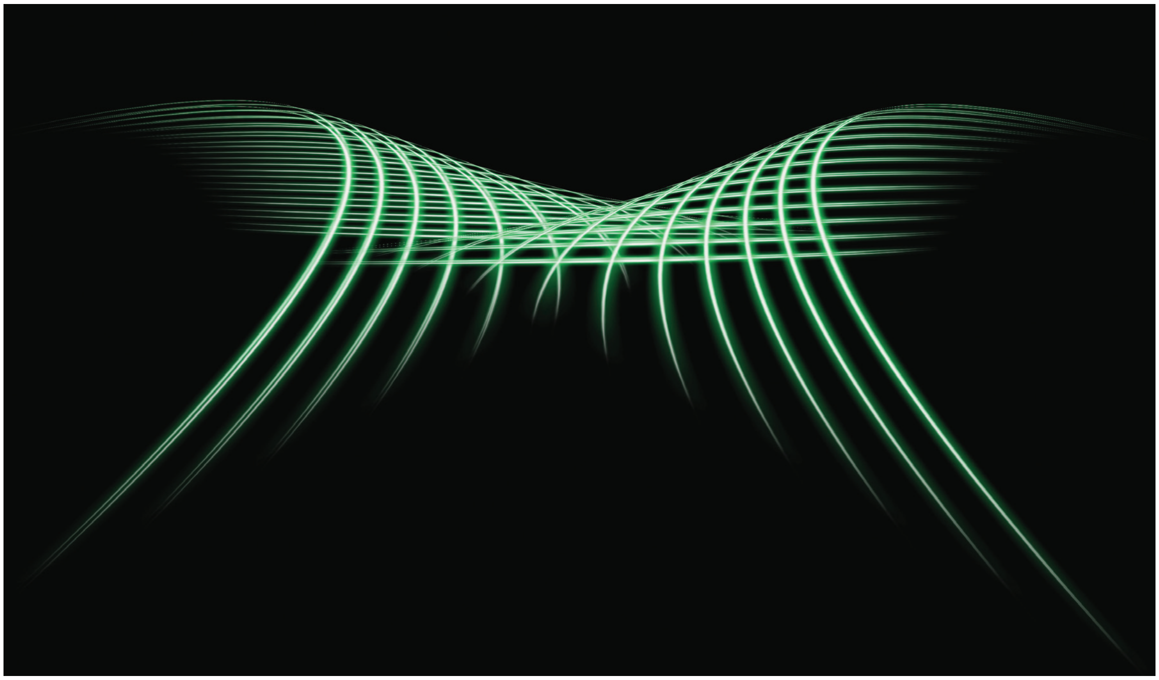
HCM2010

HIGHWAY CAPACITY MANUAL



CHAPTER 29

URBAN STREET FACILITIES: SUPPLEMENTAL

TRB TRANSPORTATION RESEARCH BOARD
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HCM2010

HIGHWAY CAPACITY MANUAL

The HCM 2010 significantly enhances how engineers and planners assess the traffic and environmental effects of highway projects by

- Providing an integrated multimodal approach to the analysis and evaluation of urban streets from the points of view of automobile drivers, transit passengers, bicyclists, and pedestrians;
- Addressing the proper application of microsimulation analysis and the evaluation of the results;
- Examining active traffic management in relation to demand and capacity; and
- Exploring specific tools and generalized service volume tables to assist planners in quickly sizing future facilities.

The HCM 2010 consists of four volumes:

Volume 1: Concepts;
Volume 2: Uninterrupted Flow;
Volume 3: Interrupted Flow; and
Volume 4: Applications Guide (electronic only).

The four-volume format provides information at several levels of detail, to help users more easily apply and understand the concepts, methodologies, and potential applications.

Volumes 1, 2, and 3 are issued as a boxed set. Volume 4 is electronic only, accessible to registered HCM users via the Internet, and includes four types of content: supplemental chapters on methodological details and emerging issues; interpretations, clarifications, and corrections; comprehensive case studies; and a technical reference library.

