determining the most appropriate operational strategy for a signalized corridor, further emphasizing the need for a case-by-case evaluation.
- Of the 20 *through* route combinations analyzed, approximately half resulted in lower travel time under a roundabout configuration and approximately half resulted in lower travel time under a non-roundabout configuration.
- Of the 60 route combinations analyzed, travel-time differences ranged from −54 percent (roundabout corridor travel time lower) to +80 percent (roundabout corridor travel time greater). The top 16 routes in which roundabouts had the greatest improvement in travel time were all left-turn routes. Forty-four routes showed a decrease in travel time with roundabouts, and 16 routes showed an increase in travel time with roundabouts.

Evaluating the Performance of Corridors with Roundabouts

- Some findings for specific corridors:
  - Approach delay was lower with roundabouts for all intersections in both major street directions except for SR 539.
  - Through-route travel time (average of both directions) increased with roundabouts on La Jolla Boulevard, Old Meridian Street, and Golden Road; decreased with roundabouts on MD 216, Spring Mill Road, Avon Road, and SR 67; and remained virtually unchanged on SR 539.
  - Travel time for routes with a left turn off the major street (average of both directions) increased with roundabouts on La Jolla Boulevard; decreased on MD 216, Old Meridian Street, Spring Mill Road, SR 539, Avon Road, and SR 67; and remained virtually unchanged on Golden Road.
  - Travel time for routes with a left turn onto the major street (average of both directions) increased with roundabouts on La Jolla Boulevard and decreased on the other corridors.
  - The La Jolla Boulevard corridor performs quite differently from the other corridors studied in this project. It is the most urban of the corridors studied, with considerable pedestrian, bicycle, and on-street parking activity. As a result, through vehicular traffic experiences more friction than was observed for other corridors. As confirmed in the corridor interviews, this outcome is consistent with the multimodal focus desired for this particular corridor.
- The comparisons were entirely operations-focused due to limitations of this project. However, safety comparisons would provide additional insights into which form of intersection control is preferred on each corridor. Section 3.5 of this report provides information on key comparative safety studies of roundabouts and signalized/stop-controlled intersections to date, and guidance on how to compare the expected number of crashes at a signalized or stop-controlled intersection and a roundabout.

In general, the findings of this project indicate a need for corridor-specific evaluations to determine which form of intersection control is operationally preferred on a given corridor. Furthermore, there are many performance measures other than traffic operations that are used to choose intersection control on a corridor.

5.2. SUGGESTED RESEARCH

While the overall corridor comparison framework developed for this project is comprehensive, the detailed data collection and analysis conducted within this project was focused on automobile traffic operations. Future studies of roundabout corridors should focus on other elements, including the following:

- Pedestrian and bicycle quality of service procedures for urban streets that include roundabouts: Like the auto mode methodology, the pedestrian and bicycle mode methodologies of the HCM 2010 Urban Streets procedure were developed using data collected on signalized arterials. The applicability of these methodologies to roundabout corridors should be assessed, and models and/or user preference surveys should be conducted for any roundabout-specific aspects of corridors

- Before/after safety analysis: Several studies have established crash modification factors (CMFs) for the conversion of signalized or stop-controlled intersections to roundabouts. However, corridor-level changes in safety performance have not been quantified, including the confounding effect of associated changes in access management.
- Predictive safety analysis: An upcoming NCHRP project (17-70) is planned to develop safety performance functions (SPFs) for roundabouts and enable greater confidence in safety comparisons of roundabouts to other intersection forms. This project should include data from roundabouts on corridors.
- Automobile traveler perception scores: In addition to level of service, the HCM 2010 Urban Streets procedure provides a traveler perception score for the automobile mode.
- Public support: There is anecdotal evidence that some roundabout corridors are favored by the public over other alternatives, but no known scientific surveys have been conducted.
- Wayfinding within the corridor: There is anecdotal evidence that travelers get lost within a roundabout corridor. Guidance on providing a system of signs within a corridor would improve the overall traveler experience.

Evaluating the Performance of Corridors with Roundabouts

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APPENDIX A. CORRIDOR COMPARISON DOCUMENT

The Corridor Comparison Document (CCD) provides an overall framework for users to compare alternative corridor configurations and to objectively inform project decisions based on the unique context of each project. The CCD provides a broad approach for helping to inform corridor solution concept evaluations; it presents a framework that could adapt to the range of potential catalysts that might be the impetus for a particular project. There is a wide range of project catalysts, and projects may have a combination of contributing factors.

Each project requires a unique range of considerations to evaluate potential solutions. Performance measures vary depending upon the project needs and context. Not all projects will have the same performance measures, nor will the full range need to be applied to each and every project. However, there are some performance measures that will generally apply to most projects. In some cases, a project may require special considerations to augment primary considerations. In such cases, the primary considerations alone may not be sufficient to differentiate the corridor solution needs. In those instances, adding additional considerations in a tiered approach could help introduce considerations that more clearly differentiate the project solution needs.

CHAPTER 1: INTRODUCTION

1.1. PURPOSE OF DOCUMENT

The CCD provides a sequence of potential criteria to be used when considering intersection control options on an arterial street with three or more major intersections that could potentially be controlled by a roundabout or traffic signal. This CCD is intended to help agencies choose between roundabouts and traffic signals on new or reconstructed arterials.

1.2. INTENDED USERS

This document is crafted for the following types of users:

- Traffic engineers
- Transportation planners
- Roadway designers
- Preparers of environmental documents

1.3. SCOPE OF GUIDE

This document guides users through the arterial corridor planning process, including project initiation, concept development, and alternatives analysis. A focus of this document is a sequential framework for selecting roundabouts or a traditional form and control (traffic signals or stop signs) for the intersection. For the purposes of this document, a corridor is considered to be an arterial street with three or more "major" intersections that could potentially be controlled with a roundabout or a traffic signal. Throughout the document, "major" intersections are defined as intersections needing roundabout, traffic signal, or stop control. "Minor" intersections are defined as intersections or driveways at which the side street is stop-controlled (with major street uncontrolled) regardless of control at the major intersections. The principles in this document could also be adapted to shorter corridors or grid networks, but these applications are not explicitly addressed.

The document provides guidance on evaluating alternatives and considering tradeoffs to make an informed intersection control decision. The document does not present standards or warrants for the use of a control device, as the choice is ultimately left to the user. Finally, the document focuses on corridor analysis, and does not explicitly address isolated intersections.

1.4. RELATIONSHIP TO OTHER RESOURCE DOCUMENTS

The document acknowledges established resource documents that could also be used when assessing roundabout and signalized alternatives for a corridor. These documents include:

- *Highway Capacity Manual* (2010)
- *Roundabouts: An Informational Guide, 2nd Edition* (NCHRP Report 672, 2010)
- *Highway Safety Manual, 1st Edition* (2011)
- *Manual of Uniform Traffic Control Devices* (2009)
- *A Policy on Geometric Design of Highways and Streets* (Green Book) (2011)

CHAPTER 2: USERS OF ARTERIALS

This chapter provides information on the different users of arterials and how each user affects or is affected by traffic signals and roundabouts, particularly throughout a corridor. This chapter helps readers consider the various modal users, their unique needs, how they interact with one another, and how they are affected by or affect roadway design elements. Key modes and information on them (generally qualitative) are presented here.

Identifying users and understanding their needs is a valuable exercise to establish key needs for a specific corridor. The function of a corridor and the modes it serves influence the project-planning process (discussed in Chapter 3) and the selection and evaluation of performance measures (discussed in Chapter 4). Many performance measures are mode-specific, and the relative weight and importance assigned to them when evaluating alternatives is, to some degree, a function of the volume and operating characteristics of a given mode. Addressing project catalysts and stakeholder priorities for a given corridor will influence selected performance measures and corridor evaluation needs.

Evaluating the Performance of Corridors with Roundabouts

2.1. PASSENGER CARS

Passenger cars often dominate arterial streets, in terms of their percentage of the overall mode split. Historical planning and design of arterials has focused on serving the needs of passenger cars. Trends in design, such as "complete streets," place greater emphasis on non-automobile modes than historical approaches. Some arterial design choices and passenger car operating characteristics are noted here:

- Traditionally, most operational analyses of arterials have been auto-based.
- In isolation, roundabouts generally have less vehicular delay than signalized intersections. However, roundabouts have geometric features that require all vehicles to slow (causing geometric delay), whereas traffic signals require non-turning vehicles to slow or stop only during the red interval or when a queue is present (i.e., at the start of green).
- Traffic signals assign right-of-way and are usually timed to favor major-street operations. This timing strategy increases minor-street delay.
- On a corridor, traffic signals can be coordinated to some extent to reduce delay and the number of vehicle stops on the arterial. Factors such as high turning volumes and balanced directional flows limit the extent to which delay and the number of vehicle stops can be reduced.
- During low volumes and off-peak periods, less delay generally occurs at roundabouts than at signalized intersections.
- Roundabouts generally have less severe vehicle crashes than signalized intersections. Single-lane roundabouts generally have fewer crashes than signalized intersections.
- Signalized intersections sometimes require more turn or through lanes than roundabouts to serve queue storage needs.

2.2. BUSES

Some corridors have scheduled service by public transit buses. Corridors may also serve a variety of other bus users such as school buses or charter buses. Some arterial design choices and bus operating characteristics are noted here:

- Public transit buses operating on a local route stop throughout a corridor to pick up and drop off passengers. Other modes, such as automobiles and bicycles, may not stop throughout a corridor unless directed to do so by a traffic control device.
- The diverse bus fleet has varying operational characteristics and design needs. The 2011 AASHTO Green Book has six types of bus design vehicles, including motor coaches, school buses, city buses, and articulated buses (*5*).
- At signalized intersections, bus-only queue-jump lanes are sometimes placed on the right side of the road to allow buses to bypass a queue of other vehicles at the start of the green phase. Similar bus bypass lanes

Evaluating the Performance of Corridors with Roundabouts

are also used at roundabouts in some European countries and in at least one US location (Oregon).

- At signalized intersections, transit signal priority is sometimes used to extend green time to allow a bus to proceed through an intersection or start the green interval early to reduce the queue delay a bus experiences.

2.3. PEDESTRIANS

Land use, proximity to walkable areas, and roadway design choices influence the volume of pedestrians on an arterial. An arterial without sidewalks or crosswalks may discourage pedestrian travel. For most users on most arterials, the arterial performance is primarily based on through-movement operation. For pedestrians, the experience of *crossing* the corridor can influence their mode choice (i.e., whether or not to walk) and quality of experience. Except when crossing a street, pedestrians generally operate beyond the vehicle travel lanes of the roadway on sidewalks (if present), grass, or dirt. Some arterial design choices and pedestrian operating characteristics are noted here:

- Traffic signals can provide pedestrians with right-of-way at an intersection. All conflicting vehicle movements are stopped, although right turns on red and permissive left turns are sometimes allowed. Pedestrians must wait for the associated signal phase to receive this right-of-way.
- A four-leg, single-lane signalized or stop-controlled intersection has 24 pedestrian-vehicle conflict points, and a four-leg, single-lane roundabout has 8 pedestrian-vehicle conflict points.
- The horizontal curvature of roundabouts slows vehicles. However, pedestrians must wait for a gap in the traffic stream before crossing. Pedestrian crossings at roundabouts may be signalized, although it is rare in the United States to date.
- Blind and visually impaired pedestrians face impediments to accessibility at roundabouts. Pedestrians must rely on gap and/or yield detection to identify when it is safe to cross.
- The HCM 2010 provides pedestrian level-of-service procedures for several types of roadways and intersections, including urban streets, signalized intersections, and stop-controlled intersections (two-way and all-way). Pedestrian delay, signal timing, vehicle speed, and crosswalk length are some of the factors that influence pedestrian level of service.
- Vehicle speed, vehicle composition, buffers, and proximity to the traveled way influence the perceived quality of service for pedestrians as computed in the HCM 2010. Roundabout corridors may have lower segment speeds than arterial corridors depending on the intersection density. This is due to deflected vehicle paths and the resulting geometric delay at roundabouts.

2.4. BICYCLES

Arterials have historically been designed without bicycle-sensitive design elements, such as bicycle lanes or storm drains without openings parallel to the direction of travel. As a result, bicyclists experience a poor quality of service on many arterials. Bicycles are considered vehicles in most state uniform vehicle codes and, therefore, are required to follow the same traffic control as motorized vehicles, except where bicycle-specific control is provided. Some arterial design choices and bicycle operating characteristics are noted here:

- At traffic signals, bicycles typically operate on the roadway (sometimes in a dedicated lane) and are typically controlled by vehicular signals. Sometimes bicycles operate on a multiuse path adjacent to the roadway.
- At roundabouts, bicycles may operate in the roadway with vehicles or, if available, they may use a multiuse path that is accessed via bicycle ramps on the entry and exit of the roundabout. When using a multiuse path, bicyclists cross any legs of the roundabout they encounter at pedestrian crosswalks.
- Along a road segment and away from intersections, bicyclists nearly always operate in the roadway. Riding on the sidewalk is prohibited in many jurisdictions in the United States, and multiuse paths are generally not common. If they prefer, bicyclists can dismount and walk through an intersection rather than riding through.
- The HCM 2010 provides bicycle level-of-service procedures for several types of roadways and intersections, including multilane highways, two-lane highways, urban street segments, and signalized intersections. Shoulder width, pavement quality, vehicle volume, presence/absence of bicycle-specific treatments, and the width of cross streets are some of the factors that influence bicycle level of service.

2.5. TRUCKS

Trucks are present on nearly all arterial streets. Truck size and acceleration/deceleration performance influence the design requirements of many roadway elements—even if truck volume is low. As truck volumes increase, their effects on the corridor and other users become more pronounced. Some arterial design choices and truck operating characteristics are noted here:

- Arterial corridors are generally designed to accommodate trucks as large as a WB-62 or WB-67, and truck volumes can be significant. Some arterials are designed to accommodate trucks larger than a WB-62 or WB-67.
- Trucks generally travel slower than passenger cars and, due to their deceleration/acceleration characteristics, experience more delay when required to stop.
- Trucks require more physical space than a passenger car for turn-lane storage and lane width to serve tracking, especially when multiple turn lanes are present.

Evaluating the Performance of Corridors with Roundabouts

- Trucks have greater air quality and noise quality impacts due to their size and weight.
- The speed of trucks, like other motorized vehicles, is limited by vehicle path radii at roundabouts.
- Oversize vehicles affect any intersection or corridor treatment.

2.6. EMERGENCY VEHICLES

Arterials usually have few design elements specific to emergency vehicles. However, elements such as paved shoulders can be beneficial to all motorized users, including emergency vehicles. Some arterial design choices and emergency vehicle operating characteristics are noted here:

- Corridors are used by police, fire, and ambulance vehicles. The frequency of this use is driven, in part, by the proximity of emergency services (fire stations, police stations, etc.) to the corridor.
- Some jurisdictions use emergency vehicle preemption at traffic signals. This technology uses some means to detect emergency vehicles as they approach the intersection, typically to initiate a sequence of signal phases to favor the approach on which an emergency vehicle is located.
- Roundabouts have no emergency vehicle preemption options without using traffic signals. Roundabouts do not assign priority to vehicle movements like traffic signals do.
- Traffic signals require electricity, while roundabouts do not (aside from illumination). Roundabouts are less affected by natural disasters than traffic signals, and an increased number of emergency vehicles may be traveling following a natural disaster.
- Roundabouts generally have fewer injury crashes than signalized intersections, which may decrease the need for travel by emergency responders.
- Some corridors feature specific treatments for accommodating emergency vehicles, such as mountable median openings.

2.7. USER SUMMARY

The user characteristics noted in this chapter are some of the many that may be relevant and appropriate to consider for a given arterial. In the early stages of the project-planning process, practitioners should make every effort to determine the types of users anticipated on a corridor and their specific needs. Chapter 3 explains the project-planning process in greater detail.

Evaluating the Performance of Corridors with Roundabouts

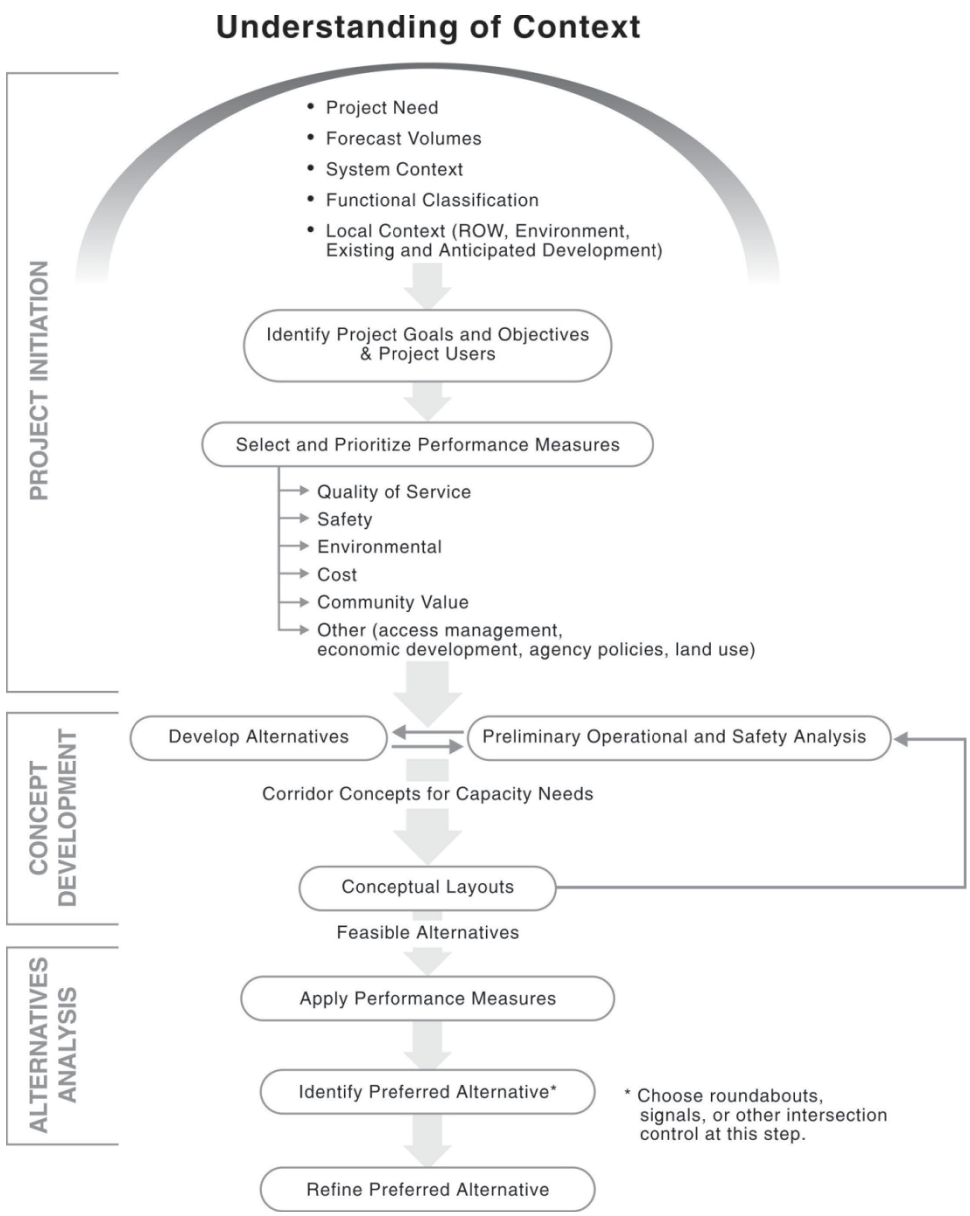
CHAPTER 3: PROJECT-PLANNING PROCESS

This chapter is primarily written from the perspective of practitioners evaluating alternatives for reconstructing an existing corridor or constructing a new roadway where the alignment has already been determined. In other words, the process presented in this chapter is focused on intersection control and cross-section decisions, not roadway alignment decisions.

Exhibit 3-1 illustrates elements of a typical project planning and development process. Community stakeholders are generally involved throughout this project-planning process. The heart of any project-planning process is developing and analyzing alternatives. When following activities in the project-initiation state, concepts can be developed and evaluated considering the identified needs of a specific corridor. This document aims to assist users with selecting and applying performance measures. Comparing roundabout and signal corridors can be challenging, as operational performance measures for roundabout corridors are less established than signal corridors. Performance measures are discussed in Chapter 4.

Exhibit 3-1 depicts three primary activity stages: Project Initiation, Concept Development, and Alternatives Evaluation. The choice between any feasible alternatives—including roundabout and traffic signal alternatives—is generally not made until the planning process is nearly at an end. This allows for all categories of performance measures (traffic operations, safety, environmental, cost, community values, and others) to be applied to all feasible alternatives before any are accepted or rejected. Often, a preferred alternative will not have the highest rating for every performance measure or even group of performance measures. Therefore, this document emphasizes the need to evaluate multiple alternatives before selecting a preferred alternative.

Exhibit 3-1: Corridor Planning Process



3.1. PROJECT INITIATION

3.1.1. Understanding of Context

> **Project Initiation**
>
> Concept Development
>
> Alternatives Analysis

Arterial streets can serve a wide variety of functions, users, and volumes. For example, some arterial streets serve large volumes of through vehicles, while others provide access to intensely developed land. An urban arterial may feature on-street parking, one-way travel, and bicycle lanes. A rural arterial may have one travel lane in each direction, with a posted speed of 55 miles per hour or more, and pass through primarily undeveloped land. When undertaking a planning effort for a new or improved corridor, a practitioner must first understand the context of the roadway. Where is it located? Who will it serve? What will be the purpose of users' trips on the roadway? What type of roadway and place are stakeholders looking to create?

Exploring these questions and others at the earliest stages of the project-initiation process helps provide an understanding of the project's context. This context will be a guiding principle throughout the corridor planning process. Identifying goals and objectives, developing alternatives, selecting performance measures, and evaluating and selecting alternatives should be based upon a project's unique contextual environment. Developing alternatives that match a roadway's context presents users with a self-describing roadway. For example, using curb and gutter on an urban arterial in a residential area reinforces an urban context, whereas an open section could suggest a higher-speed environment to drivers.

3.1.2. Identify Project Goals and Objectives & Project Users

Project goals and objectives will vary according to the context of a project and specific community needs. Therefore, performance measures will also vary depending upon the project needs and context. Not all projects will have the same criteria, nor will the full range of possible criteria need to be applied to each and every project.

Projects typically begin by identifying an existing or future need in the transportation system. Project needs may arise from a variety of catalysts. Each catalyst will influence the project goals and selection of performance measures. During these first steps in the planning process, specific solutions considered might include roundabout or signalized concepts. Project context and the unique needs of each corridor often require more than an early project visioning effort. Project goals and objectives are typically referred to during subsequent stages of the planning process to evaluate whether the alternatives effectively meet the project needs.

At the most fundamental level, project catalysts could include the following general categories, with most projects potentially including various elements of other categories:

- A new greenfield corridor;
- An existing signalized corridor being evaluated because of capacity or safety performance;
- An existing roundabout corridor;
- A corridor with a specific access-management focus;
- A corridor that is explicitly focused on multimodal considerations;
- A corridor project driven by community-enhancement objectives, speed-management needs, or economic development or growth opportunities; or,
- A hybrid corridor containing roundabouts, traffic signals, and stop-controlled intersections.

As discussed in Chapter 2, arterials serve a wide variety of users. The degree to which these users are present can vary greatly among arterials. Likewise, the

Evaluating the Performance of Corridors with Roundabouts

needs, goals, and objectives in some corridors may predominantly relate to deficiencies for one or several user groups, but not all. Therefore, during this stage of the planning process, planners and engineers should consider the needs of the community and begin to identify relevant performance measures for the corridor—including those that will vary with roundabouts and traffic signals—to help assess the performance of each control device.

3.1.3. Select and Prioritize Performance Measures

Corresponding to the wide variety of contexts in which arterials exist, there is a wide variety of performance measures available for analyzing arterial concepts. The choice between traffic signals and roundabouts for intersection control is one of many decisions faced by those planning a corridor. Practitioners must choose which performance measures are of importance to a given project, and which are not. Stakeholder input and budget constraints often influence this choice. Performance measures should be chosen in the early stages of the project-planning process, when a practitioner has gained an understanding of a project's context, but has yet to begin developing alternatives.

In this document, performance measures are grouped into six categories:

- *Quality of service measures.* Examples include delay and travel time for all modes.
- *Safety measures.* Examples include the predicted number of fatal/injury crashes or expected relative difference in crash frequency.
- *Environmental measures.* Examples include effects on public facilities, impacts to wetlands, and fuel consumption.
- *Cost measures.* Examples include economic benefits associated with a project, the capital cost of a project, and the economic cost of crashes.
- *Community values.* Examples include livability, place-making, and community acceptance.
- *Other measures.* Examples include policy choices such as "roundabouts first," tort and other legal issues, access management, economic development, speed management, and community acceptance.

While many performance measures are generally worthy of consideration, ranking is necessary to guide the comparison process. This document advocates a tiered approach to ranking project considerations. Certain tiers have broader application to all arterials, and others have lesser applicability or may apply primarily to certain contexts. In some cases, adding elements from subsequent tiers may help differentiate concepts. For example, if two corridors exhibit relatively little difference using the initial tier considerations, subsequent tier considerations could provide differentiating performance evaluation results to inform project decision making. A possible classification is as follows:

Tier I – critical considerations for most corridors (e.g., delay, safety, travel time, constructability).

Tier II – items that apply to many locations (e.g., access management, pedestrian accessibility).
Tier III – issues that may impact a smaller subset of corridors (e.g., effects on specific adjacent land uses such as schools or hospitals, familiarity of corridor drivers with certain control devices).

Exhibit 3-2 presents a number of performance measures that may be relevant when evaluating concepts for an arterial corridor. Additional information on these specific performance measures and their application is provided in the next chapter of this document.