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CHAPTER 29
URBAN STREET FACILITIES: SUPPLEMENTAL

CONTENTS

LIST OF EXHIBITS

1. INTRODUCTION

Chapter 16, Urban Street Facilities, presented a methodology for combining the performance measures from each segment on an urban street in a manner that represents the operation of the facility as a whole. The stated limitations for each type of segment apply equally to the analysis of the facilities that they make up. In addition, the Chapter 16 procedures do not recognize interactions between segments that could occur, for example, when a queue crosses segment boundaries or when the operation within a segment or at a segment boundary disturbs the progressive movement of traffic along a route.

Several other chapters present supplemental examples covering the use of alternative tools to deal with individual segment limitations such as queue spillover, interaction between segments, and certain types of self-aggravating phenomena:

- Chapter 24, Concepts: Supplemental, demonstrates the use of individual vehicle trajectory analysis to examine cyclical queuing characteristics and to assess queue spillover into an upstream segment.
- Chapter 27, Freeway Weaving: Supplemental, presents a simulation example that demonstrates the detrimental effect of queue backup from an exit ramp signal on the operation of a freeway weaving section.
- Chapter 31, Signalized Intersections: Supplemental, presents simulation examples that demonstrate the effect of storage bay overflow, right-turn-on-red operation, short through lanes, and closely spaced intersections.
- Chapter 34, Interchange Ramp Terminals: Supplemental, presents a simulation example that demonstrates the effect of ramp metering signals on the operation of a diamond interchange. Another simulation example examines the effect of the diamond interchange on the operation of a nearby intersection under two-way stop control.

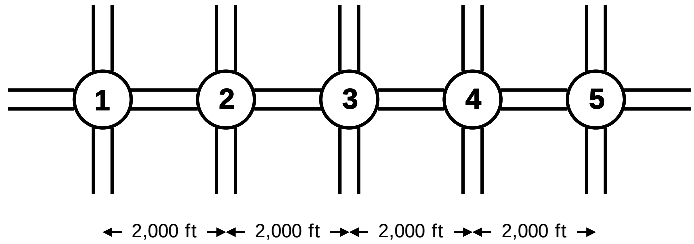
This chapter presents a few supplemental examples using alternative automobile traffic analysis tools to deal specifically with the limitations of the Chapter 16 procedures and to capitalize on the additional features of alternative tools. Both deterministic and stochastic tools are illustrated. The emphasis is exclusively on the automobile mode because alternative tools are applied more frequently to deal with automobile traffic.

The need to determine performance measures from an analysis of vehicle trajectories was emphasized in Chapter 7, Interpreting HCM and Alternative Tool Results, and Chapter 24, Concepts: Supplemental. Specific procedures for defining measures in terms of vehicle trajectories were proposed to guide the future development of alternative tools. Pending further development, most of the examples presented in this chapter have applied existing versions of alternative tools and therefore do not reflect the proposed trajectory-based measures.

2. BASIC EXAMPLE PROBLEM CONFIGURATION

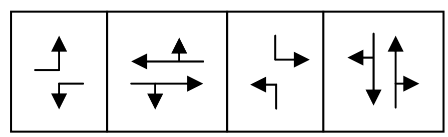
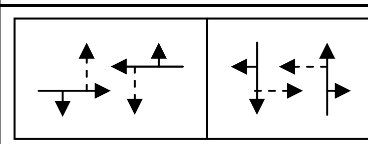
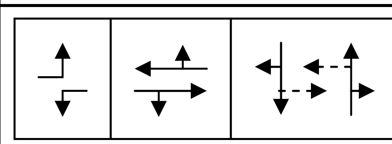
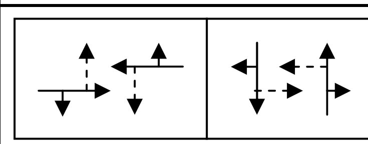
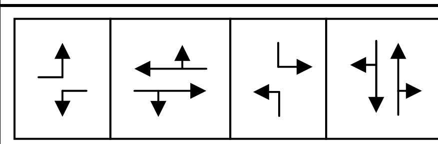
The base configuration for these examples is shown in Exhibit 29-1. Five signalized intersections are included with a spacing of 2,000 ft between the upstream stop lines of each intersection. Each intersection has the same layout, with two lanes for through and right-turn movements and one 150-ft-long left-turn bay.