Exhibit 3-2: Performance Measures

PERFORMANCE MEASURES

Quality of Service

- Corridor travel time – all modes
- Intersection delay – all modes
- Operating speeds and speed profiles – auto mode
- Queues – auto mode
- Intersection capacity – auto mode
- Arterial capacity – auto mode
- Critical headways for permitted movements – auto mode
- Auto traffic impacts on pedestrians and bicyclists – pedestrian and bicycle modes
- Pedestrian and bicycle impacts on auto traffic – auto mode
- Bus operations – bus mode
- Delay to left-turn movements – auto mode
- Lane utilization – auto mode
- Side-street/driveway traffic performance at TWSC intersections – auto and pedestrian modes
  - Delay
  - Travel time for direct and indirect left turns (autos only)
  - Percent time side street is blocked by queue
  - Roadway crossing difficulty factor from HCM 2010 Urban Streets procedure (pedestrians only)
- Side-street traffic performance at major (signal/roundabout) intersections – auto mode
- Number of stops – all modes
- Additional bicycle- and pedestrian-only performance measures
- Emergency vehicle-only performance measures
- Truck performance measures

Safety

- Predicted crash frequency and severity (estimated with safety performance functions)
- Changes in crash frequency and severity (estimated with crash modification factors)
- Surrogate measures such as:
  - Presence of vehicles in crosswalk when in use by pedestrian
  - Speed differential between bicycles and autos
  - Number of close passes of bicyclists by autos
  - Vehicles parking in bicycle lanes
  - Conflict points (vehicle, pedestrian, bike)

Environmental

- Impacts to sensitive features, such as:
  - Wetlands
  - Historic properties
  - Cultural features
  - Habitats of protected species
  - Public facilities (parks, schools, etc)
  - Other sensitive or protected areas
- Impacts to private property, including access
- Specific state and local agency environmental performance measures
- Emissions and fuel consumption
- Noise analysis
- Light pollution
- Amount of impervious surface

Costs

- Construction costs
  - Pre-construction costs such as design
  - Right-of-way acquisition cost
  - Capital construction cost
- Maintenance costs
  - Annual maintenance cost
  - Electrical consumption cost
  - Future expansion cost
- User costs and benefits
  - Lost or gained productivity due to delay and travel-time impacts
  - Cost of injuries and delay due to crashes
  - Fuel consumption
- Community costs and benefits
  - Emissions and air quality
  - Development and impact of a facility on businesses
  - Access and mobility

Community Values

- Livability
- Walkability
- Property Value
- Aesthetics
- Placemaking
- Community acceptance
- Health
- Social equality
- Access

Other

- Land-use considerations
- Access-management considerations
- Economic-development/tax base considerations
- Agency policies, including "Roundabouts First" policies
- Private investment and public-private partnerships
- Legal issues (including tort)
- Public education

Evaluating the Performance of Corridors with Roundabouts

Conceptually, the performance measures can be separated into two broad categories: quantitative and qualitative. Quantitative performance measures include delay, predicted crash frequency, and construction cost. These measures are evaluated with models, data, and computations. Qualitative performance measures include auto impacts to pedestrians and bicyclists, livability, and land-use considerations. Both categories of performance measures may be used when evaluating corridor concepts.

All projects are unique, and key performance measures will differ from project to project. For each project, relevant performance measures should be selected based on the project-initiation activities outlined in Exhibit 3-1. In many cases, it is helpful to prioritize these measures into two or three tiers. Performance measures that are most valued by the community and relevant to the goals and objectives of the project would be placed into Tier I. The process of selecting and tiering performance measures is demonstrated in the example applications at the end of this appendix.

3.2. CONCEPT DEVELOPMENT

3.2.1. Develop Alternatives and Conduct Preliminary Analysis

Project Initiation
Concept Development
Alternatives Analysis

Developing alternatives and conducting preliminary operations and safety analyses is the heart of the project-planning process. For an arterial project in which roundabouts and traffic signals are being considered, there may be more than two alternatives, as there are many design elements beyond intersection control to consider. For example, a corridor could have a two-way left-turn lane (TWLTL) or a raised median, and it could have sidewalks immediately adjacent to the curb or sidewalks offset from the curb by several feet. To isolate the comparison of these midblock elements from the comparison of intersection control, it may be helpful to develop alternatives in pairs; for example:

- Alternative 1
  - Alternative 1A: Arterial with four travel lanes, TWLTL, and signalized intersections.
  - Alternative 1B: Arterial with four travel lanes, TWLTL, and roundabouts.
- Alternative 2
  - Alternative 2A: Arterial with four travel lanes, raised median, and signalized intersections.
  - Alternative 2B: Arterial with four travel lanes, raised median, and roundabouts.

Using paired alternatives allows for comparing roundabouts, signals, and stop-control when all other roadway elements are the same. Conversely, it allows for

Evaluating the Performance of Corridors with Roundabouts

a comparison of alternatives with different midblock elements (such as a TWLTL versus a median) when intersection control is the same.

Developing alternatives should be an iterative process. A preliminary traffic assessment of an alternative may reveal it is infeasible or may dictate changes in the number of lanes or other major design elements. Some alternatives, while found to be infeasible, may have certain feasible and desirable features that can be incorporated into other alternatives. Examples of design elements of arterials that may differ between alternatives are listed below:

- Control at major intersections (traffic signal, roundabout, stop-control, or uncontrolled)
- Median type
- Number of lanes
- Width of lanes
- Presence of sidewalks
- Presence of bike lanes
- Access/control at driveways and side streets
- Access management
- Roadway cross-section
- Right-of-way
- Design speed
- Intersection spacing
- Presence of on-street parking

A number of planning-level analysis tools are available to practitioners and are appropriate to apply at this stage of the project-planning process. For example, critical movement analysis (CMA) is an analytical technique that estimates the volume-to-capacity ratio of an intersection using hourly traffic volumes, signal phasing, and lane configuration. CMA can be performed iteratively to assess the adequacy of different lane configurations. Section 7.4 of the 2004 FHWA publication *Signalized Intersections: Informational Guide* provides guidance on conducting CMA (*4*).

For roundabouts, *NCHRP Report 672: Roundabouts: An Informational Guide* offers several planning-level tools (*2*). Exhibit 3-12 of that document indicates ranges of AADT and left-turn volume percentages at which single-lane roundabouts or double-lane roundabouts may operate acceptably. Exhibit 3-14 of *NCHRP Report 672* provides similar guidance based upon hourly traffic volumes rather than AADT, and does so on an entry-by-entry basis rather than for the roundabout as a whole. Practitioners can use these tools and others to determine lane needs at the earliest stages of the project-planning process without conducting a full operational analysis. Use of these tools is illustrated in the example applications in Chapter 5.

Evaluating the Performance of Corridors with Roundabouts

Sometimes, the number of through lanes on a corridor is pre-determined. For example, an agency may have a programmed project that calls for a four-lane roadway between two points. In this situation, planning-level tools may be helpful to determine side-street lane needs, but may have limited applicability to the major-street lane needs.

3.2.2. Conceptual Layouts

Once alternatives are developed and lane needs are known, practitioners can produce conceptual layouts. These layouts can gauge impacts on the built and natural environment, connections and interaction with other elements of the transportation system, and other elements. At this time, it may become clear some alternatives are infeasible and should be eliminated from consideration. Often this can be because an intersection capacity improvement is too impacting to the surrounding land uses or because the proposed arterial typical section is too impacting in total footprint. In another example, an arterial cross-section that adds travel lanes at the expense of pedestrian and bicycle facilities (to avoid right-of-way acquisition) may prove to be infeasible if pedestrian and bicycle facilities are a project requirement.

Production of conceptual layouts may identify opportunities to revise concept designs. Design impacts may require reassessment of traffic operations and lane needs. For example, traffic analysis may indicate double-left-turn lanes are required on both major-street approaches to a signalized intersection, and a sketch may identify undesirable property impacts. Through iteration, an additional corridor concept with closer spacing of intersections, distribution of turning movements, and single-left-turn lanes could be developed.

3.3. ALTERNATIVES ANALYSIS

3.3.1. Apply Performance Measures

Project Initiation

Concept Development

Alternatives Analysis

Practitioners should apply selected performance measures to all alternatives developed during previous steps of the planning process. At least one performance measure from each of the six major groups listed in Exhibit 3-2 (traffic operations, safety, environment, cost, community values, and other) is likely to be relevant to a typical project. The tiering concept presented in Section 3.1.3 is helpful for assigning a relative importance to each performance measure. Chapter 4 presents several techniques to help practitioners compare alternatives. Example applications of the performance measures are presented in Chapter 5.

3.3.2. Identify Preferred Alternative

This is the point within the planning process at which practitioners must select one of the alternatives based upon the previously conducted evaluation. This choice will determine intersection control (roundabouts, traffic signals, or something else) and other major elements of design.

Evaluating the Performance of Corridors with Roundabouts

3.3.3. Refine Preferred Alternative

After selecting an alternative, increasingly detailed design activities will begin and culminate with a final design effort to produce construction documents. At each stage of project programming, environmental evaluations, preliminary engineering, and final design the preferred alternative is detailed sufficiently to address approval and documentation for each step leading to construction. During this refinement, there is little opportunity to change from roundabout to signal control or vice versa. The choice of control device is made prior to this stage.

CHAPTER 4: PERFORMANCE MEASURES

This chapter presents information on the performance measures previously listed in Exhibit 3-2. The performance measures discussed in this chapter are grouped into six categories and represent many of the things commonly considered when evaluating corridor arterials. The performance measures presented here are not an all-encompassing list. Some corridors will have unique contexts and needs, and considering relevant performance measures in addition to those presented here may be needed to adapt to unique project needs. The performance measures are user based (such as operations and safety) and non-user based (such as environmental impacts and costs). For user-based evaluations, this document will emphasize multimodal performance capturing the experience of various potential corridor users.

4.1. OPERATIONAL EVALUATION

Operational evaluations usually consider peak and off-peak times of the day and different analysis years, such as the opening year and the design year. Operational performance measures can be grouped into supply-side measures and demand-side/user-experience measures.

- Supply-side measures include capacity, corridor throughput, and volume-to-capacity ratio. These measures are not directly experienced by corridor users, but can impact user experience.
- Demand-side measures, such as delay and travel time, are experienced by users each time they pass through a corridor.

When possible, this document uses a multimodal approach to performance measures. Delay, for example, is experienced by drivers, pedestrians, bicyclists, and bus riders. This approach is consistent with the HCM 2010, which includes delay and level-of-service procedures for pedestrians and bicyclists in addition to autos. While not used for every evaluation, commonly used operational performance measures are listed in Exhibit 4-1, along with additional guidance and methods for assessing each performance measure.

4.1.1. Corridor Travel Time

The HCM 2010 includes a travel-time procedure for Urban Streets, defined as a street “with relatively high density of driveway access located in an urban area and with traffic signals or interrupting STOP or YIELD signs no further than 2 mi

Evaluating the Performance of Corridors with Roundabouts

apart" (*1*). Although theoretically applicable to any urban street, the procedure was developed using data from signalized corridors and does not have roundabout-specific computational steps. The NCHRP project that produced this CCD also collected field data at roundabout corridors and developed a travel-time procedure for them within the framework of the HCM 2010 Urban Streets procedure. Example Application #1 illustrates the use of the roundabout-specific urban street travel-time procedure developed as part of this project.

Exhibit 4-1: Operational Performance Measures

Performance Measure	Assessment Techniques and Notes
Corridor Travel Time	For the auto mode, practitioners can measure corridor travel time in the field, estimate it with the Urban Streets procedure of the HCM 2010 (Chapters 16 and 17), or estimate it with software. A common means of field measurement is the floating car technique, in which a test car is driven the length of the corridor and "floats" in the traffic stream by passing and being passed by the same number of vehicles. The Urban Streets procedure of the HCM computes the travel speed along a corridor; the length of the corridor can be divided by the travel speed to determine travel time. Finally, simulation models such as VISSIM or deterministic software packages such as SYNCHRO also provide estimates of corridor travel time. Once corridor travel time is known, it can be used to determine facility level of service for the auto mode. Practitioners can conduct travel-time studies of non-auto modes in a similar manner. An individual can travel the corridor on foot, by bike, or onboard a bus and measure the resulting travel time. The Urban Streets procedure of the HCM also computes travel speed for these non-auto modes, which can be converted to travel time. Many transit vehicles are now equipped with automatic vehicle location (AVL) technology, and data from this system can be analyzed to compute travel times. Finally, simulation models such as VISSIM can provide estimates of corridor travel time for non-auto modes. Field research conducted as part of this NCHRP project measured auto travel time on nine roundabout corridors and also estimated auto travel time on nine equivalent signalized corridors. These data are summarized in the main project report.
Intersection Delay	The HCM provides procedures for computing auto delay at signalized intersections, stop-controlled intersections, and roundabouts. The signalized intersection procedure also computes pedestrian and bicycle delay, and the two-way stop-control (TWSC) procedure also computes pedestrian delay. In addition to the HCM, practitioners commonly use deterministic software such as SYNCHRO, SIDRA, ARCADY, or RODEL to compute auto delay at intersections. Practitioners use simulation software such as SIMTRAFFIC and VISSIM as well, but to a lesser degree. These types of software also provide pedestrian, bicycle, and transit delay data to varying degrees.
Operating Speeds and Speed Profiles	Practitioners measure vehicle speeds at points along a corridor (spot-speeds) in the field with a radar or lidar gun, or within a simulation model. A series of spot-speeds collected along a corridor can be graphed to produce a speed profile; this allows a practitioner to understand how speeds vary along a corridor. A speed profile can also be generated by a GPS device that is onboard a vehicle while a travel-time run is performed. As part of this NCHRP project, field-measured speed profiles of nine roundabout corridors were constructed using GPS data. The data are summarized in the main project report.

Exhibit 4-1 Continued

Performance Measure	Assessment Techniques and Notes
Queues	The HCM provides procedures for determining queue lengths at signalized, stop-controlled, and roundabout-controlled intersections. Practitioners may also use deterministic software such as SYNCHRO and SIDRA or simulation software such as SIMTRAFFIC and VISSIM to estimate queue lengths. Practitioners generally analyze 50^{th}-percentile and 95^{th}-percentile queues, as these represent average and reasonable worst-case values, respectively. When queues form at roundabouts, they are generally "rolling" queues in which vehicles in queue advance one car length at a time as vehicles at the head of the queue enter the roundabout.
Intersection Capacity	Intersection capacity is generally determined for the sake of computing a volume-to-capacity (v/c) ratio. The HCM provides procedures for determining the v/c ratio of signalized, stop-controlled, and roundabout-controlled intersections. Deterministic software such as SYNCHRO, SIDRA, ARCADY, and RODEL also provide v/c ratios.
Arterial Capacity	In most cases, the capacity of an arterial is determined by the capacity of intersections along it. Exceptions to this include rural arterials and access-controlled arterials, both of which exhibit uninterrupted flow conditions.
Critical Headways for Permitted Movements	Delay at and capacity of unsignalized intersections is determined in part by a driver's acceptance or rejection of an available gap in the conflicting traffic stream. The average minimum gap a driver will accept is measured as the critical headway. Critical headway is usually determined by detailed reduction and analysis of video footage of an intersection.
Auto Traffic Impacts on Pedestrians and Bicyclists	When crossing a high auto-volume street, pedestrians and bicyclists will generally experience more delay compared to crossing a low auto-volume street. This is an example of a quantifiable impact that can be measured in the field or computed. Autos can also impact pedestrians and bicyclists in ways that are more challenging to measure and quantify. For example, bicyclists may have an improved experience on a roadway as shoulder width increases, vehicle speeds decrease, and the number of driveways decreases. The HCM 2010 provides pedestrian and bicycle level of service, which is based in part upon the comfort of these users as they travel along a roadway or through an intersection. Considering the context of a roadway and the degree of auto, pedestrian, and bicycle activity will help in selecting appropriate performance measures.
Pedestrian and Bicycle Impacts on Auto Traffic	Pedestrians and bicyclists can impact auto traffic operations on an arterial, especially as their volume increases. The intricacies of these interactions are often site-specific and not fully captured in operational models. Practitioners should perform field visits to qualitatively assess such issues.
Bus Operations	The *Transit Capacity and Quality of Service Manual* (TCQSM) provides a number of measures that gauge bus performance and rider experience on arterial roadways (*6*).
Delay to Left-Turn Movements	The same techniques used to assess intersection delay can be used to assess the delay for individual movements, including left turns. The manner in which a left turn is conducted at a roundabout is different than at a signalized intersection.
Lane Utilization	Traffic analysis procedures in HCM and software packages generally assume vehicles are equally or nearly equally distributed across multiple lanes serving the same movement. On an existing facility, this assumption can be verified in the field and analysis adjusted as necessary.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-1 Continued

Performance Measure	Assessment Techniques and Notes
Side-Street/ Driveway Traffic Performance at TWSC Intersections	Side streets and driveways can experience several operational challenges. If the major-street volume is sufficiently high, side streets and driveways may experience failing levels of delay regardless of their traffic volume. This is especially true for the left-turn movement, which may be made directly from the side street or made with a right turn followed by a U-turn. The issue can be compounded if a queue from a signalized intersection or roundabout blocks the side street. The HCM, deterministic software, and simulation models all compute side-street delay. Simulation models allow assessment of indirect left-turn treatments. Pedestrians also face challenges at TWSC intersections because of the lack of a control device to regulate major-street traffic. The Urban Streets procedure of the HCM 2010 (Chapters 16 and 17) contains a "roadway crossing difficulty factor" that quantifies this difficulty factor.
Side-Street Traffic Performance at Major (signal/ roundabout) Intersections	The same techniques used to assess intersection delay can be used to assess the delay for individual approaches. Unlike roundabouts, traffic signals can be timed to favor certain movements, and the degree to which a movement is favored can be changed by time of day or in response to traffic demands.
Number of Stops	The number of times a user stops while traveling the length of a corridor is a performance measure that is similar to delay. However, if delay is equal, users may prefer fewer, longer stops versus more frequent, shorter stops. Practitioners can use field measurements or a simulation model to determine the number of stops.
Additional Bicycle- and Pedestrian-Only Performance Measures	Some land-use developments may require considering unique pedestrian and bicycle needs. For example, a school or sports complex may have especially high crossing needs.
Emergency Vehicle–Only Performance Measures	Understanding emergency response needs early in the project-initiation stage may influence project decisions. For example, if preemption is an absolute need, signals may be favored over roundabouts. Likewise, understanding emergency vehicle design could potentially influence roundabout geometric design elements such as mountable curbs, truck apron design, or entry and exit lane widths.
Truck Performance Measures	Some arterials serve land uses such as ports or industrial facilities that generate a high percentage of truck trips, and may have unique operating characteristics and design needs.

4.2. SAFETY EVALUATION

The *Highway Safety Manual* (HSM) provides a quantitative way of assessing and comparing the safety performance of many types of roadways and intersections through the use of safety performance functions (SPFs) and crash modification factors (CMFs) (*3*). However, quantitative safety analysis is still a relatively new field and, in some situations, the use of surrogate safety measures is still appropriate. This is especially true for non-auto modes, as there has been less quantitative safety research in this area. Exhibit 4-2 lists a sample of safety performance measures and provides guidance on their application.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-2: Safety Performance Measures

Performance Measure	Assessment Techniques and Notes
Predicted Vehicle Crash Frequency	Chapter 12 of the HSM contains SPFs for segments and intersections of urban and suburban arterials. In general, fewer crashes are expected at a single-lane roundabout than at a signalized intersection. Example Application 1 illustrates the application of SPFs.
Changes in Vehicle Crash Frequency or Severity	The HSM contains CMFs for changing intersection control. For example, converting a signalized intersection to a roundabout has a CMF of 0.52. This means that, at a given intersection, only 52% of the crashes that occurred with a signal are expected to occur once the signal is replaced by a roundabout. Converting a minor-road stop-control intersection to a roundabout has a CMF of 0.56. The HSM also contains CMFs for conversion of specific intersection types in specific environments (such as a rural, four-leg, TWSC intersection) to roundabouts. The CMFs mentioned above are for all settings, all types of crashes, and all severities. The HSM also provides more specific CMFs for certain settings (such as urban or suburban, one or two lanes), types of crashes, and severities of crashes. Example Application 1 illustrates the application of CMFs.
Conflict Points	Draw all vehicle paths at an intersection and count the number of locations where they cross. Bicycle and pedestrian paths could be added as well if assessing conflicts for these modes. In general, there are fewer conflict points at roundabouts than at signalized intersections. A four-leg single-lane roundabout has 8 vehicle/vehicle and vehicle/person conflict points, and a four-leg signalized or stop-controlled intersection with single-lane entries and exits has 32 vehicle/vehicle and 24 vehicle/person conflict points. Example Application 3 contains additional discussion of conflict points.
Surrogate Measures	Examples of surrogates: • Presence of vehicles in the crosswalk when in use by a pedestrian, • Speed differential between bicycles and autos, • Number of close passes of bicyclists by autos, • Vehicles parking in bicycle lanes, and • Conflict points. Surrogates such as these can be measured in the field or estimated from preliminary plans.

4.3. ENVIRONMENTAL EVALUATION

Environmental evaluations are broad and consider impacts to the natural and built environment due to the construction and operation of the corridor. If a roundabout corridor and a signal corridor are evaluated with the same general roadway alignment, certain environmental impacts may differ only negligibly from one alternative to the other. However, different intersection footprints or roadway cross-sections could result in differing impacts, especially in sensitive areas. In general, roundabouts allow capacity to be added to the intersections with less impact to the roadway segments, compared to signalized corridor needs.

Environmental evaluations of different alternatives are conducted through processes outlined in state and federal regulations and guidance documents. The National Environmental Policy Act (NEPA) guides federal and most state environmental regulations. Examples of state-level regulations include the State

Evaluating the Performance of Corridors with Roundabouts

Environmental Policy Act (SEPA) in many states or the California Environmental Quality Act (CEQA). Exhibit 4-3 lists a sample of environmental performance measures, many of which are a component of federal and state evaluations, and provides guidance on their application.

Exhibit 4-3: Environmental Performance Measures

Performance Measure	Assessment Techniques and Notes
Impacts to Sensitive Features	Sensitive features include wetlands, historic properties, cultural features, habitat of protected species, public facilities such as parks and schools, historic main streets, and other various protected areas. The extents of these areas are generally identified by experts and then mapped. The location of these features is compared to the footprint of a proposed alternative to determine impacts.
Impacts to Private Property (including access to it)	The need to restrict access at certain points along a roadway is determined by agency guidelines and standards and engineering practices. Likely access restrictions can be determined at the conceptual planning stage using simple sketches or engineering drawings. Roundabout and signal corridors often apply different access-management strategies at intersections and midblock locations. This performance measure may often be a key part of environmental analysis of traffic signal and roundabout alternatives.
Specific State and Local Environmental Performance Measures	Jurisdictions may have additional performance measures used in their evaluation process.
Emissions and Fuel Consumption	Although not part of the NEPA process, software tools can be used to predict emissions and fuel consumption. Examples include traffic analysis software such as SIDRA and VISSIM and software specifically designed for emissions analysis such as the Environmental Protection Agency's MOBILE.
Noise	Software models may be used to predict the noise level at a location of interest such as a house or school. The differences in speed and acceleration/deceleration between roundabout operation and signalized intersection operation may result in different levels of noise.
Light Pollution	Software models may be used to predict the light level at a location of interest such as a house or park. Lighting requirements for roundabouts and signalized intersections vary from jurisdiction to jurisdiction.
Amount of Impervious Surface	The amount of paved area is typically measured during the development of pavement and grading plans. Roundabouts may require more pavement area at the intersection compared to a traffic signal but less on the entries and exits.

4.4. COSTS

Like environment evaluations, cost evaluations consider many different elements of a project. In addition to capital construction costs and various costs related to maintaining a facility, this category of performance measures also includes costs to users and the community. Exhibit 4-4 lists a sample of cost-related performance measures and provides guidance on their application.

Exhibit 4-4: Cost Performance Measures

Performance Measure	Assessment Techniques and Notes
Construction Costs	
Pre-construction Costs (such as design)	Often this is estimated as a percentage of the construction cost.
Right-of-Way Acquisition Cost	This is the value of privately-owned land to be acquired for the project. Exhibits 3-17 and 3-18 of *NCHRP Report 672* show the different right-of-way needs of typical signalized intersections and roundabouts.
Capital Construction Cost	During the evaluation of alternatives, develop cost estimates commensurable with the level of detail of the plans.
Maintenance Costs	
Annual Maintenance Cost	Elements of a roadway requiring maintenance include lighting, landscaping, grass shoulders or medians, pavement, signs, and traffic signal equipment.
Electrical Consumption Cost	Street lights and traffic signals require electricity.
Future Expansion Cost	Some roadways and intersections are designed to be expanded in the future, and the costs associated with this expansion can vary.
User Costs and Benefits	
Lost or Gained Productivity	The amount of time a person spends in congested conditions is a lost opportunity to do other, more productive things and practitioners can assign a monetary value to this time. The value of time for commercial drivers on the job is generally greater than the value of time for personal trips by non-commercial drivers.
Cost of Injuries Due to Crashes	A person injured in a crash is faced with medical bills, property damage, lost productive time, and pain and hardship. Chapter 7 of the HSM quantifies the costs associated with these elements.
Fuel Consumption	As discussed in Section 4.3 of the CCD, the amount of fuel consumed by all vehicles under different alternatives can be estimated. The average gasoline retail price then determines the associated cost of the fuel.
Community Costs and Benefits	
Emissions and Air Quality	Emissions from vehicles using a roadway and the resulting air quality have the potential to negatively impact communities adjacent to a roadway.
Development and Impact of a Roadway on Businesses	New or improved roadways may spur economic activity by creating access to undeveloped land or reducing the travel time existing in developed areas.
Access and Mobility	Roadways provide a mixture of access and mobility. An increase in one of these elements results in a decrease of the other. The degree to which each is provided should be based upon the needs of the surrounding community and the functional classification of the roadway.

4.5. COMMUNITY VALUES

Transportation facilities exist to serve people and communities. Practitioners consider the desires of communities and stakeholders in the vicinity of a roadway project. Assessing community values generally deals with qualitative issues often explored through interaction with key stakeholders and the community-at-large. Exhibit 4-5 lists a sample of community value performance measures and provides guidance on their application. Many topics are subjective and are based on the unique definitions provided by stakeholders and the project team. For example, community members may wish to improve "livability" and have corridor-specific measures for what it means to them on a particular project.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-5: Community Value Considerations

Performance Measure	Assessment Techniques and Notes
Livability	Livability often refers to the quality of life experienced by people who live, work, and recreate in a given place. Transportation infrastructure can positively or negatively affect a community's livability.
Walkability	Some communities value the ability for people to easily and comfortably make trips on foot rather than using another mode such as driving. There may be health benefits of living in a walkable community. Arterials designed in a manner to accommodate pedestrians do not create barriers to pedestrian travel.
Property Value	Roadways have the ability to increase or decrease property value. For example, roadways require right-of-way and increase or restrict access to property; these elements all affect property value.
Aesthetics	Visually appealing infrastructure can enhance the property value, the viability of businesses, and the desire of people to visit or pass through an area. The appropriate level of aesthetics is determined in part by the location and context of the roadway.
Place-making	Place-making refers to the creation of focal points and natural gathering places within a community. Elements to the transportation system such as the design of the road network and the streetscape can contribute to place-making.
Community Acceptance	Transportation projects should generally be accepted by the community before they are built, and projects should not be forced upon unsupportive communities. It may be challenging to gain support for roundabouts in communities unfamiliar with and unfavorable towards them. Special outreach and education may be needed to objectively inform and educate the community and stakeholders about the benefits and tradeoffs of roundabouts.
Health	Increased active transportation (such as walking and bicycling) may offer a health benefit to a community.
Social Equality	Transportation projects should reasonably serve the needs of users while avoiding unreasonable community impacts. Environmental justice is a component of the NEPA process that strives to prevent disproportional impacts to minority groups.
Access	Multimodal transportation infrastructure can serve a greater segment of the population than a single-mode facility. Some people are unable to drive due to age, disabilities, or other reasons, but are able to walk or use transit.

4.6. OTHER CONSIDERATIONS

In addition to the five major groups of performance measures listed above, additional considerations will be relevant on some corridors based upon their location, surrounding land use, and agency-specific policies. These other considerations are noted in Exhibit 4-6.

Exhibit 4-6: Other Considerations

Performance Measure	Comments
Land-Use Considerations	Roadway design choices can influence land-use patterns in a community, and vice versa. A certain alternative may be favorable to a community because it will fit well with existing land-use patterns or it will guide future land use in a manner that is desired.
Access-Management Considerations	The TRB *Access Management Manual* (2003) provides additional information on access management considerations and techniques for arterials. Access is generally restricted in the immediate vicinity of a roundabout, and when allowed it may be right-in right-out due to splitter islands. In practice, there is greater variability in access control in the vicinity of signalized intersections. Roundabouts naturally accommodate U-turns, whereas signalized intersections may require special treatments such as jug handles or a wide median. U-turns on multilane streets (both signals and roundabouts) require enough space for drivers to get into the correct lane to make the U-turn maneuver.
Economic-Development/Tax-Base Considerations	As discussed in Section 4.4, new or improved roadways have the potential to spur economic development and positively influence a community's tax base. Different project alternatives may have varying degrees of economic impact.
Agency Policies	Some agencies have policies favoring certain types of roadway or intersection treatments such as non-traversable medians or bicycle lanes. Some agencies have a "roundabouts first" policy. These policies generally require a roundabout (rather than a traffic signal) is constructed at new or rebuilt intersections unless it is demonstrated a roundabout is not feasible.
Private Investment and Public-Private Partnerships	Some roads, even if they are ultimately turned over to public agencies for ownership, are built with private funds. Private entities contribute funds to road projects that benefit them by improving access or mobility. Roads funded in this manner should meet the needs of investors in addition to meeting the needs of the community-at-large.
Legal Issues (including tort)	Roundabouts have documented safety benefits of reducing injury and fatal crashes. Roundabouts could be part of a community's overall risk management approach to asset management.
Public Education	New or innovative roadway and intersection treatments may be unfamiliar to drivers, and outreach and education may be desirable to improve road safety and their driving experience. Roundabouts are uncommon in some areas, and user education (for each user type) may be beneficial when a community's first roundabouts are installed.

4.7. EVALUATION TECHNIQUES

After the alternatives have been evaluated, a number of methods can be used to compare the evaluations and identify a preferred alternative. Two methods are presented here.

4.7.1. Cost/Benefit Analysis

This method compares project costs (initially defined in terms of dollars) to benefits (converted to dollar equivalents from other measures). An alternative in which benefits are greater than costs is typically seen as feasible, and the alternative with the highest ratio of benefits to costs is considered the preferred alternative. This analysis generally encompasses the entire lifecycle of a project. Capital costs are annualized over a period of time, such as 30 years. Some items generally included in a cost/benefit analysis are noted here:

- Costs: Right-of-way acquisition, capital construction, annual maintenance, electrical consumption.

Evaluating the Performance of Corridors with Roundabouts

- Benefits: Crash reduction; delay reduction (full-day and annual); fuel consumption reduction (full-day and annual); emissions reduction; economic benefits to residents, businesses, and the community-at-large; quality-of-life improvements.

Section 3.7 of *NCHRP Report 672* provides additional guidance on computing cost/benefit ratios as well as general information on costs. The construction cost of a roundabout varies greatly, but is generally greater than the cost of a stop-controlled intersection. Installation of a signal at an existing intersection typically costs less than construction of a roundabout if no lanes are added. If the installation of a signal requires approach widening, the cost is comparable to a roundabout. The operations and maintenance costs of a roundabout are generally less than a signal (*2*).

4.7.2. Scoring

This method is typically applied by constructing a matrix of all alternatives and all performance measures. Each alternative is given a score for each performance measure. The scores of performance measures can be weighted to account for some being more relevant and important to a given project than others. Summing all scores for each alternative can provide a basis for considering and selecting a preferred alternative.

CHAPTER 5: FICTIONAL EXAMPLE APPLICATIONS

Four example applications presented in this chapter illustrate the use of the CCD. Each example applies the CCD as an aid in the decision-making process. The CCD is not intended to be a standalone document, and practitioners should consult and apply standards and other guidance documents when appropriate.

The example applications are:

1. A new suburban greenfield corridor to provide access to undeveloped land and increased connectivity.
2. A community-enhancement project on an existing urban corridor.
3. An existing rural corridor in a context-sensitive environment beginning to experience suburban development.
4. An existing suburban corridor being evaluated for safety and operational improvements.

Although each of the examples listed above includes an aerial photograph of an existing corridor in the United States, they are not intended to portray actual corridors or present factual information. They use fictional street names and present fictional data and findings for the purpose of illustrating the concepts of the CCD.

Evaluating the Performance of Corridors with Roundabouts

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2. Rodegerdts, L., J. Bansen, C. Tiesler, J. Knudsen, E. Myers, M. Johnson, M. Moule, B. Persaud, C. Lyon, S. Hallmark, H. Isebrands, R. B. Crown, B. Guichet, and A. O'Brien. *NCHRP Report 672: Roundabouts: An Informational Guide, 2nd ed.* Transportation Research Board of the National Academies, Washington, D.C. 2010.

3. American Association of State Highway and Transportation Officials. *Highway Safety Manual.* Washington, D.C. 2009.

4. Federal Highway Administration. *Signalized Intersections: Informational Guide.* Washington, D.C. 2004.

5. American Association of State Highway and Transportation Officials. *A Policy on Geometric Design of Highways and Streets*. Washington, D.C. 2011.

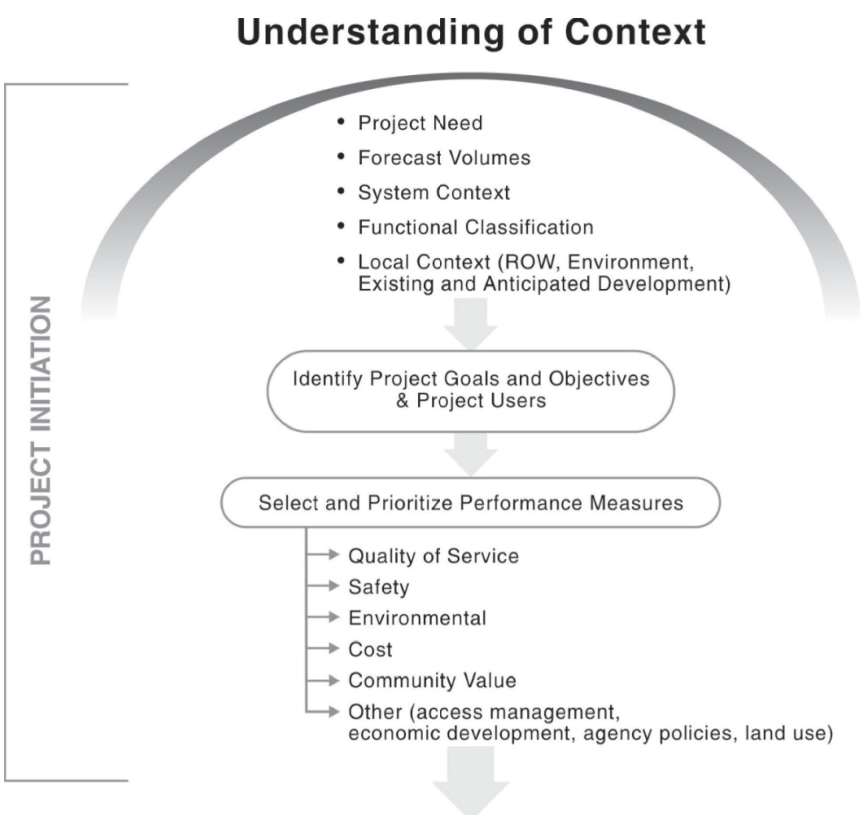
6. Transportation Research Board. *TCRP Report 100: Transit Capacity and Quality of Service Manual.* Transportation Research Board of the National Academies, Washington, D.C. 2003.

Evaluating the Performance of Corridors with Roundabouts

EXAMPLE APPLICATION 1. BEECHMONT AVENUE

This fictional example application presents a new suburban roadway built to provide access to land and increased connectivity. Example travel-time and crash-prediction calculations are presented as well.

1.1. PROJECT INITIATION



Steps in the Project Initiation phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

Exhibit 1-1: Key Data

Cross-Section	4 or 5 lanes (depending on median choice)
Travel Lanes	2 each direction
Intersection Spacing	1,500 to 2,000 feet
Average Daily Traffic (ADT)	26,000 veh/day
Peak-Hour Peak Direction Flow	800 to 1,200 veh/h
Sidewalks	To be provided on each side of roadway
Bicycle Lanes	None to be provided
Local Bus Service	None anticipated
Land Use	Currently rural, suburban development projected

1.1.1. UNDERSTANDING OF CONTEXT

Anderson County rezoned the area southwest of the I-32/I-232 interchange to encourage economic development. To provide access to this land, the county is

Evaluating the Performance of Corridors with Roundabouts

planning to extend Beechmont Avenue. Beechmont Avenue is a four-lane, minor-arterial roadway with a posted speed of 45 miles per hour. The current southern terminus of Beechmont Avenue is an interchange at I-32. 125th Street, a gravel roadway, continues south of the interchange for approximately two miles. Plans for Beechmont Avenue call for it to be extended to the south, and then to curve east and tie into the existing I-232/Westgate Boulevard interchange. The county has determined the approximate roadway alignment and is now considering the roadway typical section and intersection control. The county anticipates seven major intersections on Beechmont Avenue, spaced approximately 1500 to 2000 feet apart. The alignment is shown in Exhibit 1-2.

Exhibit 1-2: Proposed Corridor Alignment

Evaluating the Performance of Corridors with Roundabouts

1.1.2. USERS AND TRAFFIC VOLUME

The forecast ADT on the Beechmont Avenue extension is 26,000. The road is envisioned as a suburban facility serving retail and mid- to low-density residential development. A low degree of pedestrian and bicycle activity is expected. Forecasts estimate the majority of trips having an origin or destination along the corridor. Intersections serving large commercial or residential developments may experience a high percentage of turning vehicles.

For brevity, the fictional example applications only include volumes from one year. Generally, a planning study for a corridor would forecast future, design-year volumes, and practitioners would use these volumes for planning and analysis.

1.1.3. PROJECT CATALYST AND GOALS

The Beechmont Avenue extension was first proposed eight years ago in Anderson County's long-range transportation plan. The county now has programmed funds for design and construction within the next two years, and planning efforts have intensified. The primary goals of the project are:

- Provide access to the land surrounding the roadway from both I-32 and I-232,
- Transport users safely and efficiently, and
- Create additional network connectivity in the area and provide an alternate to I-32 and I-232 for local trips.

1.1.4. SELECT AND PRIORITIZE PERFORMANCE MEASURES

The sections below list the six groups of performance measures discussed in the Corridor Comparison Document (CCD), and identify specific performance measures of importance on the Beechmont Avenue corridor. The performance measures identified below are not necessarily all that could be considered for the Beechmont Avenue project. There are many performance measures that could be used to evaluate a corridor. Some are of critical importance for nearly all corridors (Tier I), and others are only applicable to some corridors (Tiers II and III). For the purpose of illustrating the use of the CCD, this example presents performance measures that are of particular interest on the Beechmont Avenue corridor and help to distinguish the alternatives from one another. This includes Tier I measures like safety and cost, and Tier II and III measures like land use and access. Performance measures of strong interest to the community are generally prioritized over those of lesser interest to the community.

1.1.4.1. *Quality of Service Performance Measures*

Quality of service refers to auto traffic operations and the experience of other corridor users such as pedestrians, bicyclists, and transit riders. Auto traffic operations are generally quantified with the procedures of the *Highway Capacity Manual*. The quality of service for other users is generally assessed qualitatively or with the multimodal procedures of the *Highway Capacity Manual.*

The Beechmont Avenue extension will be designed to adequately accommodate the forecast traffic volume. Intersections should not experience excessive delay

Evaluating the Performance of Corridors with Roundabouts

for any movement or be close to capacity. The county will assess corridor travel time as well.

Key Performance Measures: Arterial capacity; intersection delay, level of service, and volume-to-capacity ratio; delay to left-turn movements; and corridor travel time.

1.1.4.2. *Safety Performance Measures*

The *Highway Safety Manual* (HSM) provides safety performance functions and crash modification factors to quantify the expected number of crashes or changes in crash frequency associated with different roadway designs. Anderson County recently incorporated the HSM into their project-planning process, and assesses the safety performance of any potential alternative.

Key Performance Measure: Predicted crash frequency at intersections.

1.1.4.3. *Environmental Performance Measures*

Anderson County completed comprehensive environmental studies for extending Beechmont Avenue several years ago when they selected the roadway's alignment. In the current phase of this project, there will be minimal environmental regulatory issues to address. However, Anderson County policy requires fuel consumption analysis of new roadway projects. An analysis commensurate with the level of project plans will be performed in accordance with county policy.

Key Performance Measure: Fuel consumption.

1.1.4.4. *Cost Performance Measures*

Anderson County performs a cost-benefit assessment prior to investing in the final design and construction of a new roadway. The primary components of the analysis typically are construction cost, roadway maintenance costs, cost of delay, cost of crashes, and cost of fuel. The cost-benefit assessment is performed over the life cycle of the project.

Key Performance Measures: Construction cost, annual maintenance cost, cost of delay experienced by roadway users, costs of injuries and property damage suffered by roadway users, cost of fuel consumed by roadway users.

1.1.4.5. *Community Value Performance Measures*

Stakeholders support extending Beechmont Avenue on the county-selected alignment, and have not expressed strong feelings with regard to more detailed aspects of the project such as intersection or roadway cross-section design. Anderson County will continue to engage stakeholders throughout the planning process and incorporate their comments and suggestions when feasible.

Key Performance Measures: None.

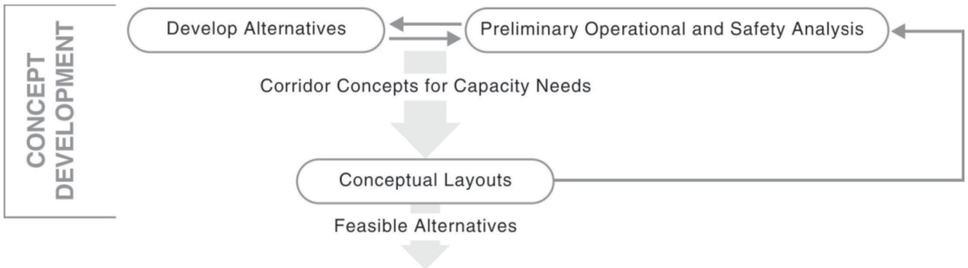
1.1.4.6. *Other Performance Measures*

One goal of the Beechmont Avenue extension is to provide access to undeveloped land. Anderson County will assess the degree to which alternatives provide access to adjacent land while also meeting mobility and safety needs.

Evaluating the Performance of Corridors with Roundabouts

Key Performance Measures: Land access and use.

1.2. CONCEPT DEVELOPMENT



Steps in the Concept Development phase of the Project-Planning Process (refer to CCD, Chapter 3)

1.2.1. DEVELOP ALTERNATIVES AND PRELIMINARY OPERATIONS AND SAFETY ANALYSIS

Studies on three-lane roadways indicate capacities approach 20,000 to 24,000 veh/day. With a forecast average daily traffic (ADT) volume of 26,000 veh/day, Beechmont Avenue will need two lanes in each direction to meet capacity needs. This will result in a four-lane or five-lane roadway, depending on median choice.

Anderson County considered three alternatives for the Beechmont Avenue extension:

1. Alternative #1 is a five-lane roadway with a two-way left-turn lane (TWLTL) and traffic signals at major intersections.
2. Alternative #2 is a four-lane roadway with a non-traversable median and traffic signals at major intersections.
3. Alternative #3 is a four-lane roadway with a non-traversable median and roundabouts at major intersections.

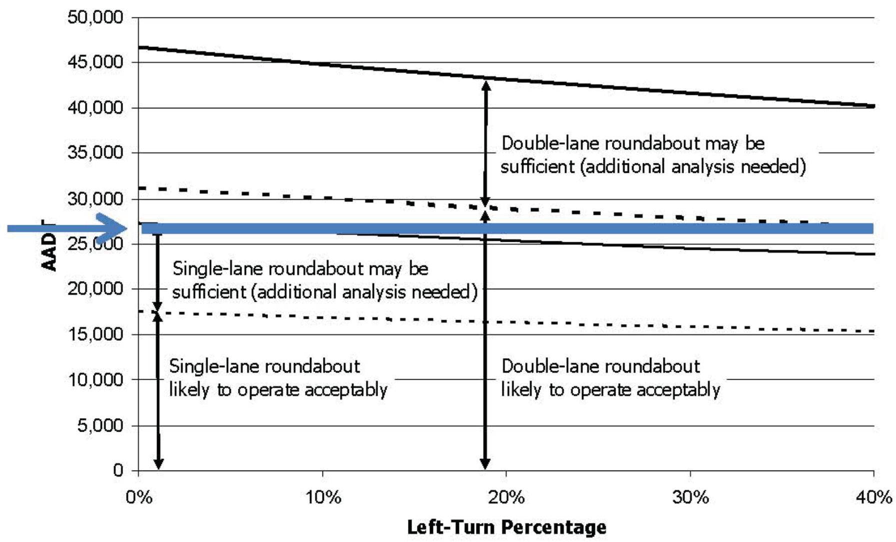
Signalized alternatives would have left-turn lanes on Beechmont Avenue and side-street lane needs will be determined on an intersection-by-intersection basis using forecast peak-hour volumes and critical-movement analysis. The FHWA publication *Signalized Intersections: Informational Guide* (2004) provides a methodology for critical-movement analysis. Lane needs will be reassessed during the alternatives analysis with a more extensive *Highway Capacity Manual* methodology-based analysis.

Roundabouts would have two through lanes on Beechmont Avenue and one or two lanes, as needed, on side-street approaches. Anderson County used Exhibit 3-12 of *NCHRP Report 672* (reproduced here as Exhibit 1-3) to assess the feasibility of double-lane roundabouts on Beechmont Avenue. As shown in Exhibit 1-3, double-lane roundabouts are generally sufficient with an ADT of 26,000 and will therefore be sufficient for the traffic volume on Beechmont Avenue.

Evaluating the Performance of Corridors with Roundabouts

The HSM includes methods to predict roadway segment crashes. Generally speaking, the HSM indicates divided roadway segments with raised medians typically have fewer crashes than undivided roadway segments with TWLTLs. Since Alternatives 2 and 3 have a raised median, these alternatives would likely show fewer predicted mid-block crashes than Alternative 1. The safety comparison of traffic signals and roundabouts is more complex and is discussed in the Alternatives Analysis section of this example application.

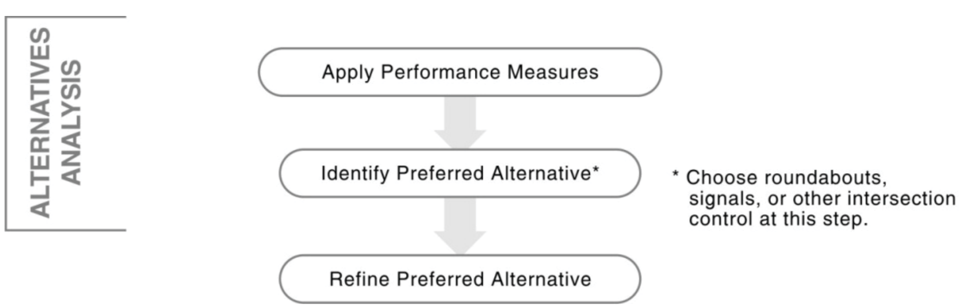
Exhibit 1-3: Planning Level Daily Intersection Volumes (Reproduced from *NCHRP Report 672*)



1.2.2. CONCEPTUAL LAYOUTS

Anderson County staff developed conceptual layouts of each alternative; the layouts depicted lane needs as determined by the preliminary operations analysis discussed in Section 1.2.1. In addition, the county roundabout concept designs considered principles from *NCHRP Report 672* to identify an appropriate inscribed circle diameter, applicable design vehicles, fastest-path evaluations, pedestrian and bicycle treatments, good path alignment for multilane entries and exits, and other applicable aspects of contemporary roundabout design.

1.3. ALTERNATIVES ANALYSIS



Steps in the Alternatives Analysis phase of the Project-Planning Process (refer to CCD, Chapter 3)

Evaluating the Performance of Corridors with Roundabouts

1.3.1. EVALUATE THE ALTERNATIVES

Exhibit 1-4 summarizes an analysis of the three alternatives proposed for Beechmont Avenue using the key performance measures identified in Section 1.1.4.

Exhibit 1-4: Alternatives Analysis

Performance Measure	Alternative 1 – Signals and TWLTL	Alternative 2 – Signals and Median	Alternative 3 – Roundabouts	Comments
Arterial capacity	Four- and five-lane signalized arterials generally have a capacity of 40,000 to 50,000 veh/day. This alternative will adequately serve the forecasted 26,000 veh/day.	Four- and five-lane signalized arterials generally have a capacity of 40,000 to 50,000 veh/day. This alternative will adequately serve the 26,000 veh/day.	Exhibit 1-3 indicates that double-lane roundabouts should adequately serve an ADT of 26,000 veh/day.	All alternatives have adequate link capacity.
Peak-hour intersection delay and LOS	Peak-hour intersection delay will range from 30 to 51 seconds (LOS C or D).	Peak-hour intersection delay will range from 31 to 53 seconds (LOS C or D).	Peak-hour critical-movement delay will range from 22 to 44 seconds, except at one roundabout where the critical approach will experience 61 seconds of delay.	Alternative 3 generally performs the best, with one exception.
Intersection v/c ratio	Peak-hour v/c ratio will range from 0.64 to 0.81.	Peak-hour v/c ratio will range from 0.65 to 0.84.	Peak-hour v/c ratio will range from 0.72 to 0.84.	In all alternatives, intersections are below capacity.
Delay to left-turn movements	Peak-hour delay for the left-turn movement at most intersections ranges from 40 to 70 seconds.	Peak-hour delay for the left-turn movement at most intersections ranges from 40 to 80 seconds.	Peak-hour delay for the left-turn movement at most intersections ranges from 15 to 40 seconds.	Alternative 3 has the lowest peak-hour left-turn delay.
Corridor travel time *(See Section 1.3.1.1)*	The HCM Urban Streets procedure predicts an a.m. peak travel time of 4.0 minutes and a p.m. peak travel time of 4.5 minutes.	Corridor travel time will be similar to Alternative 1. Mid-block speeds will increase slightly, and intersection delay will increase slightly due to increased turning volumes.	The roundabout corridor travel-time procedure developed as part of NCHRP 03-100 results in a p.m. peak travel time of 5.6 minutes.	Alternatives 1 and 2 result in a lower peak travel time.
Mid-block safety performance	Conflict points associated with right- and left-turn movements will be present at mid-block driveways.	HSM crash modification factors predict a decrease in mid-block crashes of 10% to 20% with a raised median.	The mid-block segment speeds are anticipated to be slower compared to Alternatives 1 and 2 and could reduce crash severity and improve the pedestrian's perceived quality of service.	Alternatives 2 and 3 are expected to have better mid-block safety performance than Alternative 1.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 1-4: Alternatives Analysis Con't

Perfor-mance Measure	Alternative 1 – Signals and TWLTL	Alternative 2 – Signals and Median	Alternative 3 – Roundabouts	Comments
Predicted intersec-tion crash frequency *(See Section 1.3.1.2)*	HSM predictive analysis estimates 5.28 auto crashes per year at the northernmost intersection, and similar rates at other intersections.	HSM predictive analysis estimates 5.28 auto crashes per year at the northernmost intersection, and similar rates at other intersections.	HSM comparative analysis estimates 1.21 to 2.27 auto crashes per year at the northernmost intersection, and similar rates at other intersections.	Alternative 3 is expected to have the best intersection safety performance.
Estimated construc-tion cost	$5.7 million.	$6.3 million.	$7.2 million.	Construction cost ranges from $5.7 to $7.2 million.
Annual mainte-nance cost	The annual maintenance and power needs for the seven traffic signals is estimated at $35,000.	Maintenance costs will be similar to Alternative 1, with the addition of mowing the grass in the median.	The annual maintenance cost of the seven roundabouts is estimated at $18,000 and includes maintaining landscaping and pavement markings.	The annual maintenance cost of Alternative 3 is approximate-ly half of Alternatives 1 and 2.
Annual cost of delay experi-enced by roadway users	Drivers will incur an annual cost of $7.6 million per year due to intersection delay.	Drivers will incur an annual cost of $7.9 million per year due to intersection delay.	Drivers will incur an annual cost of $4.3 million per year due to intersection delay.	Alternative 3 has the lowest delay cost.
Costs of injuries and property damage suffered by roadway users	The estimated cost of crashes at intersections will be $1.5 million per year.	The estimated cost of crashes at intersections will be $1.5 million per year. Costs associated with mid-block crashes will be lower than under Alternative 1.	The estimated cost of crashes at intersections will be $400,000 per year.	The safety cost of Alternative 3 is approximate-ly one-third of Alternatives 1 and 2.
Cost of fuel consumed by roadway users	Drivers will consume $8.0 million of fuel at intersections.	Drivers will consume $8.3 million of fuel at intersections. The median will lengthen some trips compared to Alternative 1 and increase mid-block fuel consumption as well.	Drivers will consume $3.8 million of fuel at intersections.	The fuel consumption cost of Alternative 3 is approximate-ly half of Alternatives 1 and 2.
Land access and use	This alternative will provide the highest degree of access.	This alternative prohibits left turns into and out of mid-block driveways.	Access will be similar to Alternative 2. Roundabouts will allow U-turns. There will be some further restrictions on driveways near intersections.	Alternative 1 provides more access than Alternatives 2 and 3.

Evaluating the Performance of Corridors with Roundabouts

1.3.1.1. *Corridor Travel-Time Analysis*

Anderson County estimated travel time for Alternative 3 with the procedure illustrated below. The procedure was developed as part of the NCHRP project that produced this CCD. More information on the procedure is found in the main project report. Calculations for the p.m. peak period are shown below.

STEP A: GATHER INPUT DATA

The sub-segment lengths shown in Exhibit 1-5 below were determined based on the conceptual plan in Exhibit 1-2:

Exhibit 1-5: Segment Lengths

Roundabout	Sub-segment	Length (ft)
1	US	800
1	DS	1,140
2	US	1,000
2	DS	940
3	US	800
3	DS	890
4	US	750
4	DS	940
5	US	800
5	DS	1,140
6	US	1,000
6	DS	1,140
7	US	1,000
7	DS	290

The definition of a segment and sub-segment is shown in Exhibit 3-4 in the main project report.

Exhibit 1-6 lists other data necessary for the travel-time analysis. Because this project is at the planning stage, some values are approximated and assumed to be the same for all roundabouts on the corridor.