Exhibit 29-1
Base Configuration for the Examples



The phasing and demand flow rates for each intersection are shown in Exhibit 29-2. Leading protected phases are provided for all protected left turns. Intersections 1 and 5 have protected phases for all left turns. Intersections 2 and 4 have only permitted left turns. Intersection 3 has protected left turns on the major street and permitted left turns on the minor street.

Exhibit 29-2
Demand Flow Rates and Phasing Plan for Each Intersection

Int. No.	Movement	Peak 15-min Adjusted Demand Left	Through	Right	Phasing Plan
1	Major st.	120	800	80	
	Minor st.	120	600	80	
2	Major st.	80	800	120	
	Minor st.	80	600	120	
3	Major st.	120	800	80	
	Minor st.	80	600	120	
4	Major st.	80	800	120	
	Minor st.	80	600	120	
5	Major st.	120	800	80	
	Minor st.	120	600	80	

To simplify the discussion, the examples will focus on design and analysis features that are beyond the stated limitations of the urban street analysis procedures contained in Chapters 16 through 21. For example, pretimed control will be assumed here because the ability to deal with traffic-actuated control is not a limitation of the Chapter 18 signalized intersection analysis methodology. For the same reason, the analysis of complex phasing schemes that fall within the scope of the Chapter 18 procedures (e.g., protected-permitted phasing) will be avoided. Parameters that influence the saturation flow rate (e.g., trucks, grade, lane width, parking) will not be considered here because they are accommodated in other chapters.

A symmetrical demand volume pattern will be used to facilitate interpretation of results. The demand volumes are assumed to be peak-hour adjusted. Fixed yellow-change and all-red clearance intervals of 4 s and 1 s, respectively, will be assigned to all phases. Through-traffic phases and protected left-turn phases will be assigned minimum green times of 10 s and 8 s, respectively.

3. SIGNAL TIMING PLAN DESIGN

The procedures presented throughout the *Highway Capacity Manual* (HCM) were developed to determine the performance of a roadway segment under specific conditions. In simple cases the procedures may be applied in reverse for design purposes (e.g., determining the number of required lanes). In more complex situations requiring optimization of design parameters, the procedures must be applied iteratively within an external software structure. Some alternative tools provide this type of optimization structure and therefore offer a valuable extension of the HCM's methodology. The extent of HCM compatibility in the analysis methodology varies among tools.

Two deterministic optimization tools will be applied in this section to illustrate how they can be used to produce the signal timing parameters required by the procedures of Chapters 17 and 18. This discussion is not intended as a comprehensive tutorial on signal timing plan design (STPD). A more detailed treatment of this subject is available (*1*), which serves as a comprehensive guide to traffic signal timing and includes a discussion of the use of deterministic optimization tools. It represents a synthesis of traffic signal timing concepts and their application and focuses on the use of detection, related timing parameters, and effects on users at the intersection.

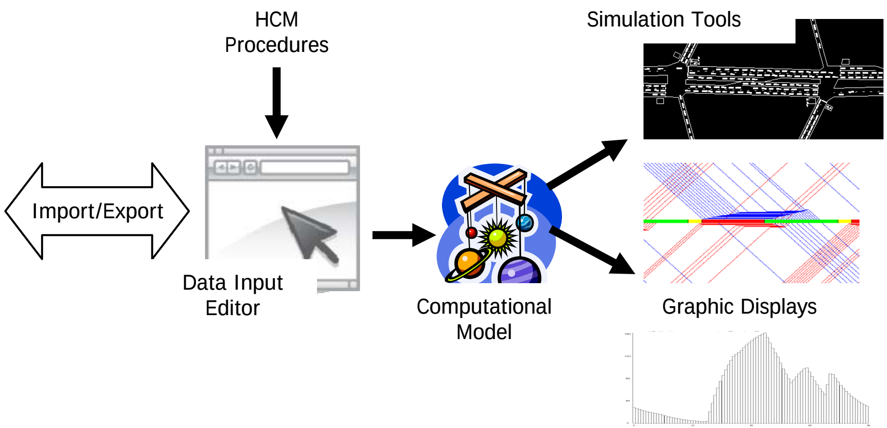
DETERMINISTIC STPD TOOLS

Several deterministic STPD tools are available commercially. Each represents a comprehensive package with its own computational and interface features designed to provide insight into operational details and to promote user productivity in the development of signal timing plans. A typical STPD tool configuration is illustrated in Exhibit 29-3. The following elements are included in the configuration:

- The computational model, which performs the design, optimization, and analysis functions. Two components are included in the computational model. The first component computes performance measures on the basis of specified input data and operating parameters. The second contains the optimization routines that seek a combination of operating parameters that will produce the best performance.
- The data input editor, which organizes and facilitates the entry of traffic data and operating parameters to be supplied to the computational model. The data input editor establishes the "look and feel" of each tool. The details vary considerably among tools. For example, some tools offer the ability to compute saturation flow rates internally by using procedures similar to those prescribed in Chapter 18, Signalized Intersections.
- Import/export features, which facilitate communication of data sets between other applications and devices. These features are intended to enhance the productivity of each tool.

- Direct links to other applications, such as microscopic simulation tools and fully HCM-compliant software.
- Graphic displays, which provide insight into time–space relationships, queuing, and platoon propagation.

Exhibit 29-3
Elements of a Typical Signal Timing Design Tool



The urban streets analysis procedures presented in the HCM deal with the operation of a single artery as a set of interconnected segments. Most of the commonly used STPD tools are configured to accommodate traffic control networks involving multiple intersecting routes. To simplify the discussion, the example presented here is limited to a single arterial route that will be analyzed as a system.

Two widely used STPD tools will be applied to this example to illustrate their features and to show how they can be used to supplement the urban street facilities analysis procedures prescribed in this manual. Both tools are commercially available software products. More information about these tools can be found elsewhere (*2*, *3*). The discussion in this section deals with the combination of features available from both tools without reference to a specific tool.

PERFORMANCE MEASURES

Both STPD tools deal with performance measures that are computed by the procedures prescribed in this manual in addition to performance measures that are beyond the scope of those procedures. The performance measures covered in Chapters 16 and 17 include delay, stops, average speed, and queue length. The discussion of those measures in this section will focus on their use in STPD and not on comparison of the values computed by different methods.

Several other measures beyond the scope of the HCM procedures are commonly associated with signal timing plan design and evaluation. The following measures are derived from analysis of travel characteristics, including stops, delay, and queuing:

- *Fuel consumption* (gal/h), the amount of fuel consumed because of vehicle miles traveled, stops, and delay, as computed by a model specific to each tool;

- *Operating cost* ($/h), the total cost of operation of all vehicles as computed by a model specific to each tool; and
- *Time jammed*, the percentage of time that the queue on a link has backed up beyond the link limit.

STPD tools also deal with a set of performance measures related to the quality of progression between intersections. These measures, all of which are outside of the HCM scope, have been defined in the literature or by developers of specific tools as follows:

- *Bandwidth* is defined by the number of seconds during which vehicles traveling at the design speed will be able to progress through a set of intersections. *Link bandwidth* is the width of the progression band (in seconds) passing between adjacent intersections that define the link. *Arterial bandwidth* is the width of the progression band that travels the entire length of the arterial route.
- *Progression efficiency* is the ratio of the arterial bandwidth to the cycle length. It thus represents the proportion of the cycle that contains the arterial progression band. Suggested upper limits for "poor," "fair," and "good" progression are 0.12, 0.24, and 0.36, respectively (*3*). Values above 0.36 are characterized as "great" progression.
- *Progression attainability* is the ratio of the arterial bandwidth to the shortest green time for arterial through traffic on the route. By definition, the arterial progression band cannot be greater than the shortest green time. Therefore, an attainability of 100% indicates that further improvement is only possible through the provision of additional green time. The need for fine-tuning is suggested for attainability values between 70% and 99%, with major changes needed for values below 70% (*3*).
- *Progression opportunities (PROS)* are a measure of arterial progression quality that recognizes progression bands that are continuous between two or more consecutive links but do not travel the full length of the arterial. The number of PROS observed by a driver at any point in time and space is defined by the number of intersections that lie ahead within the progression band. The concept is based on the premise that driver perception of progression quality increases with the number of consecutive links that can be traversed within the progression band. The measure is accumulated in a manner similar to the score in a game of bowling, where success in one frame is passed on to the next frame to increase the total score if the success continues. More detailed information on the computation of PROS is available elsewhere (*3*).
- *Interference* is expressed as the percentage of time that an arterial through vehicle entering a link on the green signal and traveling at the design speed will be stopped at the next signal. This measure is arguably an indication of poor perceived progression quality (*3*).
- *Dilemma zone vehicles* indicates the number of vehicles arriving on the yellow interval. As such, it offers a potential safety-related measure. The computational details are described elsewhere (*2*).