Exhibit 1-6: Data for Analysis

Variable	Value	Unit	Notes
Posted speed limit (SL)	45	mph	Planned speed limit for Beechmont Avenue
Volume-to-capacity (v/c) ratio	0.78	none	The average of the range of v/c based on preliminary traffic analysis
Circulating speed (S_L)	20	mph	Typical R^2 fastest-path speed for a double-lane roundabout
Peak-hour directional entry flow	1,000	vph	The average of the range of flow in the corridor traffic projections
Inscribed circle diameter (ICD)	160	feet	Typical value for a roundabout with two circulating lanes
Central island diameter (CID)	100	feet	Typical value for a roundabout with two circulating lanes

Evaluating the Performance of Corridors with Roundabouts

STEP B: DETERMINE FREE-FLOW SPEED

Initially assume the roundabout influence areas of adjacent roundabouts do not overlap. This assumption is checked in Step D, and addressed in Step E if it proves to be incorrect. The free-flow speed over each segment can be estimated using the free-flow speed models:

$$S_{f,US} = 15.1 + 0.0037*L + 0.43 * SL + 0.05 * CID - 4.73 * OL$$

$$S_{f,DS} = 14.6 + 0.0039*L + 0.48 * SL + 0.02 * CID - 4.43 * OL$$

where

$S_{f,US}$ = upstream free-flow speed (mph);

$S_{f,DS}$ = downstream free-flow speed (mph);

L = sub-segment length (feet);

SL = posted speed limit (mph);

CID = central island diameter (feet); and

OL = binary variable equal to one when overlapping influence areas are present on the sub-segment, zero otherwise.

The results are shown in Exhibit 1-7:

Exhibit 1-7: Free-Flow Speed Results

Roundabout	Sub-segment	Free-Flow Speed (mph)
1	US	42.4
1	DS	42.6
2	US	43.2
2	DS	41.9
3	US	42.4
3	DS	41.7
4	US	42.2
4	DS	41.9
5	US	42.4
5	DS	42.6
6	US	43.2
6	DS	42.6
7	US	43.2
7	DS	39.3

For example, the free-flow speed for sub-segment 1US can be computed using the free-flow speed model for an upstream sub-segment:

Evaluating the Performance of Corridors with Roundabouts

$$S_{f-US} = 15.1 + 0.0037 \times (800\ feet) + 0.43 \times (45\ mph) + 0.05 \times (100\ feet) - 4.73 \times (0)$$

= 42.41 mph

Using the downstream sub-segment free-flow speed model, the estimated FFS for sub-segment 1DS follows as:

$$S_{f-DS} = 14.6 + 0.0039 \times (1{,}140\ feet) + 0.48 \times (45\ mph) + 0.02 \times (100\ feet) - 4.43 \times (0)$$

= 42.646 mph

The above values are shown rounded in Exhibit 1-7. However, unrounded values for these and other intermediate calculations should be used for subsequent calculations.

STEP C: DETERMINE ROUNDABOUT INFLUENCE AREA LENGTH

The length of each roundabout influence area can be estimated using the roundabout influence area models:

$RIA_{US} = 165.9 + 13.8 * S_f - 21.1 * S_c$

$RIA_{DS} = -149.8 + 31.4 * S_f - 22.5 * S_c$

where

RIA_{US} = upstream roundabout influence area length (feet);

RIA_{DS} = downstream roundabout influence area length (feet); and

S_c = circulating speed (mph).

The resulting lengths are shown in Exhibit 1-8.

Exhibit 1-8: Roundabout Influence Area Results

Roundabout	Sub-segment	Roundabout Influence Area Length (feet)
1	US	329
1	DS	739
2	US	339
2	DS	715
3	US	329
3	DS	709
4	US	327
4	DS	715
5	US	329
5	DS	739
6	US	339
6	DS	739
7	US	339
7	DS	496

For example, the roundabout influence area of sub-segment 1US can be calculated using the roundabout influence area model for an upstream sub-segment:

$RIA_{US} = 165.9 + 13.8 \times (42.41\, mph) - 21.1 \times (20\, mph)$

= 329 feet

The roundabout influence area of sub-segment 1DS can be calculated using the roundabout influence area model for a downstream sub-segment:

$RIA_{DS} = -149.8 + 31.4 \times (42.646\, mph) - 22.5 \times (20\, mph)$

= 739 feet

STEP D: CHECK OVERLAPPING ROUNDABOUT INFLUENCE AREAS

Step B assumed roundabout influence areas did not overlap. To check this assumption, compare the roundabout sub-segment lengths in Exhibit 1-5 to the roundabout influence areas calculated in Step C and listed in Exhibit 1-8. All sub-segments except for one (7DS) are longer than their respective roundabout influence areas and do not overlap. The one overlapping sub-segment—7DS—is not a true example of two sub-segments having overlapping influence areas because it lies beyond the last roundabout on the corridor. However, because the roundabout influence area is still longer than the sub-segment, it is considered to "overlap" and free-flow speed is recalculated in the next step.

STEP E: RECALCULATE FREE-FLOW SPEED OF SEGMENTS WITH OVERLAPPING ROUNDABOUT INFLUENCE AREAS

Treating sub-segment 7DS with OL = 1, the free-flow speed is now 34.9 mph.

STEP F: SELECT CONTROLLING FREE-FLOW SPEED FROM EACH PAIR OF SUB-SEGMENTS

This step takes the minimum free-flow speed within each pair of sub-segments for use in future calculations. For example, sub-segment 1DS has a free-flow speed of 42.6 mph, and sub-segment 2US has a free-flow speed of 43.2 mph, so the controlling free-flow speed for segment 1DS/2US is 42.6 mph.

STEP G: DETERMINE SEGMENT RUNNING TIME

Referring to Equation 17-6 from the HCM 2010 (Step 2 of Chapter 17), the running times are calculated for each segment, as shown in Exhibit 1-9:

Exhibit 1-9: Segment Running Time Results

Roundabout	Sub-segment	Proximity Adjustment Factor	Sub-segment Running Time (s)
1	US	1.027	14.6
1	DS	1.026	19.7
2	US	1.026	17.5
2	DS	1.027	16.9
3	US	1.027	14.7
3	DS	1.027	16.2
4	US	1.027	14.1
4	DS	1.027	16.9
5	US	1.027	14.7
5	DS	1.026	19.7
6	US	1.026	17.5
6	DS	1.026	19.7
7	US	1.026	17.5
7	DS	1.033	9.6

This process also requires the computation of the proximity adjustment factor (HCM 2010 Equation 17-5). Due to the access management policy associated with the context of the site development, all midsegment access point delays on Beechmont Avenue were assumed to be zero.

STEP H: DETERMINE GEOMETRIC DELAY OF EACH SUB-SEGMENT

Using these controlling free-flow speeds, the geometric delay incurred over the roundabout influence area can be estimated for each segment using the following model:

$$Delay_{geom,\ US} = 1.57 + 0.11*S_f - 0.21*S_c$$

$$Delay_{geom,\ DS} = -2.63 + 0.09*S_f + 0.73 * ICD * (1/S_c - 1/S_f)$$

where

$Delay_{geom,\ US}$ = upstream geometric delay (seconds); and

$Delay_{geom,\ DS}$ = downstream geometric delay (seconds).

The resulting geometric delays are shown in Exhibit 1-10:

Exhibit 1-10: Geometric Delay Results

Roundabout	Sub-segment	Geometric Delay (s)
1	US	2.0
1	DS	4.2
2	US	2.1
2	DS	4.1
3	US	2.0
3	DS	4.1
4	US	2.0
4	DS	4.1
5	US	2.0
5	DS	4.2
6	US	2.1
6	DS	4.2
7	US	2.1
7	DS	2.9

For example, the geometric delay of sub-segment 1US can be calculated using the geometric delay model for an upstream sub-segment:

$$Delay_{geom,US} = 1.57 + 0.11 \times (42.4\ mph) - 0.21 \times (20\ mph)$$

= 2.0 seconds

The geometric delay of sub-segment 1DS can be calculated using the geometric delay model for a downstream sub-segment:

Evaluating the Performance of Corridors with Roundabouts

$$Delay_{geom,DS} = -2.63 + 0.11 \times (42.6\ mph) - 0.21 \times (20\ mph) + 0.73 \times \left(\frac{1}{20\ mph} - \frac{1}{42.6\ mph}\right)$$

= 4.2 seconds

STEP I: DETERMINE IMPEDED DELAY OF EACH SUB-SEGMENT

Using the controlling free-flow speeds and traffic characteristics, impeded delay (i.e., the delay incurred due to traffic conditions and not geometric constraints) of each sub-segment is now calculated. The following are the impeded delay models:

$Delay_{imp,\ US}$ = -5.35 + 0.15*S_f+ 42.50*x – 0.03 * $v_{entering}$

$Delay_{imp,\ DS}$ = -2.65 + 0.07*S_f+ 3.10*x + 0.0020 *L – 0.0010 *L_{median} + 0.0014 * L_{curb}

where

x = volume-to-capacity ratio;

$v_{entering}$ = entering flow (vph);

L_{median} = length of sub-segment with restrictive median (feet); and

L_{curb} = length of sub-segment with curb (feet).

The results are shown in Exhibit 1-11:

Exhibit 1-11: Impeded Delay Results

Roundabout	Sub-segment	Impeded Delay (s)
1	US	4.2
1	DS	5.5
2	US	4.1
2	DS	5.0
3	US	4.1
3	DS	4.8
4	US	4.1
4	DS	5.0
5	US	4.2
5	DS	5.5
6	US	4.2
6	DS	5.5
7	US	3.0
7	DS	2.9

Evaluating the Performance of Corridors with Roundabouts

For example, the impeded delay of sub-segment 1US can be calculated using the impeded delay model for an upstream sub-segment:

$$Delay_{imp,US} = -5.35 + 0.15 \times (42.4\ mph) + 42.50 \times (0.78) - 0.03 \times (1{,}000\ vph)$$

= 4.2 seconds

The impeded delay of sub-segment 1DS can be calculated using the impeded delay model for a downstream sub-segment:

$$Delay_{imp,DS} = -2.65 + 0.07 \times (42.6\ mph) + 3.10 \times (0.78) + 0.0020 \times (1{,}140\ feet) - 0.0010 \times (1{,}140\ feet) + 0.0014 \times (1{,}140\ feet)$$

= 5.5 seconds

STEP J: AGGREGATE SUB-SEGMENT PERFORMANCE MEASURES TO CHAPTER 17 SEGMENT LEVEL

The average travel time over each segment is calculated by adding the following elements of each (non-overlapping) sub-segment:

1. The sub-segment running time,
2. The geometric delay, and
3. The impeded delay.

Exhibit 1-12 displays the average travel time for each segment, as well as a list of the sub-segments that comprise each segment:

Exhibit 1-12: Average Travel Time for Each Segment

Segment	Sub-segments Aggregated to Comprise Segment		Average Travel Time (s)
	Downstream	Upstream	
A	N/A	1US	20.8
B	1DS	2US	53.2
C	2DS	3US	46.7
D	3DS	4US	45.2
E	4DS	5US	46.7
F	5DS	6US	53.2
G	6DS	7US	53.2
H	7DS	N/A	15.5

For example, the average travel time of Segment A is 14.6 seconds (sub-segment 1US running time) + 2.0 seconds (sub-segment 1US geometric delay) + 4.2 seconds (sub-segment 1US impeded delay)

= 20.8 seconds

STEP K: DETERMINE SEGMENT AVERAGE TRAVEL SPEED

After the travel times are computed, the average segment travel speed is computed by dividing each segment length by the respective average travel time. This performance measure is consistent with the methodology in HCM Chapter 17. The results are shown in Exhibit 1-13:

Exhibit 1-13: Average Travel Speed for Each Segment

Segment	Average Travel Time (s)	Segment Length (ft)	Average Travel Speed (mph)
A	20.8	800	26.2
B	53.2	2,140	27.4
C	46.7	1,740	25.4
D	45.2	1,640	24.7
E	46.7	1,740	25.4
F	53.2	2,140	27.4
G	53.2	2,140	27.4
H	15.5	290	12.8

For example, the average travel speed of Segment A is computed using the segment length (800 feet):

$$\frac{800\ feet}{20.8\ seconds} \times \frac{3{,}600\,\frac{seconds}{hour}}{5{,}280\,\frac{feet}{mile}}$$

= 26.2 mph

STEP L: DETERMINE SEGMENT LEVEL OF SERVICE

Referring to Exhibit 17-2 in the HCM, the level of service can then be computed for each segment using the percentage of the base FFS at which the segment operates. The results are shown in Exhibit 1-14:

Exhibit 1-14: Level of Service for Each Segment

Segment	Average Travel Speed (mph)	Base Free-Flow Speed (mph)	Travel Speed as a Percentage of Base Free-Flow Speed	LOS
A	26.2	42.4	61.8	C
B	27.4	42.6	64.4	C
C	25.4	41.9	60.6	C
D	24.7	41.7	59.3	C
E	25.4	41.9	60.6	C
F	27.4	42.6	64.4	C
G	27.4	42.6	64.4	C
H	12.8	39.3	32.5	E

The results indicate all but one segment (the short Segment H at the end of the route) operate at LOS C. The final segment, Segment H, operates at LOS E, likely

Evaluating the Performance of Corridors with Roundabouts

because the entire segment lies within the influence area of Roundabout 7; i.e., vehicles are accelerating or decelerating over the entire sub-segment.

FACILITY LEVEL OF SERVICE

To aggregate the travel times over the entire facility, HCM Chapter 16 is used directly. The facility travel speed is the aggregation of all segment travel speeds. The facility base FFS is the aggregation of all segment FFS.

For Beechmont Avenue, the travel speed is 25.8 mph and the facility base FFS is 42.4 mph. Per Exhibit 16-4 of the HCM 2010, the facility operates at LOS C.

1.3.1.2. *Predicted Intersection Crash Frequency*

Anderson County assessed intersection traffic safety using the crash-prediction method from the HSM. The crash-prediction method estimates the number of crashes that could be expected as a function of geometry and ADT. The northernmost intersection on the Beechmont Avenue extension was selected to demonstrate the procedure and potential results for each alternative. Basic assumptions used in the analysis and the calculations are summarized below.

ALTERNATIVES 1 AND 2 CRASH PREDICTION

Conditions:

- Four-lane, divided major road
- Two-lane, divided minor road
- One left-turn lane on each major road approach
- Protected/permitted left-turn signal phasing on major road
- Design AADT of major road is 26,000 vehicles/day
- Design AADT of minor road is 7,000 vehicles/day
- Lighting is present
- Suburban environment
- No available estimate of pedestrian or bicycle volume

Calculations:

Chapter 12 in Part C of the HSM provides Safety Performance Functions for segments and intersections on urban and suburban arterial highways. The intersection crash-prediction models are presented below for single- and multiple-vehicle crashes. Vehicle-pedestrian and vehicle-bicycle crashes are assumed to be negligible.

Safety Performance Functions (SPFs):

SPF for multiple-vehicle crashes at the intersection (Equation 12-21):

$$N_{bimv} = exp(-10.99 + 1.07 * ln(26{,}000) + 0.23 * ln(7{,}000)) = 6.845$$

SPF for single-vehicle crashes at the intersection (Equation 12-24):

$$N_{bisv} = exp(-10.21 + 0.68 * ln(26{,}000) + 0.27 * ln(7{,}000)) = 0.404$$

Additional equations are provided in the HSM for determining the proportion of total crashes that are Fatal and Injury (FI) crashes and Property Damage–Only (PDO) crashes. Anderson County's safety policy is more focused on total crashes than specific severities of crashes, so the additional calculations are omitted here for brevity.

The base conditions of the SPF assume no turn lanes and permitted left-turn phasing, permitted right-turn-on-red, no lighting, and no red-light cameras. To account for site-specific variations from these base conditions, Crash Modification Factors must be applied as multiplicative factors to the predicted number of crashes.

Crash Modification Factors (CMFs):

- Left-turn lanes on major approaches: 0.81 (Table 12-24)
- Protected/permitted phasing on major approaches: 0.99 (Table 12-25)
- Lighting: 0.91 (Equation 12-36 and Table 12-27)

Calculate Predicted Average Crash Frequency:

$N_{predicted\ int}$ = (6.845 + 0.404) * 0.81 * 0.99 * 0.91 = 5.28 auto crashes per year. (Some additional, pedestrian and bicycle crashes may occur, but this cannot be predicted without pedestrian and bicycle volumes.)

ALTERNATIVE 3 PREDICTED CHANGE IN CRASHES

Part C of the HSM does not contain SPFs for roundabouts, but Part D contains CMFs for converting various traditional intersection forms into roundabouts (these CMFs also appear in *NCHRP Report 672*). CMFs for converting signalized intersections to roundabouts are found in Table 14-3 of the HSM, reproduced here in Exhibit 1-15:

Exhibit 1-15: CMFs for Signal to Roundabout Conversion (Reproduced from HSM)

Table 14-3. Potential Crash Effects of Converting a Signalized Intersection into a Modern Roundabout (29)

Treatment	Setting (Intersection Type)	Traffic Volume	Crash Type (Severity)	CMF	Std. Error
Convert signalized intersection to modern roundabout	Urban (One or two lanes)	Unspecified	All types (All severities)	0.99*	0.1
			All types (Injury)	0.40	0.1
	Suburban (Two lanes)		All types (All severities)	0.33	0.05
	All settings (One or two lanes)		All types (All severities)	0.52	0.06
			All types (Injury)	0.22	0.07

Base Condition: Signalized intersection.

NOTE: **Bold** text is used for the most reliable CMFs. These CMFs have a standard error of 0.1 or less.
*Observed variability suggests that this treatment could result in an increase, decrease, or no change in crashes. See Part D—Introduction and Applications Guidance.
The study from which this information was obtained does not contain information related to the posted or observed speeds at or on approach to the intersections that were converted to a modern roundabout.

Evaluating the Performance of Corridors with Roundabouts

As shown in Exhibit 1-15, the CMF for converting a suburban signalized intersection to a two-lane roundabout is 0.33, with a standard error of 0.05. With 95% confidence, the crashes are reduced by a factor of:

$$0.33 \pm (2 * 0.05)$$

$$= 0.23 – 0.43$$

Therefore, if Beechmont Avenue is constructed with roundabouts (Alternative 3):

$$5.28 * 0.23 \text{ to } 5.28 * 0.43$$

$$= 1.21 \text{ to } 2.27$$

auto crashes per year are expected.

1.3.2. IDENTIFY PREFERRED ALTERNATIVE

All alternatives provide adequate capacity and generally operate acceptably under design-year traffic forecasts. Construction cost estimates range from $5.7 to $7.2 million.

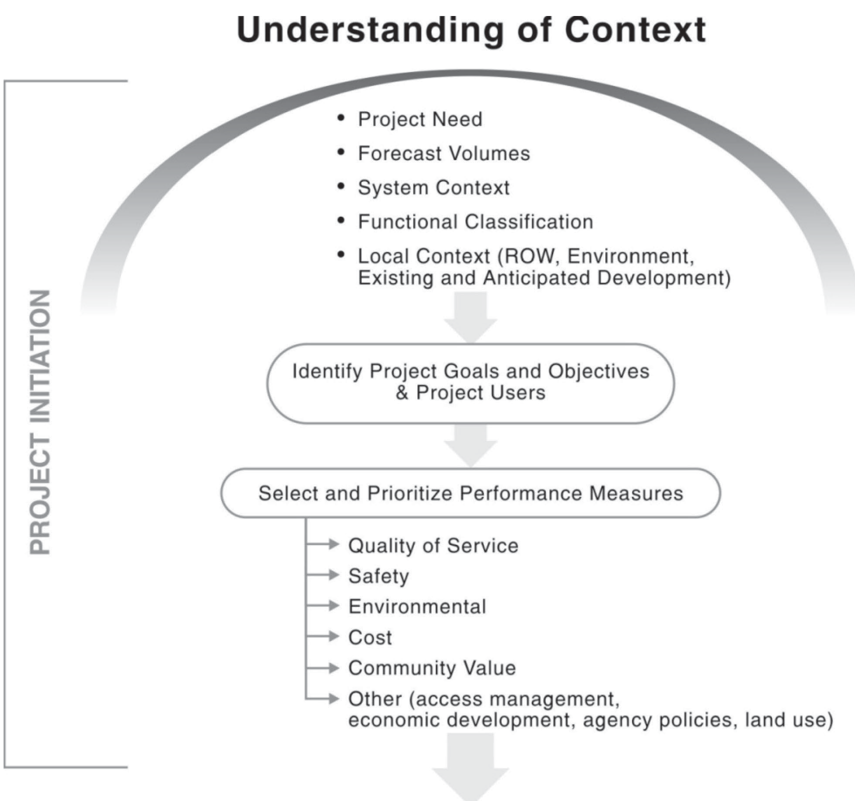
There was initially disagreement among Anderson County staff regarding alternatives. Drivers will experience longer travel times with Alternative 3 than they would with Alternatives 1 or 2 because of the geometric delay associated with the roundabouts. However, the HSM estimates roundabouts will have approximately one-third the number of crashes that signalized intersections on Beechmont Avenue would have, and the corresponding (safety-related) economic cost borne by drivers would be several times higher with signalized intersections than with roundabouts.

Anderson County ultimately selected Alternative 3 (roundabouts). Safety performance played a large role in the decision. It was agreed travel time should be of lesser importance on this corridor given that the primary function of the Beechmont Avenue extension is to increase access to land, not provide mobility through the area.

EXAMPLE APPLICATION 2. OCEAN DRIVE

This fictional example application presents a community enhancement project on an urban corridor.

2.1. PROJECT INITIATION



Steps in the Project Initiation phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

Exhibit 2-1: Key Data

Cross-Section	5 lanes plus parallel parking on both sides
Travel Lanes	2 each direction. The 5th lane is a two-way left-turn lane
Intersection Spacing	600 to 800 feet
ADT	16,000 veh/day
Peak-Hour Peak-Direction Flow	600 to 750 veh/h
85th Percentile Speed	40 mph
Existing Control	2 signals, 3 TWSC
Peak-Hour Pedestrian Volume Along Ocean Drive	50 to 100 p/h
Peak-Hour Pedestrian Volume Crossing Ocean Drive	35 to 80 p/h per intersection
Sidewalks	Present on both sides
Bicycle Lanes	None
Local Bus Service	15 minute headways
Land Use	Dense Residential/Commercial

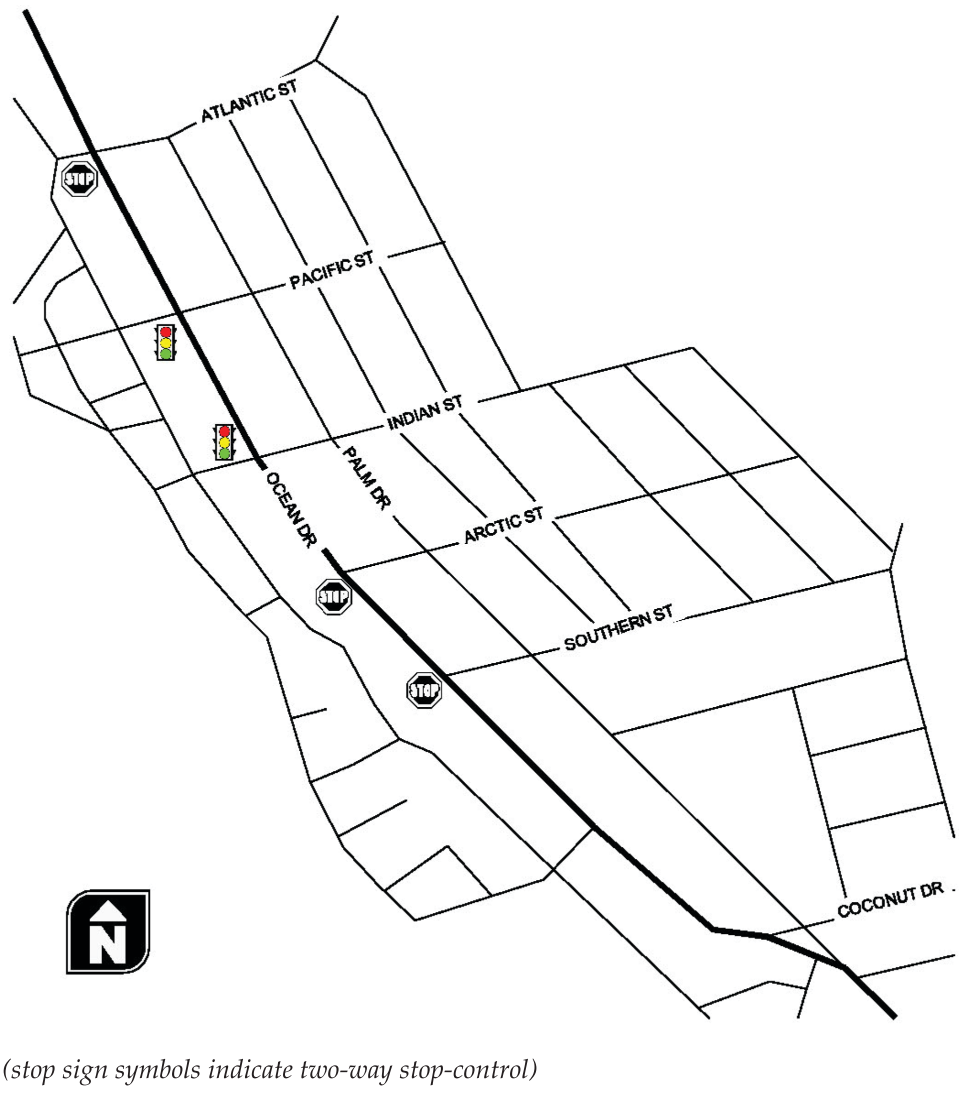
Evaluating the Performance of Corridors with Roundabouts

2.1.1. UNDERSTANDING OF CONTEXT

Ocean Drive is a five-lane roadway with two travel lanes in each direction, a two-way left-turn lane, and parallel on-street parking. The surrounding road network is a grid, and the ocean is two blocks west of the corridor. Beyond the roadway are sidewalks and businesses that primarily front the sidewalk directly. Some businesses with parking lots and residential units are located along the corridor. In the neighborhood surrounding Ocean Drive, development consists primarily of single-family housing, with some multifamily housing.

The intersections at Atlantic Street, Arctic Street, and Southern Street are two-way stop-controlled, and the intersections at Indian Street and Pacific Street are signalized. The intersections are between 600 and 800 feet apart. The corridor is illustrated in Exhibit 2-2.

Exhibit 2-2: Existing Corridor



(stop sign symbols indicate two-way stop-control)

2.1.2. USERS AND TRAFFIC VOLUME

Ocean Drive serves a variety of users. The ADT is 16,000 veh/day, and peak-hour, peak-direction flows range from 600 to 750 veh/h. The 85th-percentile

Evaluating the Performance of Corridors with Roundabouts

speeds along Ocean Drive are approximately 40 miles per hour. There is relatively little east-west traffic crossing the corridor. Peak-hour, bi-directional, cross-street volumes range from 50 to 100 vehicles per hour. The majority of vehicles on Ocean Drive have an origin or destination located within several miles of the project area; longer-distance traffic primarily uses a freeway located several miles inland. Crash rates on Ocean Drive over the past five years are below the state average for similar facilities.

Pedestrian volumes along Ocean Drive range from 50 to 100 persons during the peak hour. Pedestrian volumes crossing Ocean Drive at each intersection range from 35 to 80 persons during the peak hour. No bicycle counts are available. Observed bicycle activity suggests the majority of cyclists travelling through the area avoid Ocean Drive and instead ride on the streets one block to the east or west. Anecdotal evidence suggests riders are most comfortable travelling on these streets. Local bus service operates on Ocean Drive with 15-minute headways.

For brevity, the fictional example applications only include volumes from one year. Generally, a planning study for a corridor would forecast future, design-year volumes and practitioners would use these volumes for planning and analysis.

2.1.3. PROJECT CATALYST AND GOALS

Members of the community expressed a desire to improve the walkability and create a business-friendly atmosphere on Ocean Drive. Traffic speeds and the width of the roadway—approximately 70 feet curb-to-curb—make it difficult for pedestrians to cross Ocean Drive. Many businesses along the corridor lack parking lots and are patronized by area residents who walk to them or drivers who park on the street. Business owners state the limited on-street parking and poor pedestrian atmosphere of the corridor are negatively impacting their customer base. The owners have worked with the local community association to encourage the city to change the roadway to a slower-speed facility with a pedestrian-friendly configuration. The primary goals of the project are to:

- Improve walking and bicycling conditions,
- Increase the supply of on-street parking, and
- Maintain acceptable auto operations.

2.1.4. SELECT AND PRIORITIZE PERFORMANCE MEASURES

The sections below list the six groups of performance measures discussed in the CCD, and identify specific performance measures of importance on the Ocean Drive corridor. The performance measures identified below are not necessarily all that could be considered for the Ocean Drive project. There are many performance measures that could be used to evaluate a corridor. Some are of critical importance for nearly all corridors (Tier I), and others are only applicable to some corridors (Tiers II and III). For the purpose of illustrating the use of the CCD, this example presents performance measures that are of particular interest on the Ocean Drive corridor and help to distinguish the alternatives from one another. This includes Tier I measures like intersection level of service and cost,

Evaluating the Performance of Corridors with Roundabouts

and Tier II and III measures like crosswalk length and aesthetics. Performance measures of strong interest to the community are generally prioritized over those of lesser interest to the community.

2.1.4.1. *Quality of Service Performance Measures*

Quality of service refers to auto traffic operations and the experience of other corridor users such as pedestrians, bicyclists, and transit riders. Auto traffic operations are generally quantified with the procedures of the *Highway Capacity Manual*. The quality of service for other users is generally assessed qualitatively or with the multimodal procedures of the *Highway Capacity Manual.*

Traditional auto performance measures, such as intersection delay, volume-to-capacity ratio, and corridor travel time, should be considered in the Ocean Drive study to assess the potential for congestion. Additionally, some of the two-way stop-controlled streets currently experience high delay for the left-turn movement onto Ocean Drive. Side-street delay should also be considered.

Collectively, pedestrians, bicyclists, and bus riders account for 20 to 30 percent of the peak-hour trips along Ocean Drive. The current roadway is difficult for pedestrians to cross due to its width and the speed of traffic. The alternatives analysis should assess the length of the pedestrian crossings and the speed of vehicles on Ocean Drive.

Roundabout influence area, a component of the roundabout corridor travel-time procedure developed as part of this NCHRP project, is used to estimate the extents of the corridor over which travel speeds are lowered due to roundabouts.

Key Performance Measures: Crosswalk length, traffic speed, peak-hour intersection level of service, intersection v/c ratio, auto delay for minor-street left-turn movements.

2.1.4.2. *Safety Performance Measures*

Improving pedestrian and bicycle quality of experience is a goal of this project. Although the transportation profession generally lacks tools for quantitatively assessing pedestrian and bicycle safety, a number of surrogate safety measures can be considered. In the case of Ocean Drive, the community identified the roadway crossing distance and high traffic speeds—especially at two-way stop-controlled intersections—as challenges to pedestrians. Improvements in these elements can be considered as surrogates for pedestrian safety when evaluating alternatives.

Key Performance Measures: Crosswalk length (covered under traffic operations), traffic speed (covered under traffic operations), bicyclist comfort.

2.1.4.3. *Environmental Performance Measures*

This project will modify existing roadways in a developed urban area. No undeveloped land will be disturbed, and nearly all work for any of the alternatives would occur within the existing right-of-way. There are several parks and schools in the project area away from Ocean Drive. It is unlikely these facilities will be impacted.

Key Performance Measures: None.

2.1.4.4. *Cost Performance Measures*

Any build alternative will have capital construction and annual maintenance costs. Business and community associations indicated they will contribute to some maintenance needs such as landscaping and sidewalk cleaning. The anticipated impact on businesses in the corridor, including their profits and their impact on the tax base, is another cost to consider in the alternatives analysis.

Key Performance Measures: Construction cost, anticipated impact on businesses.

2.1.4.5. *Community Value Performance Measures*

The study of Ocean Drive began at the request of community members; therefore, an alternative should only be selected if it is embraced by the community. The community is particularly concerned about the pedestrian environment and the image of the corridor. Some property owners, particularly business owners, have a strong interest in preserving or increasing property values.

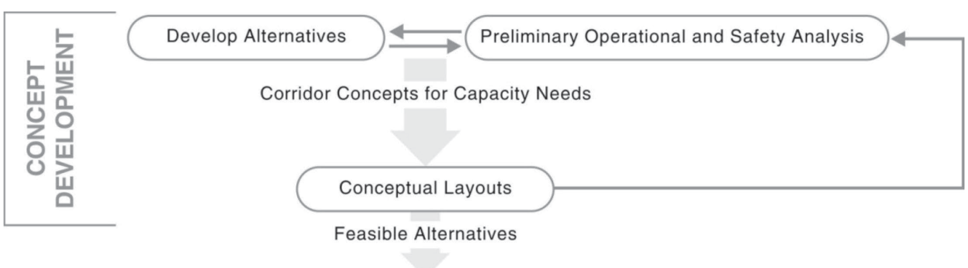
Key Performance Measures: Walkability, property value, aesthetics, community acceptance.

2.1.4.6. *Other Performance Measures*

On-street parking on Ocean Drive is highly used at certain times of the day, making it difficult for customers and residents to find open parking spaces.

Key Performance Measure: Parking supply.

2.2. CONCEPT DEVELOPMENT

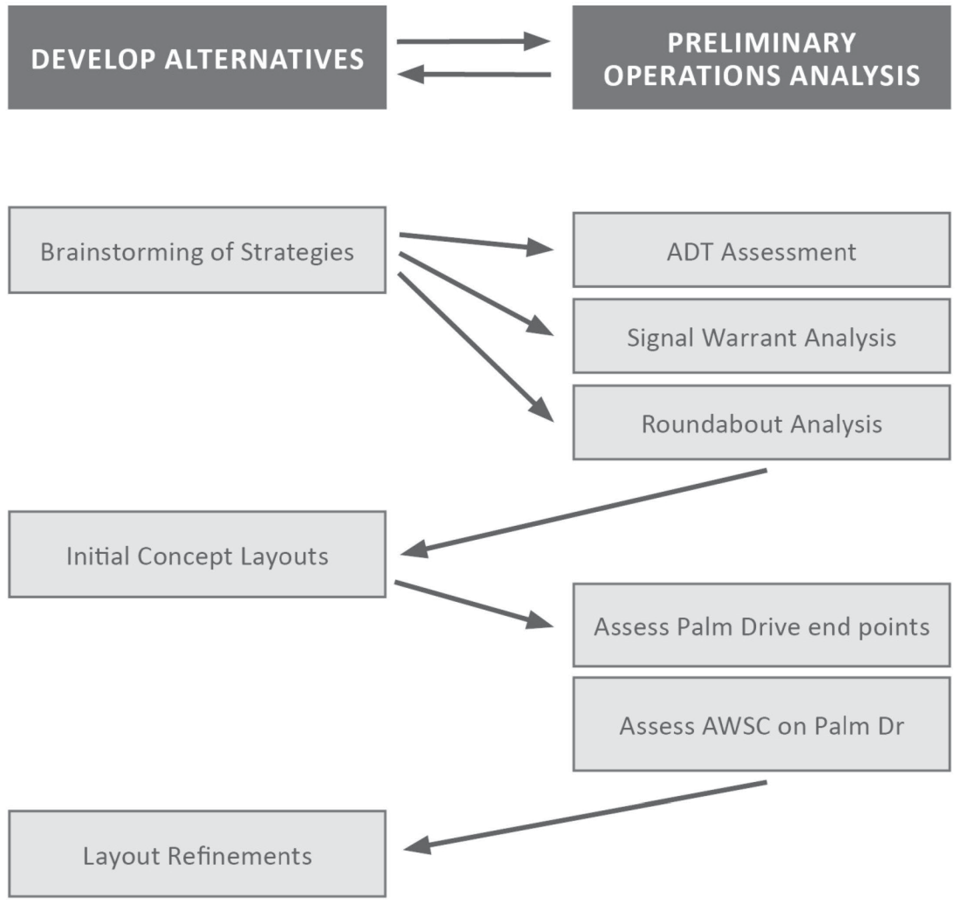


Steps in the Concept Development phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

2.2.1. DEVELOP ALTERNATIVES AND PRELIMINARY OPERATIONS ANALYSIS

Exhibit 2-3 illustrates the iterative process of developing alternatives and conducting the preliminary operations analysis as it occurred on the Ocean Drive project.

Exhibit 2-3: Iterative Process of Developing Alternatives and Conducting Preliminary Operations Analysis



2.2.1.1. *Brainstorming of Strategies*

The community-engagement process resulted in several strategies for achieving project goals:

- Road diet: Reduce the number of lanes on Ocean Drive, add bike lanes, potentially widen sidewalks, potentially narrow travel lanes.
- Couplet: Convert Ocean Drive to one-way (direction to be determined), and divert traffic travelling in the other direction to a parallel roadway.
- Traffic signals: Convert the three unsignalized major intersections along Ocean Drive to signal control.
- Roundabouts: Convert the five major intersections along Ocean Drive to roundabouts.

These strategies are commonly used tools a practitioner could consider to achieve project goals. They are not alternatives in the sense that each one is not necessarily a complete solution, and they may be combined in various ways. For example, a road diet could use traffic signals or roundabouts at intersections, or Ocean Drive could be converted to one-way with additional traffic signals. Prior to developing alternatives, city engineers assessed the viability of additional traffic signals or roundabouts on Ocean Drive using traffic volumes.

2.2.1.2. *Preliminary Operations Analysis*

Road Diet Strategy: The ADT of Ocean Drive is 16,000 veh/day. Other two-lane roadways in the city have an ADT greater than 20,000 veh/day and do not experience degraded operations. Based upon ADT, a road diet may be feasible on Ocean Drive.

Couplet Strategy: Streets one block to the east and to the west of Ocean Drive have a curb-to-curb width of 50 to 65 feet and a number of connections with Ocean Drive via the street grid. Converting a parallel street to carry one direction of Ocean Drive's traffic may be feasible.

Traffic Signal Strategy: Based upon peak-hour turning-movement counts, none of the unsignalized intersections meet the MUTCD's peak-hour signal warrant. Additionally, if peak-hour volumes were consistent for four or eight hours, they would not satisfy the four-hour vehicular volume or the eight-hour vehicular volume warrants. Hourly volumes over the course of four or eight hours will actually be lower than peak-hour volumes, indicating that these warrants are not satisfied. The most recent pedestrian counts, although several years old, indicate the pedestrian volume warrant is not met. Therefore, the city determined it was not feasible to add traffic signals at each intersection on Ocean Drive and no alternatives will include this strategy. Exhibits 2-4, 2-5, and 2-6 present the signal warrant analysis for Atlantic Street, the highest volume TWSC intersection on the corridor. In Exhibits 2-4 and 2-5, note that the minor-street volume (shown on the *y*-axis) does not meet the minimum value of 100 vehicles per hour (Exhibit 2-4) or 80 vehicles per hour (Exhibit 2-5), so no analysis of the major-street volume is necessary.

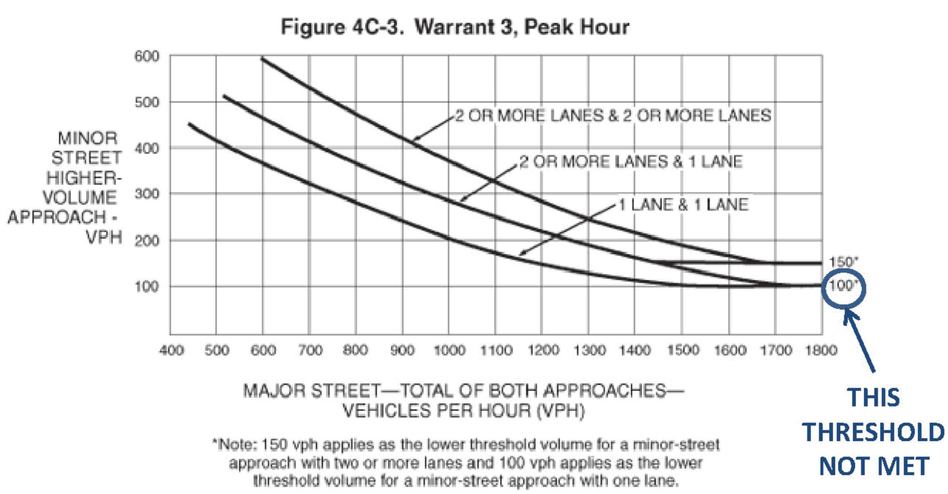
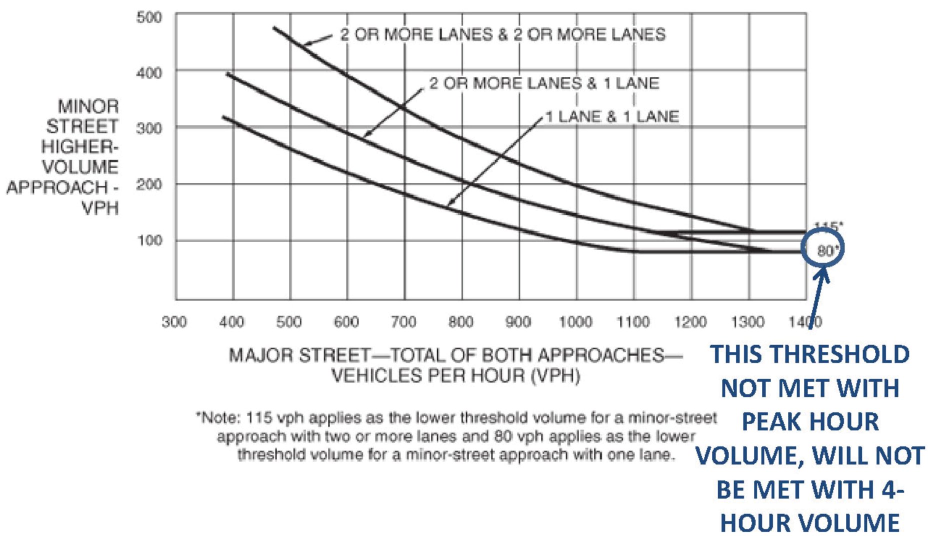


Exhibit 2-4: Analysis of Peak-Hour Vehicular Volume Signal Warrant at Highest-Volume Intersection Under Road Diet Concept (reproduced from 2009 MUTCD Figure 4C-3)

Exhibit 2-5: Analysis of Four-Hour Vehicular Volume Signal Warrant at Highest-Volume Intersection Under Road Diet Concept (reproduced from 2009 MUTCD Figure 4C-1)

Figure 4C-1. Warrant 2, Four-Hour Vehicular Volume



*Note: 115 vph applies as the lower threshold volume for a minor-street approach with two or more lanes and 80 vph applies as the lower threshold volume for a minor-street approach with one lane.

Exhibit 2-6: Analysis of Eight-Hour Vehicular Volume Signal Warrant at Highest-Volume Intersection Under Road Diet Concept (reproduced from 2009 MUTCD Table 4C-1)

Table 4C-1. Warrant 1, Eight-Hour Vehicular Volume

Condition A—Minimum Vehicular Volume

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	500	400	350	280	150	120	105	84
2 or more	1	✔	✔	420	336	✗	✗	105	84
2 or more	2 or more	600	480	420	336	200	160	140	112
1	2 or more	500	400	350	280	200	160	140	112

Condition B—Interruption of Continuous Traffic

Number of lanes for moving traffic on each approach		Vehicles per hour on major street (total of both approaches)				Vehicles per hour on higher-volume minor-street approach (one direction only)			
Major Street	Minor Street	100%[a]	80%[b]	70%[c]	56%[d]	100%[a]	80%[b]	70%[c]	56%[d]
1	1	750	600	525	420	75	60	53	42
2 or more	1	✔	✔	630	504	✗	✔	53	42
2 or more	2 or more	900	720	630	504	100	80	70	56
1	2 or more	750	600	525	420	100	80	70	56

[a] Basic minimum hourly volume
[b] Used for combination of Conditions A and B after adequate trial of other remedial measures
[c] May be used when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000
[d] May be used for combination of Conditions A and B after adequate trial of other remedial measures when the major-street speed exceeds 40 mph or in an isolated community with a population of less than 10,000

PEAK HOUR VOLUMES DO NOT SATISFY THIS WARRANT, THEREFORE 8 HOUR VOLUMES WILL NOT

Roundabout Strategy: City engineers evaluated the geometric and operational feasibility of roundabouts. Exhibit 2-7 presents the available right-of-way at an intersection along Ocean Drive. Constraints at other intersections are similar.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-7: Available Distance Between Buildings at a Typical Intersection on Ocean Drive

The 125-foot dimension depicted in Exhibit 2-7 would accommodate an inscribed circle diameter (ICD) of approximately 110 feet, plus sidewalks beyond the roundabout. Guidance in *NCHRP Report 672* identifies the common range of single-lane roundabout ICDs as 90 to 180 feet depending on the design vehicle. A roundabout with a 110-ft ICD will generally accommodate a city bus. The diameter would need to be increased to accommodate a truck such as a WB-67. City engineers determined a single-lane roundabout is potentially geometrically feasible, and larger trucks such as WB-67s, which are uncommon on Ocean Drive, can be directed to parallel streets.

To assess the operational feasibility of single-lane roundabouts, city engineers used Exhibit 3-14 of *NCHRP Report 672* (reproduced here as Exhibit 2-8). Total entering peak-hour volumes at each intersection range from 1,000 to 1,100 veh/h, with the majority of traffic making through movements on Ocean Drive. Therefore, single-lane roundabouts may be operationally feasible, but a more detailed analysis should be conducted to confirm this. Multilane entry configuration would require a larger ICD. The larger ICD would require acquiring buildings, which is not considered feasible on this corridor.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-8: Planning-Level Analysis of Roundabout Lane Needs (reproduced from *NCHRP Report 672* Exhibit 3-14)

Volume Range (sum of entering and conflicting volumes)	Number of Lanes Required
0 to 1,000 veh/h	▪ Single-lane entry likely to be sufficient
1,000 to 1,300 veh/h	▪ Two-lane entry may be needed ▪ Single-lane may be sufficient based upon more detailed analysis.
1,300 to 1,800 veh/h	▪ Two-lane entry likely to be sufficient
Above 1,800 veh/h	▪ More than two entering lanes may be required ▪ A more detailed capacity evaluation should be conducted to verify lane numbers and arrangements.

2.2.1.3. *Initial Layouts*

Based on the initial operational analysis of the strategies, the city developed three concepts for corridor improvements:

1. Alternative #1 is a road diet converting Ocean Drive from a five-lane section to a three-lane section. The two-way left-turn lane and one travel lane in each direction are preserved, bicycle lanes are added, and, on some blocks, parallel parking is converted to angle parking. Intersection control remains the same as existing conditions (see Exhibit 2-2).
2. Alternative #2 creates a one-way couplet. Palm Drive, located one block east of Ocean Drive, is converted to one-way northbound with two travel lanes and on-street parking on both sides of the street. Ocean Drive is converted to two southbound travel lanes, and a curb-separated parking area provides angle and parallel parking. Northbound traffic is transitioned to and from Ocean Drive one-to-two blocks north and south of the study area where connections already exist. Control on Ocean Drive remains the same as existing conditions (see Exhibit 2-2). Control on Palm Drive will be determined at a later stage of the planning process.
3. Alternative #3 is a road diet adding roundabouts at each of the five intersections in the Ocean Drive study area. Ocean Drive is reduced to one travel lane in each direction, a median is added, bicycle lanes are added, and some parallel parking is converted to angle parking.

2.2.1.4. *Additional Preliminary Operations Analysis*

While considering the corridor strategies, two potential operational issues became apparent with the one-way couplet:

- With increased traffic on Palm Drive, it may be appropriate to convert the AWSC intersections on Palm Drive to TWSC.

- Detailed analysis of the operations and geometrics at the northern and southern ends of the couplet will be needed. At the southern end, Palm Drive will need to be connected to the Ocean Drive/Coconut Lane intersection, and the intersection will need to be reconfigured. At the northern end, the transition of northbound traffic back to Ocean Drive via Atlantic Street should be studied.

These issues are site-specific and cannot be assessed with a "rule of thumb" or planning-level analysis. The city decided to retain the couplet strategy (as Alternative 2) and explore the issues as part of the alternatives analysis.

2.2.2. CONCEPTUAL LAYOUTS

Exhibits 2-9 through 2-11 depict the conceptual layouts of the three alternatives.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-9: Typical Ocean Drive Intersection and Road Segment, Alternative 1

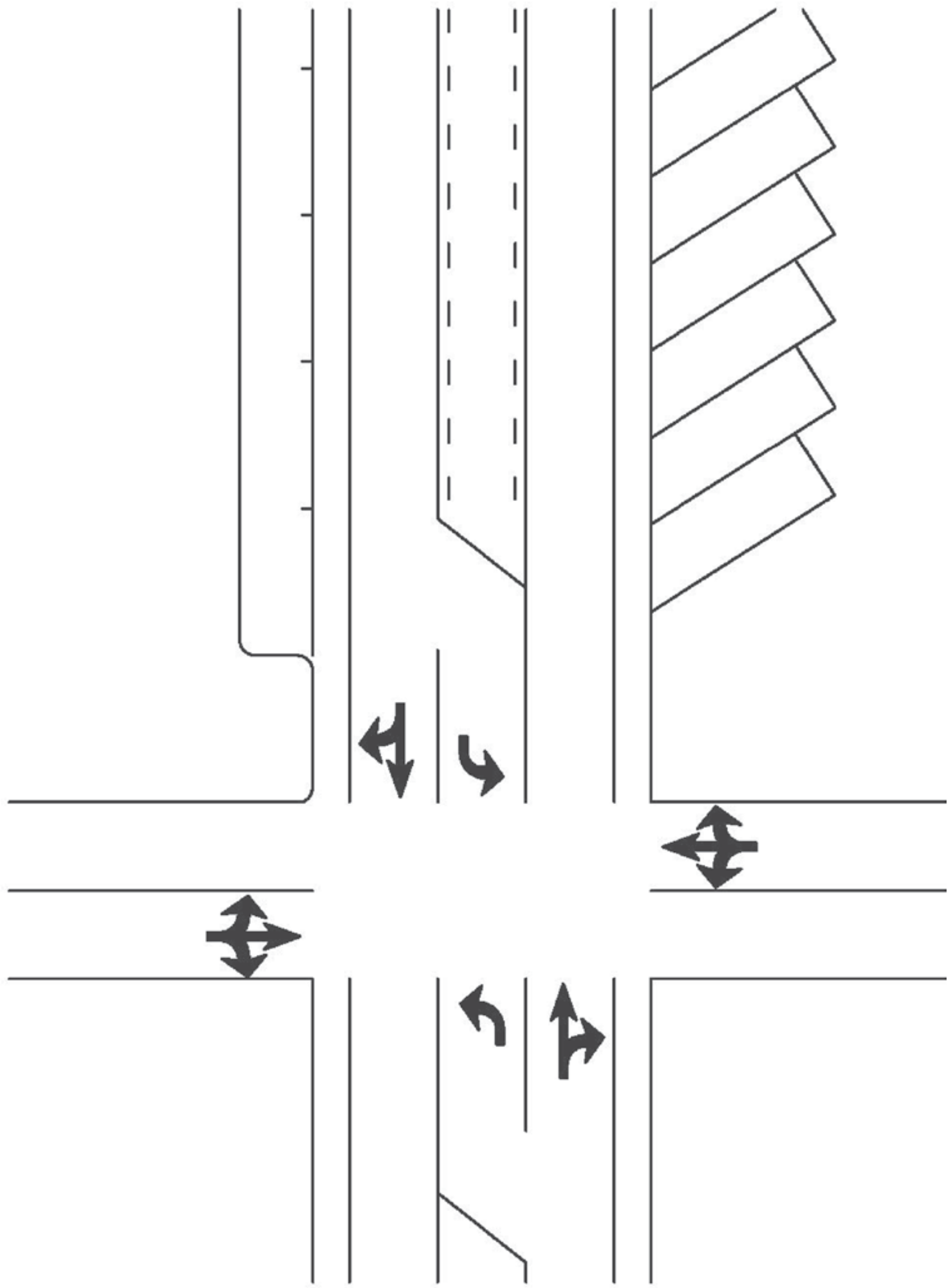
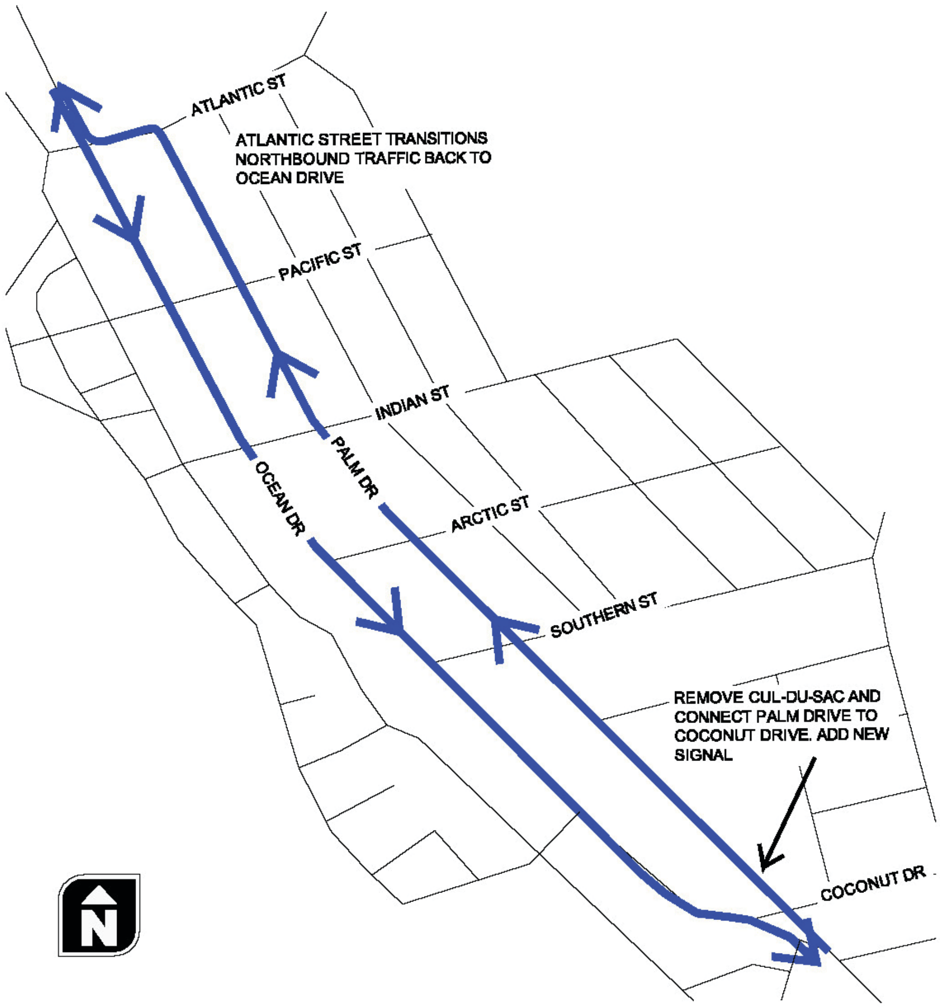
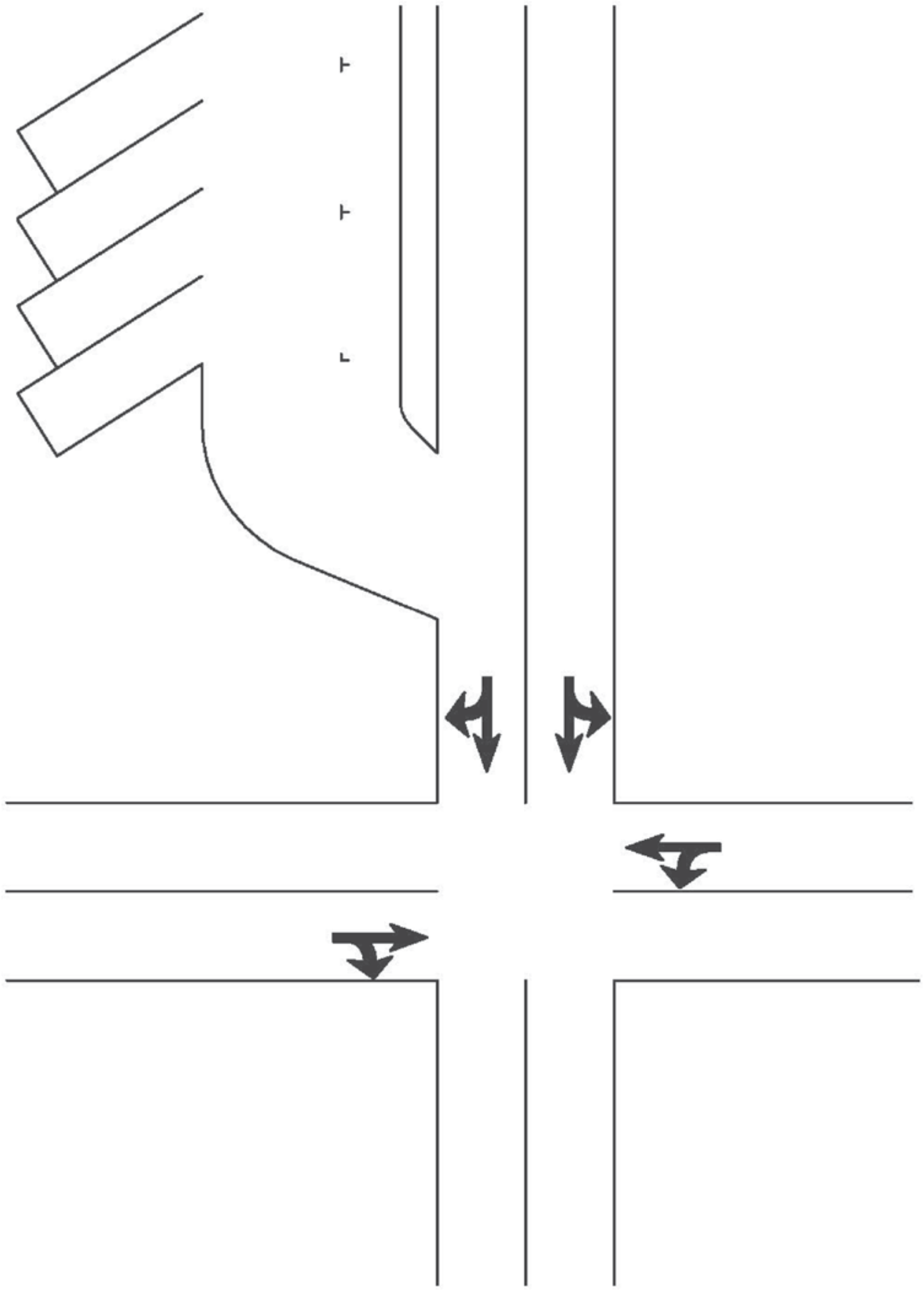