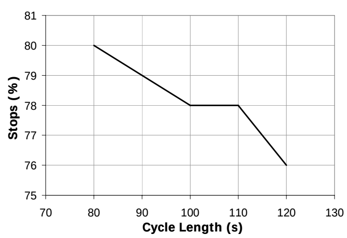
- *Coordinatability factor (CF)*: While it is not strictly a performance measure as defined in this manual, the CF is a measure of the desirability of coordinating two intersections on the basis of several factors including intersection spacing, speeds, and platoon formation. It is expressed as a relative value between 0 and 100. This measure is described in more detail elsewhere (*2*), where it is suggested that values above 80 indicate a definite need for coordination.

INITIAL TIMING PLAN DESIGN

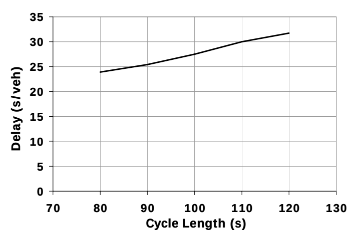
An initial timing plan design will first be performed by using one of the STPD tools. From the list of performance measures just discussed, fuel consumption will be chosen in this example as the performance measure for optimization. Other measures or combinations of measures could have been selected. No recommendation is implied in the selection of this particular measure. It serves this discussion because it supports an analysis of the trade-off between other measures such as stops and delay.

A cycle length within a specified range must be selected first. Minimum and maximum cycle lengths of 80 and 120 s, respectively, will be used. The cycle optimization results are presented in Exhibit 29-4, which shows the effect of the cycle length on delay, stops, and fuel consumption as computed by the STPD. While delay and stops move in opposite directions, their combined effect suggests that the minimum fuel consumption will be reached with an 80-s cycle. This is not surprising because it is generally recognized that the optimal cycle length for balanced progression is twice the link travel time at the design speed, which is 2 × 34 = 68 s for a 2,000-ft link at 40 mi/h. However, 68 s is below the minimum cycle length constraint. On the basis of these results, an 80-s cycle will be selected for optimization of the other timing plan parameters.

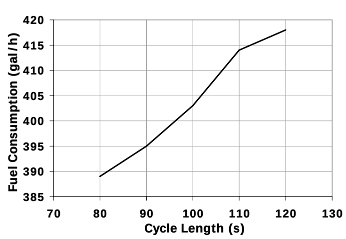
Exhibit 29-4
Cycle Length Optimization Results



(a) Stops Optimization



(b) Delay Optimization



(c) Fuel Consumption Optimization

The split and offset optimization was carried out next. The resulting timing plan is shown in Exhibit 29-5. This table represents the initial timing plan to be investigated and refined.

Exhibit 29-5
Timing Plan Developed by Split and Offset Optimization

Intersection	Offset	Phase 1	Phase 2	Phase 3	Phase 4	Total
1	0	13	29	13	25	80
2	34	45	35			80
3	3	13	33	34		80
4	31	45	35			80
5	78	13	29	13	25	80

Notes: All times are in seconds.
Offsets are referenced to the first arterial through-traffic phase.