Exhibit 2-10A: Alternative 2 Overview



Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-10B: Typical Ocean Drive Intersection and Road Segment, Alternative 2



Evaluating the Performance of Corridors with Roundabouts

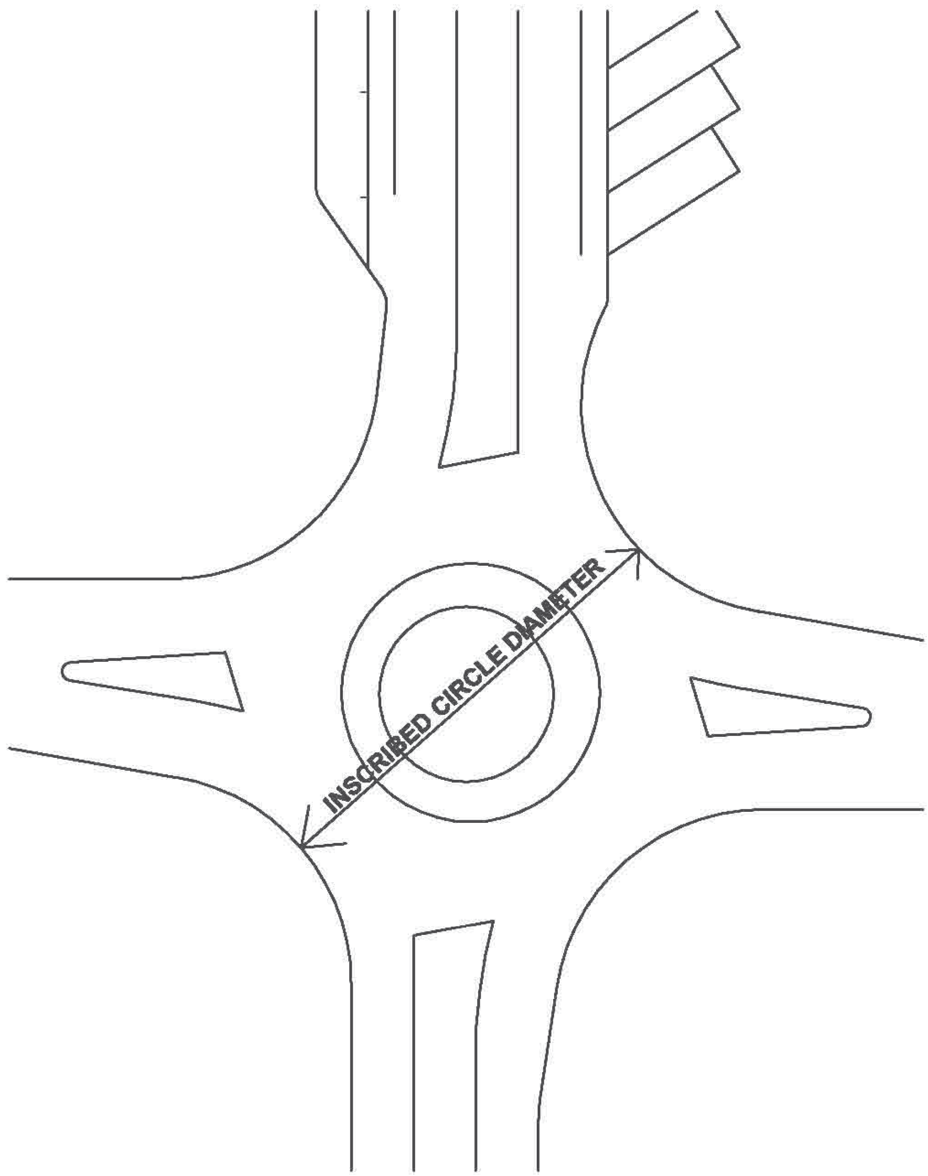
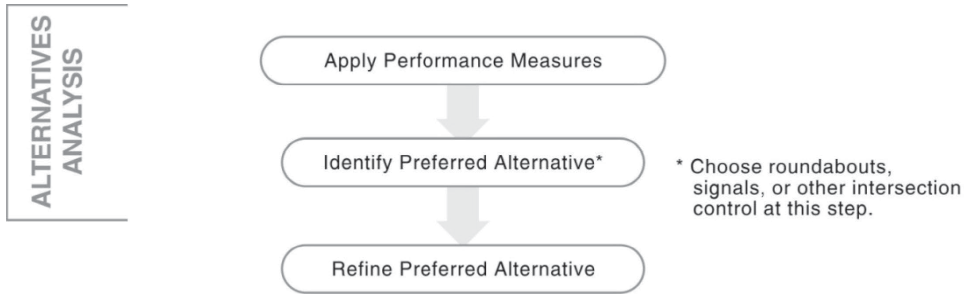


Exhibit 2-11: Typical Ocean Drive Intersection and Road Segment, Alternative 3

2.3. ALTERNATIVES ANALYSIS



Steps in the Alternatives Analysis phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

2.3.1. EVALUATE THE ALTERNATIVES

Exhibit 2-12 summarizes an analysis of the three alternatives proposed for Ocean Drive using the key performance measures identified in Section 2.1.4.

Exhibit 2-12: Alternatives Analysis

Perfor-mance Measure	**Alternative 1 – Road Diet with Signals and TWSC**	**Alternative 2 – One-Way Couplet with Signals and TWSC**	**Alternative 3 – Road Diet with Roundabouts**	**Comments**
Crosswalk length	The number of lanes being crossed decreases, and the overall crossing distance may decrease if curb extensions are used.	Pedestrians will cross one direction of traffic and two travel lanes. The entrances and exits to the median-separated parking areas may be confusing and challenging to pedestrians.	Crossings will become two-stage; each stage will only cross one lane of traffic.	Each alternative appears to address this performance measure.
Traffic speed *(see Section 2.3.1.1)*	Reducing travel lanes may slow traffic by changing the character of the roadway and increasing the density of traffic. However, there are no geometric features that reinforce the desired speed reduction.	This design may increase the speed of traffic by physically separating on-street parking from the travel lanes on Ocean Drive. Speeds may increase on Palm Drive as it becomes a higher-order roadway and some AWSC intersections become TWSC.	Roundabouts slow drivers to 25 mph or less at every intersection. The roundabouts are 600 to 800 feet apart, and roundabout influence areas are nearly this long. Roundabouts will reduce speeds on most of the corridor. *Calculations for roundabout influence area lengths are shown in Section 2.3.1.1.*	Roundabouts appear to provide the best speed management technique, followed by the road diet. The couplet may potentially be worse than existing conditions.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-12: Alternatives Analysis Con't

Performance Measure	Alternative 1 – Road Diet with Signals and TWSC	Alternative 2 – One-Way Couplet with Signals and TWSC	Alternative 3 – Road Diet with Roundabouts	Comments
Peak-hour intersection LOS	During the a.m. and p.m. peak hours, the two signalized intersections will operate at LOS A or B and the TWSC intersections will operate at C, D, or E. Ocean Drive/Atlantic Street operates at LOS E during the p.m. peak hour. *Existing signals and stop-control were left in place.*	During the a.m. and p.m. peak hours, the two signalized intersections will operate at LOS A or B and the TWSC intersections will operate at LOS B or C. *Analysis assumed AWSC intersections on Palm Drive are converted to TWSC, the left turn from Palm Drive to Atlantic Street is uncontrolled, and existing signals and stop-control were left in place on Ocean Drive.*	During the a.m. and p.m. peak hours, the roundabouts will operate at LOS B or C.	Each concept performs acceptably during the peak hours. Roundabouts may offer less delay during non-peak conditions compared to the other concepts.
Intersection v/c ratio	During the a.m. and p.m. peak hours, the v/c of the intersections (TWSC and signalized) will be 0.64 or less. *Same assumptions as "Delay & LOS."*	During the a.m. and p.m. peak hours, the v/c of the intersections (TWSC and signalized) will be 0.48 or less. *Same assumptions as "Delay & LOS."*	During the a.m. and p.m. peak hours, the v/c of the roundabouts will range from 0.52 to 0.74.	Each concept performs acceptably. The couplet has lower v/c ratios than the other concepts.
Minor-street left-turn delay	During the a.m. and p.m. peak hours, the minor-street left-turn delay will be 38 seconds or less at TWSC intersections. *Same assumptions as "Delay & LOS."*	During the a.m. and p.m. peak hours, the minor-street left-turn delay will be 22 seconds or less at TWSC intersections. *Same assumptions as "Delay & LOS."*	During the a.m. and p.m. peak hours, minor-street delay for any movement at any roundabout will be 10 seconds or less.	Roundabouts have the lowest delay, followed by the couplet.
Bicyclist comfort	The road diet creates a bicycle lane in each direction on Ocean Drive. In some areas, bicyclists ride behind angle parking, which may reduce their visibility.	Southbound bicyclists on Ocean Drive ride through the parking area and are separated from through auto traffic. Northbound bicyclists share a lane with autos on Palm Drive, as they do on Ocean Drive today.	Between intersections, conditions are similar to Alternative 1 (including the potential for reducing visibility) but with lower auto speeds. Roundabouts may improve intersection comfort for side-street bicyclists at intersections that are currently TWSC.	Each alternative improves comfort; the roundabouts may create lower vehicle speeds.
Estimated construction cost	$3 million.	$10 million.	$6 million.	Alternative 1 has the lowest cost, followed by Alternative 3.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-12: Alternatives Analysis Con't

Perfor-mance Measure	Alternative 1 – Road Diet with Signals and TWSC	Alternative 2 – One-Way Couplet with Signals and TWSC	Alternative 3 – Road Diet with Roundabouts	Comments
Anticipated impact on businesses	Business owners generally believe this alternative would result in a modest increase in profits due to the increased parking and enhanced pedestrian environment.	Business owners are concerned that removing northbound traffic from Ocean Drive would decrease the visibility of their businesses and thus their profits. A few business owners are supportive of this plan because it would create well-defined parking areas.	Business owners generally believe this alternative would result in an increase in profits, and the increase would be greater than under Alternative 1 because of the greater investment in and appearance of the corridor. A few business owners are concerned that drivers and pedestrians will be uncomfortable with the roundabouts and avoid the area.	Business owners are generally supportive of the road diet, with or without roundabouts.
Walkability	The road diet would improve walkability by reducing crossing length, providing ROW for sidewalk improvements, and introducing a facility for another active transportation mode (the bicycle lanes). However, crossing Ocean Drive might remain challenging at TWSC intersections.	A one-way Ocean Drive would have fewer lanes and likely more gaps for pedestrian crossings. However, vehicle speeds are unlikely to be reduced and separation of parking from other roadway elements creates an environment that is generally more auto-centric and less walkable.	The walkability of this alternative will be similar to Alternative 1, although vehicles will travel slower at roundabouts than at TWSC intersections.	The road diet with or without roundabouts appears to improve walkability. Roundabouts may reduce speeds, making Ocean Drive crossings easier.
Property value	Most property owners believe this alternative will result in a modest increase in property value because of the increased parking supply and enhanced walkability. Some believe it will have no impact.	Owners of property on Palm Drive are concerned that increased traffic will decrease their property value. Residential property owners on Ocean Drive do not anticipate significant impacts to property value. Commercial property owners on Ocean Drive are concerned that reduced traffic may decrease their property value.	Most property owners believe this alternative will result in a modest increase in property value, similar to Alternative 1. A few are concerned that roundabouts will decrease property value.	Property owners believe the road diet with or without roundabouts will increase property value and the couplet will decrease property value.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 2-12: Alternatives Analysis Con't

Performance Measure	Alternative 1 – Road Diet with Signals and TWSC	Alternative 2 – One-Way Couplet with Signals and TWSC	Alternative 3 – Road Diet with Roundabouts	Comments
Aesthetics	By changing the cross-section of the roadway, this alternative creates the opportunity for aesthetic improvements such as landscaping and decorative pavement.	This alternative creates the opportunity for aesthetic improvements on both Ocean Drive and Palm Drive. However, the dedicated parking areas on Ocean Drive may be visually unappealing.	This alternative offers greater aesthetic potential than Alternative 1. The roundabouts may be landscaped, and they may also define the corridor and provide a sense of place.	All alternatives create opportunity for aesthetic improvements.
Community acceptance	Community members generally feel this alternative improves the corridor but fails to address vehicular and pedestrian concerns at the TWSC intersections.	The majority of the community is opposed to this alternative. Palm Drive residents are concerned about increased traffic and Ocean Drive business owners are concerned about decreased traffic. Reaction to the dedicated parking areas is mixed.	Most community members prefer this alternative and feel it offers the greatest potential to improve the image and walkability of the corridor. It addresses their concerns about the TWSC intersections by removing them. A few community members have concerns about roundabouts and are strongly opposed to this alternative.	The community generally prefers roundabouts and is opposed to the couplet. Public outreach and education may help address concerns about roundabouts.
Parking supply	This alternative adds 48 parking spaces, a 23% increase.	This alternative adds 100 parking spaces, a 48% increase.	This alternative adds 38 parking spaces, an 18% increase.	The couplet adds the most parking, followed by Alternative 1.

2.3.1.1. *Roundabout Influence Area Length Computation Example*

The roundabout influence area was calculated for one of the roundabouts on Ocean Drive to get a sense of the extent of areas in which speeds will be reduced due to the presence of roundabouts. Data used in these calculations are listed below:

- Upstream sub-segment length = 350 feet;
- Downstream sub-segment length = 420 feet;
- Central island diameter = 50 feet;
- Circulating speed = 20 mph; and
- The influence areas do not overlap (see Step B of the modeling framework in Chapter 3 of the main report).

First, the free-flow speed was determined using the upstream and downstream sub-segment models:

Evaluating the Performance of Corridors with Roundabouts

$$S_{f,US} = 15.1 + 0.0037 \times (350\ feet) + 0.43 \times (40\ mph) + 0.05 \times (50\ feet)$$

= 36.1 mph

$$S_{f,DS} = 14.6 + 0.0039 \times (420\ feet) + 0.48 \times (40\ mph) + 0.02 \times (50\ feet)$$

= 36.4 mph

Using free-flow speed values, the corresponding roundabout influence areas can be computed as follows:

$$RIA_{US} = 165.9 + 13.8 \times (36.1\ mph) - 21.1 \times (20\ mph)$$

= 242 feet

$$RIA_{DS} = -149.8 + 31.4 \times (36.4\ mph) - 22.5 \times (20\ mph)$$

= 544 feet

These values indicate that the influence areas do not overlap, so the assumption in Step B was correct.

Intersections on Ocean Drive are spaced between 600 and 800 feet apart. Most of the corridor will lie within a roundabout influence area, meaning that the geometry of roundabouts will limit speeds along most of the corridor.

2.3.2. IDENTIFY PREFERRED ALTERNATIVE

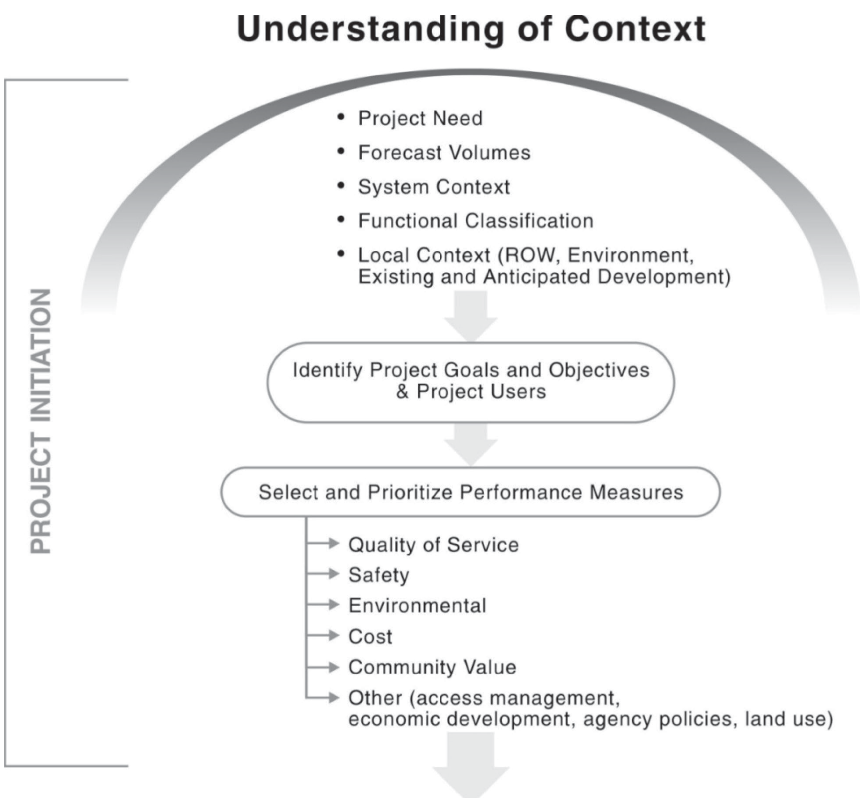
Alternative 2 (the one-way couplet) was eliminated based upon negative feedback from the community. It was also the most expensive alternative.

Many in the community supported Alternative 1 (road diet with existing traffic control). Most supported Alternative 3 as most desirable, as it would provide greater benefits in terms of traffic calming and the overall image of the corridor, even if it offered less additional parking than Alternative 1. Several roundabouts had been installed in other neighborhoods of the city, and both residents and city staff members were pleased with their performance, which made the city willing to install more. The city selected Alternative 3 based on strong support from the community. Residents and business owners advocated for Alternative 3 based on their belief it would improve the business environment on the corridor and improve aesthetics.

EXAMPLE APPLICATION 3. US 7

This fictional example application presents a context-sensitive access-management project on a rural corridor beginning to suburbanize.

3.1. PROJECT INITIATION



Steps in the Project Initiation phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

Exhibit 3-1: Key Data

Cross-Section	2 lanes
Travel Lanes	1 each direction
Corridor Length	3.25 miles
Forecast ADT	14,000 veh/day
Forecast Peak-Hour Peak-Direction Flow	450 to 600 veh/h
85th Percentile Speed	46 mph
Existing Control	Signal at US 7/SR 272, TWSC elsewhere
Peak-Hour Pedestrian Volume Along US 7	5 to 30 p/h
Peak-Hour Pedestrian Volume Crossing US 7	10 to 25 p/h per intersection
Sidewalks	Varies
Bicycle Lanes	No
Local Bus Service	No
Land Use	Rural, changing to suburban

Evaluating the Performance of Corridors with Roundabouts

3.1.1. UNDERSTANDING OF CONTEXT

Elk Grove is a historically rural town that experienced substantially increased population growth the past decade. A metropolitan area with three million people lies 75 miles to the south, and Elk Grove emerged as a popular location for second homes and retirement homes due to its natural beauty and rural character. US 7 is a two-lane roadway through Elk Grove, and improvements to a 3.25-mile segment are under consideration. Due to increased growth in the area, the state DOT wants to address access-management issues on US 7 in Elk Grove.

US 7 is the primary link between Elk Grove and the metropolitan area to the south. It is a two-lane, rural highway. Fifteen years ago, the state DOT improved a mountainous, four-mile section of US 7 immediately south of Elk Grove and added truck climbing lanes. Within Elk Grove, there have been few improvements to the roadway in recent decades. Some businesses have parking lots without defined driveways directly fronting the roadway. Some houses lack driveways, and residents parallel-park along the edge of the roadway. Two two-way stop-controlled intersections have left-turn lanes on US 7. The only signal on US 7 in the area is at SR 272; the DOT signalized this intersection over 30 years ago. A half-mile section of US 7 in Elk Grove has sidewalks that are in need of repair. There are no dedicated bicycle facilities on US 7. The town is interested in pedestrian and bicycle enhancements as part of the access-management project. The project area is shown in Exhibit 3-2.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-2: Existing Corridor

3.1.2. USERS AND TRAFFIC VOLUME

US 7 serves a variety of users. The design-year forecast ADT within Elk Grove is 14,000 veh/day, and forecast peak-hour, peak-direction flows range from 450 to 600 veh/h. Eighty-fifth-percentile speeds along US 7 are approximately 46 miles

Evaluating the Performance of Corridors with Roundabouts

per hour. There is relatively little east-west traffic crossing the corridor, and only SR 272 has more than 150 side-street vehicles in the peak hour. Crash rates on US 7 over the past five years are near the state average for similar facilities.

Pedestrian volumes along US 7 range from 5 to 30 persons during the peak hour, and pedestrian volumes crossing US 7 at each intersection range from 10 to 25 persons during the peak hour. Pedestrian activity is highest in the southern portion of Elk Grove. No bicycle counts are available and there is no local bus service.

For brevity, the fictional example applications only include volumes from one year. Generally, a planning study for a corridor would forecast future, design-year volumes and practitioners would use these volumes for planning and analysis.

3.1.3. PROJECT CATALYST AND GOALS

The DOT is concerned that ongoing growth in Elk Grove may lead to increased access onto US 7, potentially decreasing operational and safety performance. The town wishes to improve the aesthetics of US 7, which serves as their Main Street. Residents desire sidewalks and bicycle lanes and have identified and shared their difficulty crossing US 7 at TWSC intersections. The primary goals of the project are to:

- Improve access management,
- Improve the aesthetics, and
- Provide pedestrian and bicycle facilities.

3.1.4. SELECT AND PRIORITIZE PERFORMANCE MEASURES

The sections below list the six groups of performance measures discussed in the CCD, and identify specific performance measures of importance on the US 7 corridor. The performance measures identified below are not necessarily all that could be considered for the US 7 project. There are many performance measures that could be used to evaluate a corridor. Some are of critical importance for nearly all corridors (Tier I), and others are only applicable to some corridors (Tiers II and III). For the purpose of illustrating the use of the CCD, this example presents performance measures of particular interest on the US 7 corridor and that help to distinguish the alternatives from one another. This includes Tier I measures like intersection level of service and cost, and Tier II and III measures like impacts to public facilities and livability. Performance measures of strong interest to the community are generally prioritized over those of lesser interest to the community.

3.1.4.1. *Quality of Service Performance Measures*

Quality of service refers to auto traffic operations and the experience of other corridor users such as pedestrians, bicyclists, and transit riders. Auto traffic operations are generally quantified with the procedures of the *Highway Capacity Manual*. The quality of service for other users is generally assessed qualitatively or with the multimodal procedures of the *Highway Capacity Manual.*

Evaluating the Performance of Corridors with Roundabouts

The DOT has level of service and volume-to-capacity (v/c) ratio standards for new or reconstructed state highways, and DOT policy requires these standards to be assessed at the planning stage of the project. Currently, pedestrians have difficulty crossing US 7 due to the high vehicle speeds and few gaps in the traffic stream.