INITIAL TIMING PLAN PERFORMANCE

A summary of the performance measures for the initial timing plan is presented in Exhibit 29-6. Separate columns are included in this table for *route totals*, which include only the segments that make up the urban street facility as defined in Chapter 16, and *system totals,* which include the measures from the cross-street segments. Note that some of the performance measures reported in this table are also reported by the Chapter 16 procedures. While the STPD tool definitions and model structures are similar to the HCM (e.g., uniform and random components), no comparison of the values will be offered in this discussion because the focus is on the STPD and not on modeling differences.

Exhibit 29-6
Performance Measures for the Initial Timing Plan

Performance Measure	Units	System Totals	Route Totals
Total travel	veh-mi/h	4,927	3,063
Total travel time	veh-h/h	240	120
Uniform delay	veh-h/h	95	34
Random delay	veh-h/h	22	8
Total delay	veh-h/h	116	43
Average delay	s/veh	23.5	17.4
Passenger delay	p-h/h	140	51
Uniform stops	veh/h	12,893	5,576
Uniform stops	%	72	63
Random stops	veh/h	1,277	440
Random stops	%	7	5
Total stops	veh/h	14,171	6,016
Total stops	%	79	68
Links with *d/c* >1		0	0
Links with queue overflow		0	0
Time jammed	%	0	0
Period length	s	900	900
System speed	mi/h	20.5	25.6
Fuel consumption	gal/h	387	195
Operating cost	$/h	3,063	1,049

The initial timing plan design was based on minimizing fuel consumption as a performance measure. The signal progression characteristics of this design are also of interest. The progression characteristics will be examined in both numerical and graphics representations. The numbers are presented in Exhibit 29-7 and are based on the progression performance measures that were defined earlier. The interference values indicate the proportion of time that a vehicle entering a link in the progression band would be stopped at the next signal. The PROS are accumulated from progression bands that pass through some adjacent signals along the route. The low progression efficiency and attainability and PROS values suggest that this design, while optimal in some respects, would not produce a very favorable motorist perception of progression quality.

Performance Measure	Westbound	Eastbound	Average
Bandwidth efficiency	10%	5%	8%
Progression attainability	28%	14%	21%
Interference	9%	10%	
PROS	30%	28%	29%

Exhibit 29-7
Progression Quality Measures for the Initial Design

ADJUSTMENTS TO IMPROVE PROGRESSION QUALITY

Because of the low quality of progression, it is logical to revisit the initial design with the objective of maximizing progression quality instead of minimizing fuel consumption. The same cycle length range (80 to 120 s) was used for this purpose, and the runs were repeated with the objective of maximizing PROS. The maximum value of PROS was obtained with the same cycle length and phase times as the initial design. The progression performance measures associated with this timing plan are shown in Exhibit 29-8. These measures do not differ substantially from the initial design, nor do the offsets. The total PROS value increased from 29% to 30%, but the performance was somewhat better balanced by direction. Thus, there is not a large trade-off between the objectives of maximizing performance and maximizing progression quality in this case.

A combination of factors peculiar to this example has led to the conclusion that the signal timing parameters for optimizing performance and progression are basically the same. The symmetry of the layout and phasing created a situation in which fuel consumption could be minimized by favoring either direction at the expense of the other. The balanced design was favored by the PROS optimization because it offered a minimal numerical advantage (30% versus 29%). One of the main reasons why both design approaches chose the lowest acceptable cycle length is that, as pointed out previously, the theoretical optimum cycle length was below the lowest acceptable cycle length.

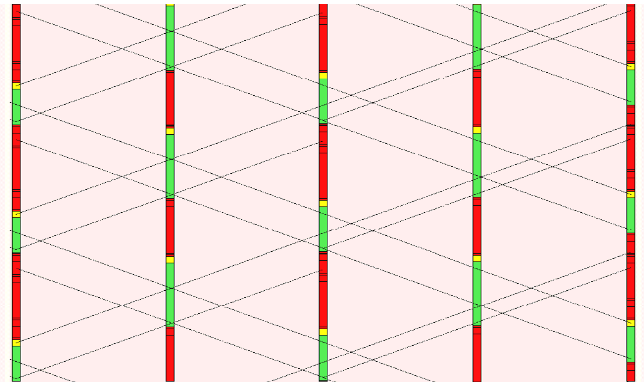
Performance Measure	Westbound	Eastbound	Average
Bandwidth efficiency	8%	8%	8%
Attainability	21%	21%	21%
Interference	9%	9%	
PROS	30%	30%	30%