Key Performance Measures: Intersection level of service, intersection capacity, arterial capacity, availability of gaps for pedestrian crossings.

3.1.4.2. *Safety Performance Measures*

US 7 currently operates acceptably from a safety perspective. The state DOT wants to maintain this level of performance into the future, when volumes are forecast to increase. This desire may be partially addressed through access-management improvements (see Section 3.1.4.6) that preserve available segment capacity and enhance traffic flow by reducing conflicts and friction at driveways. Reducing driveway conflicts on US 7 is beneficial for bicyclists and pedestrians traveling along US 7. Pedestrian and bicycle activity is expected to increase if improvements are made on the corridor. Treatments that promote pedestrian and cyclist safety could be integrated into potential project solutions.

Key Performance Measures: Conflict points, auto/bicycle speed differential, pedestrian level of service, bicycle level of service.

3.1.4.3. *Environmental Performance Measures*

Much of the land surrounding Elk Grove is publicly owned. Many residents and visitors take advantage of the close proximity of this land and the recreational opportunities it offers. The Red River is the largest river in this part of the state and is a valued recreational and natural resource.

Key Performance Measure: Impacts to public facilities.

3.1.4.4. *Cost Performance Measures*

Any build alternative will have capital construction costs, although they may vary greatly. Improvements expanding the footprint of the roadway or using a new alignment will require buying right-of-way.

Key Performance Measures: Right-of-way acquisition cost, construction cost.

3.1.4.5. *Community Value Performance Measures*

US 7 is the Main Street of Elk Grove and a key element of the town's identity. Some residents and town officials are concerned the current roadway appearance negatively impacts people's perceptions of the community. The poor condition of sidewalks, where they exist, discourages residents from walking. Curbs, pavement, and other elements of the roadway are in poor condition as well. These conditions detract from the Main Street image of US 7. Residents believe it is an opportune time to address aesthetics as part of the DOT's access-management efforts.

Key Performance Measures: Livability, walkability, aesthetics, community acceptance.

Evaluating the Performance of Corridors with Roundabouts

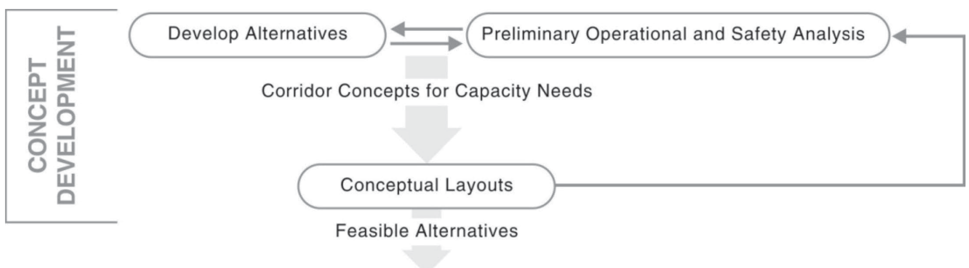
3.1.4.6. *Other Performance Measures*

The DOT initiated this project to better manage access on US 7. The following access-management issues are present:

- Side streets and driveways are full access (no left-turn restrictions).
- Some parking lots continuously front the roadway, with no defined driveways.
- Driveways are generally unconsolidated, with each house and business having a dedicated access.
- Drivers park on the shoulders of the roadway and within DOT right-of-way off the shoulders.

Key Performance Measure: Access management (such as driveway closures or consolidations and better defining driveway accesses).

3.2. CONCEPT DEVELOPMENT



Steps in the Concept Development phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

3.2.1. DEVELOP ALTERNATIVES AND PRELIMINARY OPERATIONS AND SAFETY ANALYSIS

The DOT considered three alternatives for US 7:

1. Alternative #1 is a three-lane roadway with a two-way left-turn lane (TWLTL) and bicycle lanes. Sidewalks are added. Three traffic signals are added at intersections where warrants are satisfied, and some driveways are consolidated to direct traffic to the signals.
2. Alternative #2 varies between a two-lane roadway with a raised median and a three-lane roadway with left-turn lanes. Sidewalks and bicycle lanes are added. Five roundabouts are added, and some driveways are consolidated to direct traffic to the roundabouts.
3. Alternative #3 is a bypass from south of Elk Grove to SR 272 that reduces regional trips on existing US 7 in Elk Grove. Sidewalks and sharrows are added to existing US 7 in Elk Grove.

The DOT used a software program implementing the HCM signalized intersection procedure to assess future lane needs for Alternative 1. DOT staff assumed a single-lane approach on side streets and a two-lane approach with a

Evaluating the Performance of Corridors with Roundabouts

through-right lane and a left-turn lane on US 7. The analysis determined this lane configuration is adequate to meet DOT performance criteria.

The DOT used Exhibits 3-12 and 3-14 of *NCHRP Report 672* to assess roundabout lane needs for Alternative 2 based upon ADT and peak-hour volume. Use of Exhibit 3-12 was previously shown in Example Application 1 (Exhibit 1-3) and use of Exhibit 3-14 was previously shown in Example Application 2 (Exhibit 2-8). DOT staff determined single-lane roundabouts will adequately serve forecast traffic on US 7.

Under Alternative 3, traffic forecasts call for an ADT of 9,000 on the new bypass and 5,000 on existing US 7. The state's transportation planning guidelines recommend that new rural roadways with an ADT of 15,000 or higher have four lanes. The forecast for the new bypass is below that threshold, and a two-lane roadway will be sufficient. Some three-lane sections with truck climbing lanes may be desirable on either side of the Red River crossing.

3.2.2. CONCEPTUAL LAYOUTS

Following the initial operational checks, the DOT developed two conceptual layouts for each alternative. One layout is an overview of the entire corridor, and the other is a more detailed concept for a short segment of the corridor several blocks in length. The detailed concepts were used at a public meeting and provided a visualization of specific design elements of each alternative.

Exhibits 3-3 through 3-7 depict the conceptual layouts of the three alternatives.

Exhibit 3-3: Alternative 1 Overview

Evaluating the Performance of Corridors with Roundabouts

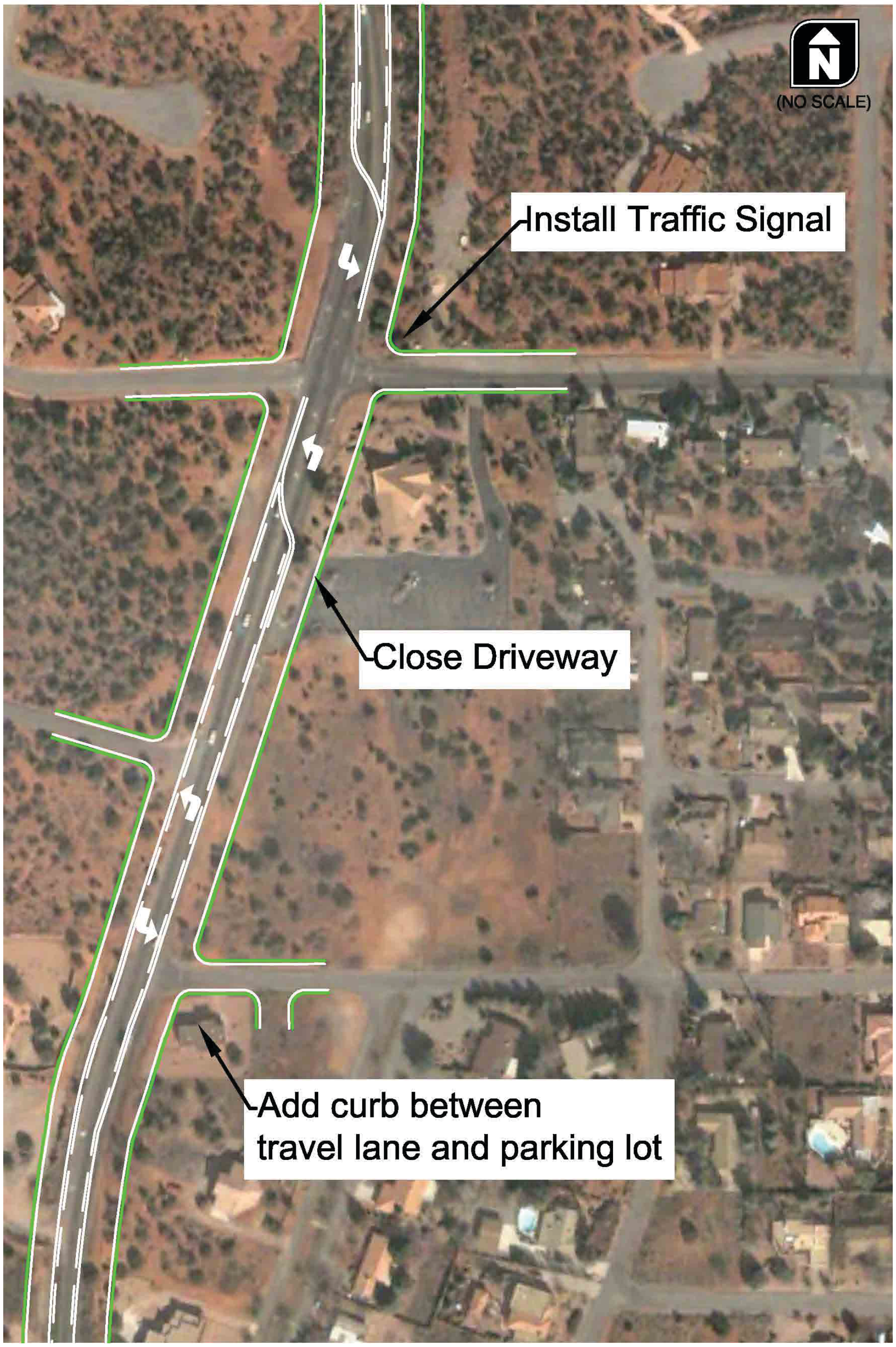
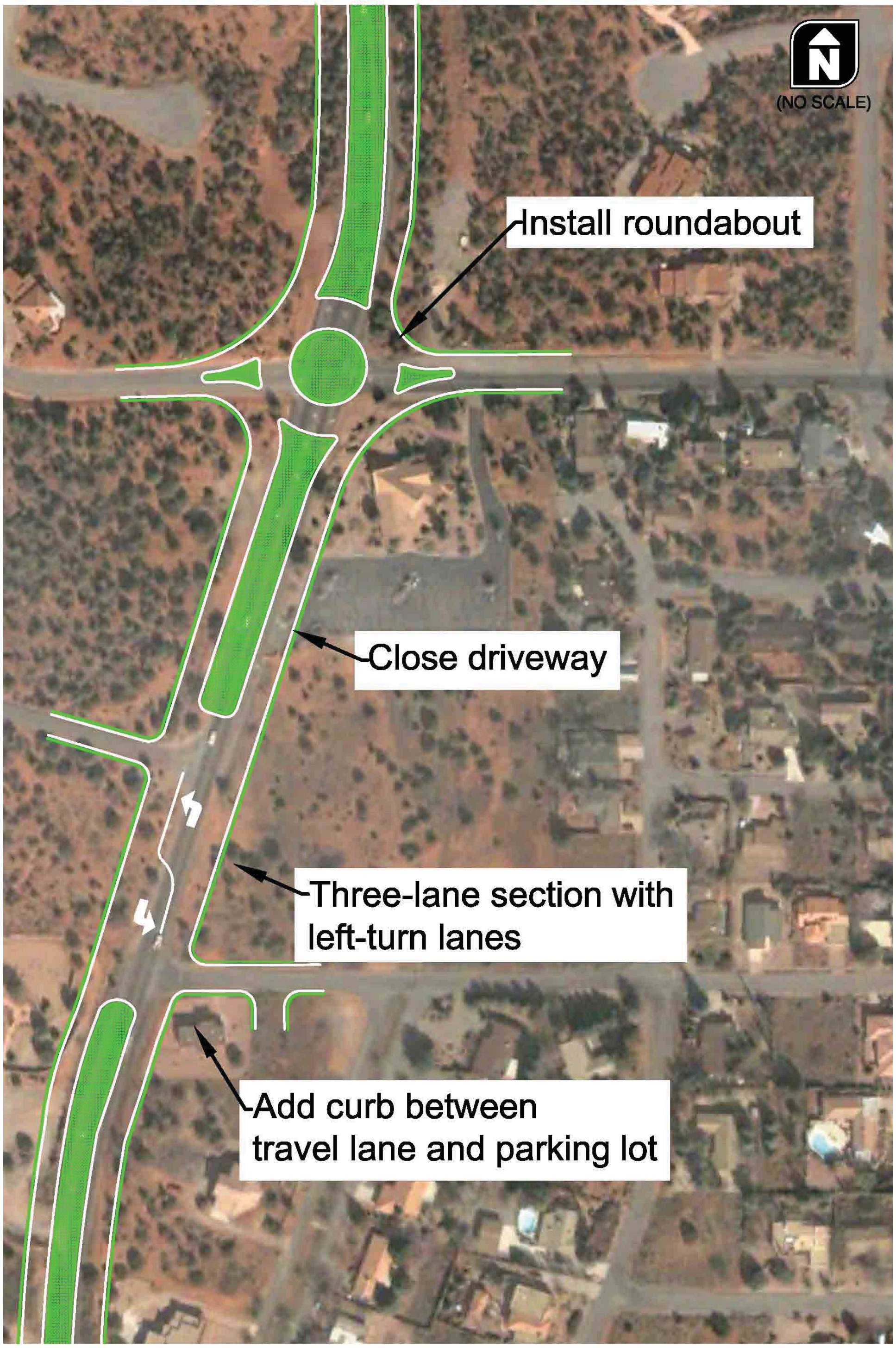


Exhibit 3-4: Representative Segment of Alternative 1

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-5: Alternative 2 Overview

Exhibit 3-6: Representative Segment of Alternative 2

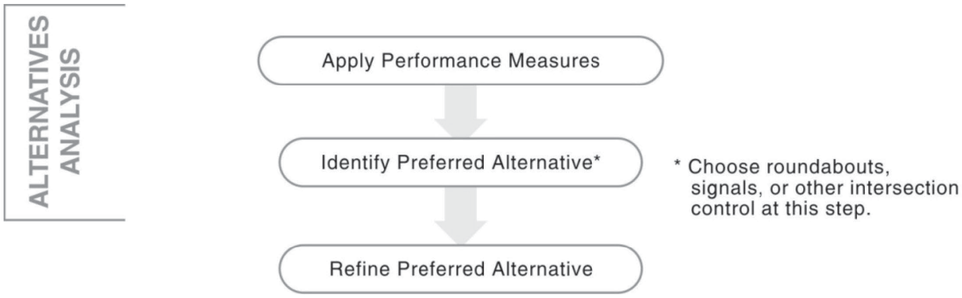


Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-7: Alternative 3 Overview

Evaluating the Performance of Corridors with Roundabouts

3.3. ALTERNATIVES ANALYSIS



Steps in the Alternatives Analysis phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

3.3.1. EVALUATE THE ALTERNATIVES

Exhibit 3-8 summarizes an analysis of the three alternatives proposed for US 7 using the key performance measures identified in Section 3.1.4.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-8: Alternatives Analysis

Perfor-mance Measure	Alternative 1 – Signals and TWLTL	Alternative 2 – Roundabouts and Median	Alternative 3 – Two-Lane Bypass	Comments
LOS	During the a.m. and p.m. peak hours, the three new signalized intersections will operate at LOS A or B.	During the a.m. and p.m. peak hours, the five roundabouts will operate at LOS A or B.	During the a.m. and p.m. peak hours, the signals or roundabouts at the endpoints of the bypass will operate at LOS B or C. Intersections on existing US 7 will remain TWSC and operate acceptably.	For each alternative, intersection operations are similar and meet the DOT's standard of LOS D or better.
Intersec-tion capacity	During the a.m. and p.m. peak hours, the three new signalized intersections will operate at a v/c of 0.59 or better.	During the a.m. and p.m. peak hours, the five roundabouts will operate at a v/c of 0.71 or better.	During the a.m. and p.m. peak hours, signals at the end of the bypass would operate at a v/c of 0.67 or better. Roundabouts would operate at 0.80 or better.	For each alternative, intersections meet the DOT's standard of 0.95 or lower for signals and 0.85 or lower for roundabouts.
Arterial capacity	The state DOT requires new highways be 4-lane if the ADT is 15,000 or greater. The forecast ADT of US 7 is 14,000 and a TWLTL will be added. The roadway links will be below capacity. Intersections will operate below capacity (see above) and not constrain the roadway.	According to Exhibit 3-12 of *NCHRP Report 672*, single-lane roundabouts are likely to operate acceptably with an ADT under 15,000.	During a.m. and p.m. peak hours, the v/c of the bypass (per the HCM two-lane highway procedure) will be 0.41 or better.	All alternatives provide adequate roadway capacity.
Availability of gaps for pedestrian crossings	Signals create additional mid-block gaps on US 7 by stopping major street traffic at nearby intersections and forming platoons of vehicles. Alternative 1 also includes several pedestrian refuge islands that create opportunities for two-stage crossings. Three signalized crossings are provided.	Fewer platoons are likely to form with roundabouts than with signals. Five roundabouts with two-stage crossings are provided.	The reduced traffic volume on US 7 creates additional gaps. No signals or roundabouts are available to assist pedestrians with a crossing.	The differences between the alternatives are unclear. This performance measure does little to inform the selection of corridor alternatives.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-8: Alternatives Analysis Con't

Perfor-mance Measure	Alternative 1 – Signals and TWLTL	Alternative 2 – Roundabouts and Median	Alternative 3 – Two-Lane Bypass	Comments
Conflict points	The number of intersection conflict points remains approximately the same. A four-leg intersection with single-lane approaches has 32 auto-auto conflict points and 24 auto-pedestrian conflict points. Some mid-block conflict points are eliminated by driveway closures or consolidations.	The number of intersection conflict points is reduced. A four-leg, single-lane roundabout has 8 auto-auto conflict points and 8 auto-pedestrian conflict points. The median reduces mid-block conflict points to a greater degree than Alternative 1.	Conflict points on existing US 7 will remain, although traffic volume will be reduced by more than half. The new roadway will have three to five driveways, plus the intersections at the endpoints.	Alternative 2 eliminates the most intersection and mid-block conflict points.
Auto/Bicycle speed differential *(see Section 3.3.1.1)*	Based on a study of a similar roadway, the DOT estimates bicyclists on US 7 currently travel 12 – 14 mph. The 85th percentile auto speed is currently 46 mph. Signals will slow some proportion of auto and bicycle traffic because they will sometimes be red. The effect of the TWLTL on auto speed is unclear.	Vehicles will slow to 25 miles per hour or less when passing through a roundabout. Compared to existing conditions, the auto/bicycle speed differential will be reduced within the roundabout influence area, which will extend 300-400 feet upstream and 800-900 feet downstream of each roundabout (see calculations in Section 3.3.1.1). Roundabouts are located substantially further apart than this, so most of the corridor will not have a reduced auto/bicycle speed differential.	Bicyclists will be encouraged to use existing US 7 rather than the bypass. With few physical changes to the roadway, auto speeds and the auto/bicycle speed differential can be expected to remain similar.	Alternative 2 reduces the auto/bicycle speed differential on some portions of the corridor compared to other alternatives.
Pedestrian level of service *(see Section 3.3.1.1)*	Pedestrian LOS improves by adding a sidewalk, signals, and a buffer (bike lane) between autos and pedestrians.	Pedestrian LOS improves by adding a sidewalk and a buffer (bike lane) between autos and pedestrians, reduced auto speeds, and decreased intersection width.	Pedestrian LOS improves due to the addition of a sidewalk and reduction in auto volume.	Alternatives 1 and 2 improve pedestrian LOS to a greater degree than Alternative 3.
Bicycle level of service	Bicycle LOS improves by adding a bike lane and reduction in the number of access points.	Bicycle LOS improves by adding a bike lane, reduction in the number of access points, and reduced auto speeds.	Bicycle LOS improves due to reduced auto volumes.	Alternatives 1 and 2 improve bicycle LOS to a greater degree than Alternative 3.

Exhibit 3-8: Alternatives Analysis Con't

Performance Measure	Alternative 1 – Signals and TWLTL	Alternative 2 – Roundabouts and Median	Alternative 3 – Two-Lane Bypass	Comments
Impacts to public facilities	The DOT will purchase a 7-foot-wide strip of land from a local school to accommodate a widened US 7. This is currently part of a lawn and impacts are considered minimal.	The DOT will purchase a 7-foot-wide strip and a triangular piece of land from a local school to accommodate a widened US 7 and a roundabout. This is currently part of a lawn, and impacts are considered minimal.	The new roadway will primarily be located on public land. Several hiking trails will need to be relocated, and there will be a decrease in the overall amount of recreational land.	Alternative 3 has the greatest impacts to public facilities.
Estimated right-of-way cost	$400,000.	$500,000.	$1.8 million.	Alternative 3 is more than three times the cost of the others. Alternatives 1 and 2 are of similar magnitude.
Estimated construction cost	$2.2 million.	$3.1 million.	$16 million.	Alternative 3 is more than five times the cost of the others. Alternative 1 is the lowest.
Livability	Alternative 1 improves the roadway with curbs, sidewalks, new pavement, and other enhancements. However, the TWLTL may create a suburban feel for the roadway and surrounding area.	Alternative 2 also improves the roadway with curbs, sidewalks, and new pavement. Additionally, the landscaping in the median and roundabout central islands complements the rural and natural character. Roundabouts slow vehicles, and create a gateway into Elk Grove.	Alternative 3 has the fewest improvements within Elk Grove. The traffic volume on existing US 7 is reduced, but the context of the new roadway may be inconsistent with the surrounding area.	Alternatives 1 and 2 improve existing US 7 and enhance livability to a greater degree than Alternative 3. Alternatives 1 and 3 create facilities that may be inconsistent with the character of Elk Grove.
Walkability	Adding sidewalks and creating defined driveways improves walkability and bicycling conditions. The widening of the road from two lanes to three may have a negative impact on walkability.	Like Alternative 1, adding sidewalks and creating defined driveways improves walkability and bicycling conditions. A median allows for two-stage crossings and the roadway remains two lanes. Vehicle speeds are reduced in the vicinity of the roundabouts.	Adding sidewalks and reducing traffic volume improve walkability and bicycling conditions.	Alternatives 1 and 2 feature more physical improvements than Alternative 3. Alternative 2 has one less lane on the roadway than Alternative 1, and changes control at five intersections (roundabouts) versus three intersections (signals).

Evaluating the Performance of Corridors with Roundabouts

Exhibit 3-8 Alternatives Analysis Con't

Perfor-mance Measure	Alternative 1 – Signals and TWLTL	Alternative 2 – Roundabouts and Median	Alternative 3 – Two-Lane Bypass	Comments
Aesthetics	Streetscape improvements such as new sidewalks and curbs will improve the appearance of the corridor.	Alternative 2 includes many of the same streetscape improvements as Alternative 1. Plus, the median and the roundabout central islands create opportunities for enhancing landscaping and the rural and natural context of the corridor.	Alternative 3 has fewer aesthetic improvements to existing US 7 than other alternatives. The new roadway may detract from the vistas of the Red River canyon outside of Elk Grove.	Alternative 2 offers the greatest potential for aesthetic improvements.
Communi-ty acceptance	Community members generally agree Alternative 1 improves the corridor to a marginal degree.	Many in the community support the context-sensitive nature of Alternative 2. The potential for landscaping and concept of roundabouts as gateways are supported by citizens and officials in Elk Grove. However, there is some concern roundabouts will be confusing to drivers and increase crashes.	There is little support for Alternative 3 within Elk Grove. It has the fewest improvements to the existing roadway and impacts undeveloped land outside of the town. Businesses in Elk Grove that serve through travelers are concerned about diverting traffic onto the new roadway. Alternative 3 is supported by the owners of several industrial businesses along SR 272 for mobility reasons.	The community is most supportive of Alternative 2 and least supportive of Alternative 3.
Access manage-ment	Alternative 1 eliminates parking lot access that continuously fronts the roadway and replaces it with defined and consolidated driveways. Some existing driveways near new signalized intersections are closed, such as those serving properties with other access points.	Alternative 2 eliminates more driveways than Alternative 1, and the median restricts many driveways with right-in right-out access. Roundabouts enable U-turns and indirect left turns. Like Alternative 1, Alternative 2 eliminates parking lot access that continuously fronts the roadway.	Alternative 3 does not change access points on existing US 7.	Alternative 2 offers the greatest access-management benefit.

Evaluating the Performance of Corridors with Roundabouts

3.3.1.1. *Roundabout Influence Area Length Computation Example*

Roundabout influence area was calculated for one of the roundabouts on US 7 to get a sense of the extent of areas in which speeds will be reduced due to the presence of roundabouts. Data used in these calculations are listed below:

- Upstream sub-segment length = 2,145 feet;
- Downstream sub-segment length = 2,215 feet;
- Central island diameter = 50 feet;
- Circulating speed = 20 mph; and
- The influence areas do not overlap (see Step B of the modeling framework in Chapter 3 of the main report).

First, the free-flow speed was determined using the upstream and downstream sub-segment models:

$$S_{f,US} = 15.1 + 0.0037 \times (2{,}145\ feet) + 0.43 \times (45\ mph) + 0.05 \times (50\ feet)$$

= 44.9 mph

$$S_{f,DS} = 14.6 + 0.0039 \times (2{,}215\ feet) + 0.48 \times (45\ mph) + 0.02 \times (50\ feet)$$

= 45.8 mph

Using these values, the corresponding roundabout influence areas can be computed as follows:

$$RIA_{US} = 165.9 + 13.8 \times (44.9\ mph) - 21.1 \times (20\ mph)$$

= 363 feet

$$RIA_{DS} = -149.8 + 31.4 \times (45.8\ mph) - 22.5 \times (20\ mph)$$

= 838 feet

These values indicate the influence areas do not overlap, so the assumption they do not was correct.

Intersections on US 7 are spaced at a distance much greater than 840 feet. Most of the corridor will not lie within a roundabout influence area, meaning roundabouts will not reduce speeds at most locations along the corridor.

3.3.2. IDENTIFY PREFERRED ALTERNATIVE

Alternative 3 was eliminated based upon the high construction cost, impact to public land, and negative feedback from the community.

The town of Elk Grove advocated for Alternative 2 because it was favored by attendees of public meetings, and offered place-making and aesthetic improvement opportunities in context with the surrounding land. The town believes Alternative 2 will preserve the integrity of US 7 serving as the Main Street of their community, rather than just a through highway. The town was encouraged by national data showing roundabouts reduce the number and severity of crashes compared to conventional intersection forms. The town offered to maintain roundabout and median landscaping. The town and the DOT executed a memorandum of understanding solidifying the agency partnership.

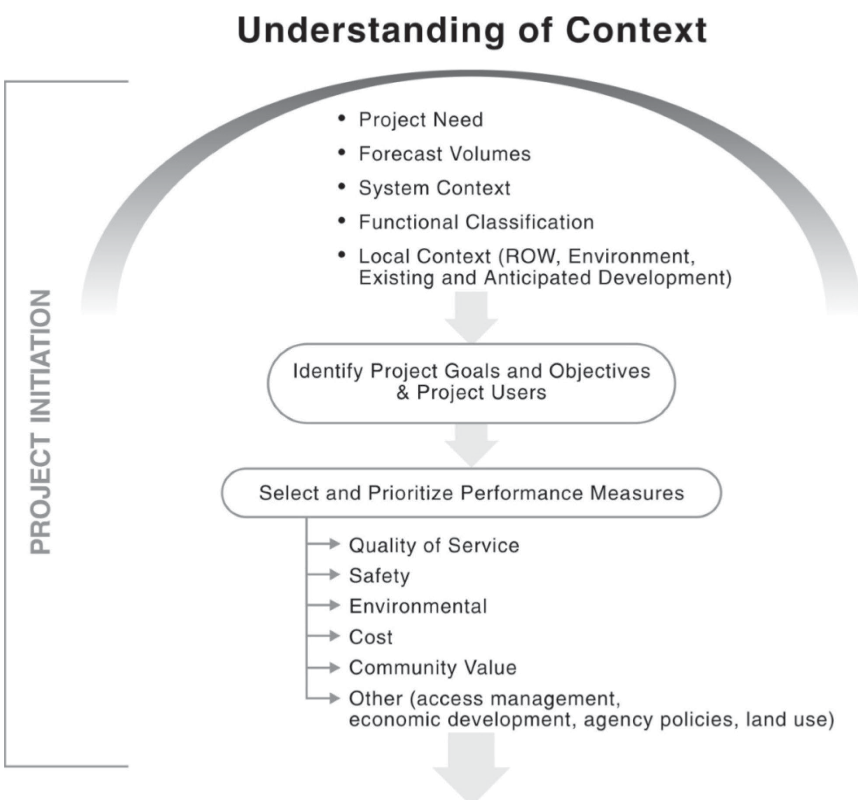
Evaluating the Performance of Corridors with Roundabouts

Maintenance of these elements was a concern of the DOT. The DOT selected Alternative 2 based on access-management benefits and support from the community.

EXAMPLE APPLICATION 4. STEVENS STREET

This fictional example application presents a suburban corridor being rebuilt for maintenance reasons. The project presents an opportunity to remake the image of the corridor and implement safety improvements.

4.1. PROJECT INITIATION



Steps in the Project Initiation phase of the Project-Planning Process (refer to Corridor Comparison Document, Chapter 3)

Exhibit 4-1: Key Data

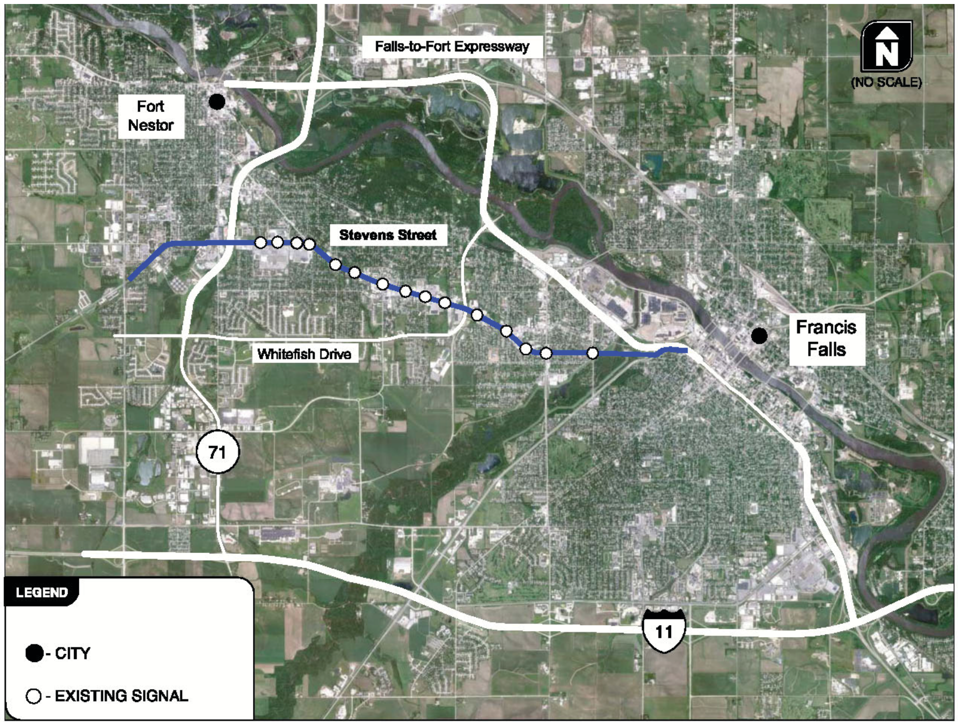
Cross-Section	6 lanes with raised median
Travel Lanes	3 each direction
Forecast ADT	30,000 veh/day
Peak-Hour Pedestrian Volume Crossing Corridor	0 to 20 p/h per intersection
Peak-Hour Pedestrian Volume Along Corridor	0 to 10 p/h
Sidewalks	Varies by segment, existing sidewalks generally in poor condition
Local Bus Service	60 minute headways
Land Use	Suburban

4.1.1. UNDERSTANDING OF CONTEXT

Fifteen years ago, the Falls-to-Fort Expressway opened. This facility improved mobility between Francis Falls and Fort Nestor and reduced traffic volume on

Stevens Street, which previously served as the primary route between the two cities. The segment of Stevens Street between the Falls-to-Fort Expressway and SR 71 is nearly five miles long and has 15 signalized intersections. It was originally a two-lane roadway, widened to four lanes with a median in the 1960s. It was widened to six lanes in the 1980s. The posted speed limit is 40 miles per hour, and 85th-percentile speeds range from 45 to 50 mph along the corridor. The Fort Nestor/Francis Falls area, including Stevens Street, is shown in Exhibit 4-2. A representative intersection on Stevens Street is shown in Exhibit 4-3 in the Concept Development section of this example application.

Exhibit 4-2: Proposed Corridor Alignment



4.1.2. USERS AND TRAFFIC VOLUME

The design-year forecast ADT on Stevens Street is 30,000. This represents an increase from the current ADT of 26,000, and remains below the ADT of 44,000 recorded the year before the Falls-to-Fort Expressway opened. Stevens Street is an auto-focused facility. Sidewalks are present along the corridor, but they have been poorly maintained and most blocks have 10 or fewer pedestrians walking along the corridor during the peak hour. Peak-hour pedestrian-crossing volumes at signalized intersections range from 0 to 20. There is less bicycle activity than pedestrian activity. Most of the roadway has no shoulder due, in part, to the widening in the 1980s. A bus route with 60-minute headways operates on the corridor.

For brevity, the fictional example applications only include volumes from one year. Generally, a planning study for a corridor would forecast future, design-year volumes and practitioners would use these volumes for planning and analysis.

Evaluating the Performance of Corridors with Roundabouts

4.1.3. PROJECT CATALYST AND GOALS

Stevens Street is in need of maintenance. Most of the roadway has been resurfaced several times and Fort Nestor wants to remove all layers of pavement and rebuild the roadway from the subgrade up to minimize future pavement maintenance needs. The city and community groups are interested in reviving the retail market along Stevens Street. Most of the commercial buildings along the corridor were constructed prior to 1985 and some major retailers have moved to newer properties along Whitefish Drive or SR 71. Commercial properties on Stevens Street have a vacancy rate above the city average. This project presents the city with an opportunity to remake the image and design of Stevens Street.

4.1.4. SELECT AND PRIORITIZE PERFORMANCE MEASURES

The sections below list the six groups of performance measures discussed in the Corridor Comparison Document (CCD) and identify specific performance measures of importance on the Stevens Street corridor. The performance measures identified below are not necessarily all that could be considered for the Stevens Street project. Many performance measures could be used to evaluate a corridor. Some are of critical importance for nearly all corridors (Tier I), and others are only applicable to some corridors (Tiers II and III). For the purpose of illustrating the use of the CCD, this example presents performance measures of particular interest on the Stevens Street corridor that help to distinguish the alternatives from one another. This includes Tier I measures like intersection level of service and cost, and Tier II and III measures like economic development. Performance measures of strong interest to the community are generally prioritized over those of lesser interest to the community.

4.1.4.1. *Quality of Service Performance Measures*

Stevens Street may be capable of meeting operating standards with a reduced number of lanes. Changes to lane configurations, intersections, or other roadway elements also have the potential to positively change pedestrian quality of service.

Key Performance Measures: Peak-hour intersection level of service, intersection capacity, Urban Street LOS for pedestrians.

4.1.4.2. *Safety Performance Measures*

Rear-end crashes are the most frequently occurring crash type at signalized intersections on Stevens Street. City engineers attribute some of these crashes to high speeds on Stevens Street, and believe reducing speeds will improve intersection safety. Corridor residents also expressed interest in speed reduction. The city will explore using the HSM to quantitatively assess how potential geometric changes in various alternatives may change safety performance.

Key Performance Measure: Predicted changes in crash frequency at intersections.

4.1.4.3. *Environmental Performance Measures*

This project will modify an existing roadway in a developed suburban area. Little or no undeveloped land will be disturbed. There are several parks and

Evaluating the Performance of Corridors with Roundabouts

schools in the project area away from Stevens Street. It is unlikely these facilities will be impacted.

Key Performance Measures: None.

4.1.4.4. *Cost Performance Measures*

Rebuilding Stevens Street will be the largest project undertaken by the city of Fort Nestor in recent years. Some funds are programmed, but they may not be sufficient for all alternatives. Due to the condition of the pavement on Stevens Street, the city does not intend to delay the project. Alternatives costing more than programmed funding are likely infeasible.

Key Performance Measures: Pre-construction costs such as planning, preliminary engineering, and final design; right-of-way acquisition cost; capital construction cost.

4.1.4.5. *Community Value Performance Measures*

Stevens Street is the main arterial link between Fort Nestor and Francis Falls. Stakeholders want to improve the image of the corridor and transform it into a signature roadway that defines the community. This could also establish a sense of place and appeal to businesses and consumers.

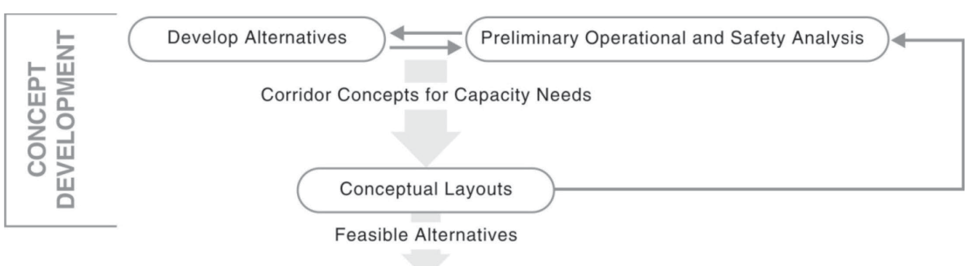
Key Performance Measures: Aesthetics, place-making, overall public opinion.

4.1.4.6. *Other Performance Measures*

The city hopes this project revitalizes the Stevens Street corridor. Some larger, regional-drawing stores moved to Whitefish Drive or SR 71 in recent years. Changes to Stevens Street may make it a more viable location for smaller, neighborhood-serving businesses and reduce the commercial property vacancy rate.

Key Performance Measures: Land-use considerations, economic-development/tax-base considerations.

4.2. CONCEPT DEVELOPMENT



Steps in the Concept Development phase of the Project-Planning Process (refer to CCD, Chapter 3)

4.2.1. DEVELOP ALTERNATIVES AND PRELIMINARY OPERATIONS AND SAFETY ANALYSIS

With a forecast ADT of 32,000, two lanes in each direction should be sufficient for segment capacity needs. This creates an opportunity to reduce the number of travel lanes on Stevens Street.

Evaluating the Performance of Corridors with Roundabouts

Fort Nestor considered three alternatives for Stevens Street:

1. Alternative #1 rebuilds the pavement of the existing six-lane roadway and replaces some signal equipment at the end of its life cycle. This is effectively a no-build alternative.
2. Alternative #2 reduces the roadway to four lanes. The existing outside lanes are replaced with a bicycle lane, curb and gutter, and a widened sidewalk. Intersections remain signalized.
3. Alternative #3 reduces the roadway to four lanes and replaces signals with two-lane roundabouts. The existing outside lanes are replaced with a bicycle lane, curb and gutter, and a widened sidewalk.

Based on guidance in *NCHRP Report 672* and the city's experiences with roundabouts to date, Alternative 3 uses two-lane roundabouts with inscribed circle diameters (ICDs) in the range of 180 to 200 feet. This will require right-of-way acquisition at most intersections, including acquiring and demolishing some structures. Exhibit 4-3 shows an intersection where existing right-of-way does not accommodate a roundabout.

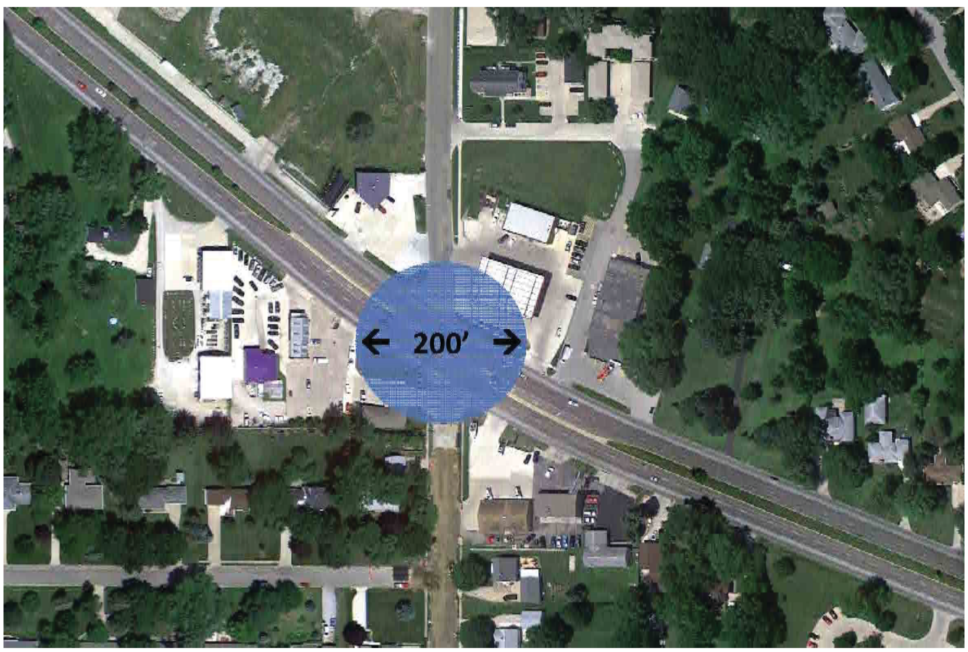


Exhibit 4-3: Approximate Footprint of Roundabout at Existing Intersection

4.2.2. CONCEPTUAL LAYOUTS

Fort Nestor city staff developed typical sections of each alternative to present to the public and to decision makers. These typical sections are shown in Exhibits 4-4 and 4-5. The typical sections illustrate mid-block (not intersection) conditions, so Alternatives 2 and 3 are depicted to be the same.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-4: Alternative 1 Typical Section

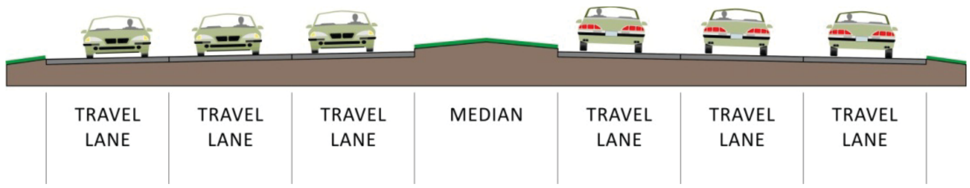
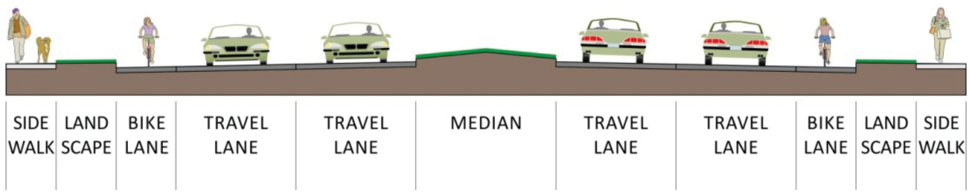
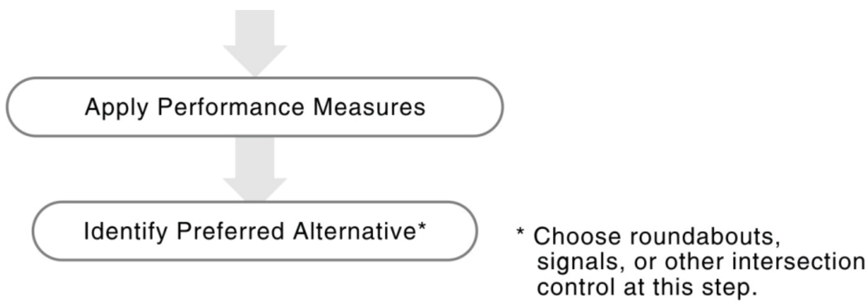


Exhibit 4-5: Alternatives 2 and 3 Typical Section



The city also developed plan-view layouts of the alternatives to assess right-of-way needs and perform initial cost estimates. These layouts are omitted from the example application for brevity.

4.3. ALTERNATIVES ANALYSIS



Steps in the Alternatives Analysis phase of the Project-Planning Process (refer to CCD, Chapter 3)

4.3.1. EVALUATE THE ALTERNATIVES

Exhibit 4-6 summarizes an analysis of the three alternatives proposed for Stevens Street using the key performance measures identified in Section 4.1.4.

Exhibit 4-6: Alternatives Analysis

Performance Measure	Alternative 1 – Rebuild 6-Lane Existing	Alternative 2 – 4-Lane with Signals	Alternative 3 – 4-Lane with Roundabouts	Comments
Peak-hour intersection LOS	All signalized intersections operate at LOS C or better. Most operate at LOS B.	All signalized intersections operate at LOS D or better. Most operate at LOS C or better.	The critical movement at all roundabouts is LOS C or better.	Intersections operate at LOS D or better under all alternatives.

Exhibit 4-6: Alternatives Analysis Con't

Perfor-mance Measure	Alternative 1 – Rebuild 6-Lane Existing	Alternative 2 – 4-Lane with Signals	Alternative 3 – 4-Lane with Roundabouts	Comments
Intersec-tion capacity	All signalized intersections have a v/c of 0.73 or less.	All signalized intersections have a v/c of 0.86 or less.	All roundabouts have a critical-movement v/c of 0.82 or less.	All intersections operate at acceptable levels.
Urban Street LOS for pedestri-ans	Many segments are LOS F because there is no sidewalk. Other segments vary from LOS C to E.	All segments are LOS E or better. Most are LOS C or D.	Results are similar to Alternative 2. The Pedestrian Urban Street LOS procedure in the HCM 2010 does not directly accommodate segments with roundabouts.	Alternatives 2 and 3 provide a higher pedestrian LOS than Alternative 1.
Predicted changes in crash frequency at intersec-tions *(See Section 4.3.1.1)*	Few changes in crash frequency or severity are expected.	Safety changes are unclear; see Section 4.3.1.1 for more detail.	Per Exhibit 14-3 of the HSM, the CMF for replacing a suburban signalized intersection with a two-lane roundabout is 0.33 with a standard error of 0.05. Therefore, 0.23 to 0.43 times as many intersection crashes are expected with this alternative compared to existing conditions. Roundabouts also reduce the severity of crashes by reducing vehicle speeds and angle conflict points. A CMF for injury crashes in this situation (suburban signalized intersection replaced with two-lane roundabout) is not provided in the HSM.	Alternative 3 is expected to reduce the frequency and severity of intersection crashes.
Estimated pre-construc-tion cost	$400,000.	$1.1 million.	$ 3.9 million.	Alternative 3 is the most expensive, followed by Alternative 2.
Estimated right-of-way acquisition cost	$0.	$400,000. For sidewalks in areas where current ROW is insufficient.	$15 million. ROW is purchased at most intersections and several structures will be acquired.	Alternative 3 is more than 30 times the cost of Alternative 2. Alternative 1 has no cost.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-6: Alternatives Analysis Con't

Perfor-mance Measure	Alternative 1 – Rebuild 6-Lane Existing	Alternative 2 – 4-Lane with Signals	Alternative 3 – 4-Lane with Roundabouts	Comments
Estimated capital construc-tion cost	$5 million.	$19 million.	$29 million.	Alternative 3 is $10 million more than Alternative 2. Alternative 2 is nearly triple the cost of Alternative 1.
Intersec-tion operations and mainten-ance costs	$120,000 annually, including signal maintenance, signal power supply, and signal retiming every several years.	Similar to Alternative 1.	$30,000 annually, primarily landscaping.	Alternative 3 has the lowest costs.
Aesthetics	The corridor will look the same as today, but with new pavement.	Addition of curb and gutter, sidewalks, and bicycle lanes creates a more urban appearance and shows the city's investment in the corridor.	In addition to improvements noted under Alternative 2, roundabouts make intersections more visually appealing and provide opportunities for landscaping, public art, and other decorative treatments.	Alternatives 2 and 3 improve the aesthetics of the corridor. Alternative 3 creates more opportunities than Alternative 2.
Place-making	The corridor will essentially be the same place it is today.	The corridor will look newer than other streets in this part of the city and have amenities such as sidewalks and bicycle lanes that are uncommon on Fort Nestor's arterials.	The corridor will be a unique and recognizable element of the city's transportation network. Roundabouts serve as gateways and can help to brand the corridor. Place-making benefits noted under Alternative 2 exist as well.	Alternative 3 creates the greatest sense of place, followed by Alternative 2.
Land-use considera-tions	Land use will likely remain similar to current conditions.	The changes in the corridor may make it a more viable location for neighborhood-oriented businesses that would benefit from improved multi-modal conditions. The roadway will still have sufficient capacity for autos and remain a viable location for auto-oriented land uses.	The uniqueness of the corridor may attract upscale and specialty businesses. Benefits noted under Alternative 2 exist as well.	Alternatives 2 and 3 have the greatest potential to change land use in the corridor.

Evaluating the Performance of Corridors with Roundabouts

Exhibit 4-6: Alternatives Analysis Con't

Perfor-mance Measure	Alternative 1 – Rebuild 6-Lane Existing	Alternative 2 – 4-Lane with Signals	Alternative 3 – 4-Lane with Roundabouts	Comments
Overall public opinion	There is little support for this alternative.	Approximately half of attendees to public meetings favor this alternative.	Approximately half of attendees to public meetings favor this alternative. Some residents are concerned that driving the corridor will be slow and challenging due to the numerous roundabouts.	Public opinion is generally divided between Alternatives 2 and 3.
Economic develop-ment/tax-base considera-tions	This alternative does not create an impetus for investment in the corridor.	The visible signs of investment by the city and increased potential for neighborhood-oriented businesses may reduce the vacancy rate and encourage new construction.	The potential for economic development is similar to Alternative 2. Some properties will be reduced in size and value due to the acquisition of land for the roundabouts. The greater magnitude of changes in the corridor compared to Alternative 2 could spur additional investment.	Alternatives 2 and 3 may encourage economic development and improve the corridor.

4.3.1.1. *Alternative 2 Predicted Change in Crash Frequency and Severity*

Part C of the HSM contains a predictive method for four-lane divided arterials in Chapter 12 – Predictive Method for Urban and Suburban Arterials. However, there is no predictive method for six-lane divided arterials in the HSM, and no calibration factors have been developed for the Fort Nestor/Francis Falls area. Comparing observed crash history of the existing roadway to an uncalibrated predictive model may not present a representative comparison. Also, Part D of the HSM does not have a crash modification factor for converting a six-lane arterial to a four-lane arterial. The HSM does not provide a means of comparing the safety performance of Alternative 2 to existing conditions. The decrease in speed discussed in the previous section may reduce the frequency and/or the severity of crashes.

4.3.2. IDENTIFY PREFERRED ALTERNATIVE

Alternative 3 had the greatest potential to change the image and land use of the corridor. It would have created a unique corridor unlike any other in the city. However, public opinion was divided. Some residents favored roundabouts for the place-making and traffic calming reasons, and others were concerned about repeatedly slowing when driving the corridor and felt a coordinated signal system would be operationally superior. City staff generally favored Alternative 3 for place-making and economic-development reasons as well as safety benefits.

Evaluating the Performance of Corridors with Roundabouts

Unfortunately, Fort Nestor had only $25 million available for the Stevens Street project. This was enough to fund Alternative 2, but less than half of the estimated total cost (ROW, pre-construction, construction) of Alternative 3. Sufficient funds for Alternative 3 would not have been available for at least five years.

4.3.3. ITERATION

Alternative 3 was not financially feasible. However, city staff remained interested in constructing a roundabout at one intersection on the corridor: Stevens Street/Robinson Street. This is the fourth intersection northeast of SR 71 in Exhibit 4-2, where Stevens Street curves. The skew of this intersection has long created safety issues, and Alternatives 1 and 2 will not fundamentally change this. Residents and business owners on Stevens Street immediately east of this intersection expressed a strong desire for speed control, particularly to slow drivers leaving the mall area between Robinson Street and SR 71.

4.3.4. IDENTIFY MODIFIED PREFERRED ALTERNATIVE

Fort Nestor selected a modified version of Alternative 2, with a roundabout at Stevens Street/Robinson Street and signals at other existing signalized intersections.

Abbreviations and acronyms used without definitions in TRB publications:

A4A	Airlines for America
AAAE	American Association of Airport Executives
AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
ACI–NA	Airports Council International–North America
ACRP	Airport Cooperative Research Program
ADA	Americans with Disabilities Act
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
DHS	Department of Homeland Security
DOE	Department of Energy
EPA	Environmental Protection Agency
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
HMCRP	Hazardous Materials Cooperative Research Program
IEEE	Institute of Electrical and Electronics Engineers
ISTEA	Intermodal Surface Transportation Efficiency Act of 1991
ITE	Institute of Transportation Engineers
MAP-21	Moving Ahead for Progress in the 21st Century Act (2012)
NASA	National Aeronautics and Space Administration
NASAO	National Association of State Aviation Officials
NCFRP	National Cooperative Freight Research Program
NCHRP	National Cooperative Highway Research Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
PHMSA	Pipeline and Hazardous Materials Safety Administration
RITA	Research and Innovative Technology Administration
SAE	Society of Automotive Engineers
SAFETEA-LU	Safe, Accountable, Flexible, Efficient Transportation Equity Act: A Legacy for Users (2005)
TCRP	Transit Cooperative Research Program
TEA-21	Transportation Equity Act for the 21st Century (1998)
TRB	Transportation Research Board
TSA	Transportation Security Administration
U.S.DOT	United States Department of Transportation