Exhibit 29-8
Progression Quality Measures for the Improved Progression Design

TIME–SPACE DIAGRAMS

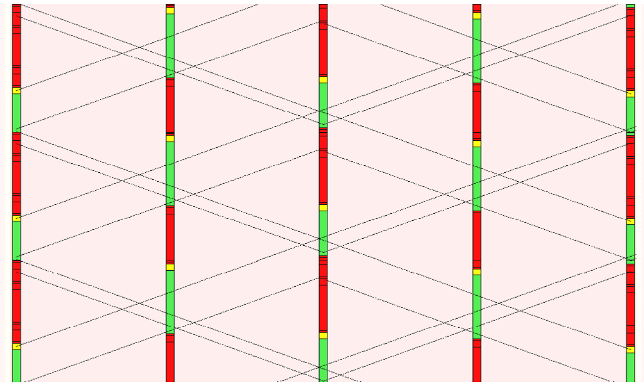
STPD tools typically produce graphic displays depicting progression characteristics. The most common display is the time–space diagram, which is well documented in the literature and understood by all practitioners. The time–space diagram reflecting the initial design is shown in Exhibit 29-9. Note that, even though the traffic volumes are balanced in both directions, the design appears to favor the westbound (right-to-left) direction. Because of the symmetry of this example, it is likely that a dual solution exists that yields the same performance but that favors the eastbound direction.

Exhibit 29-9
Time–Space Diagram for the Initial Design



The time–space diagram depicting the modified progression design is shown in Exhibit 29-10. This design shows a better balance between the eastbound and westbound directions. There is good progression into the system from both ends, but the band in both directions is halted at the center intersection. The PROS accumulation is evident in the bands that progress between some of the intersections.

Exhibit 29-10
Time–Space Diagram for the Modified Progression Design



There appears to be minimal difference between the initial and modified designs. The modified design will be chosen for further investigation because it offers a better balance between the two directions. The offset changes for this design are presented in Exhibit 29-11.

Exhibit 29-11
Offset Changes for the Modified Progression Design

Intersection	Initial Offsets	Revised Offsets
1	0	0
2	34	30
3	3	76
4	31	30
5	78	0

The time–space diagram for this operation from another STPD tool is shown in Exhibit 29-12. The timing plan is the same as the plan that was depicted in Exhibit 29-10, but the format of the display differs slightly. Both the link band and the arterial band as defined previously are shown on this display. The individual signal phases are also depicted. Both types of time–space diagrams offer a manual adjustment feature whereby the offsets may be changed by dragging the signal display back and forth on the monitor screen.

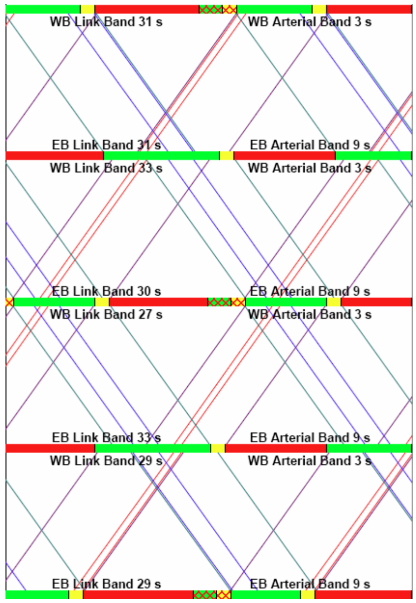


Exhibit 29-12
Alternative Time–Space Diagram Format

OTHER GRAPHIC DISPLAYS

Other graphics formats are not as ubiquitous as the time–space diagram but can provide useful insights into the operation at and between intersections.

Flow Profile Diagrams

One example is the flow profile diagram, which is simply a plot of the flow rate over one complete cycle. Flow profiles may be created to depict either the arrival or departure flows at a stop line.

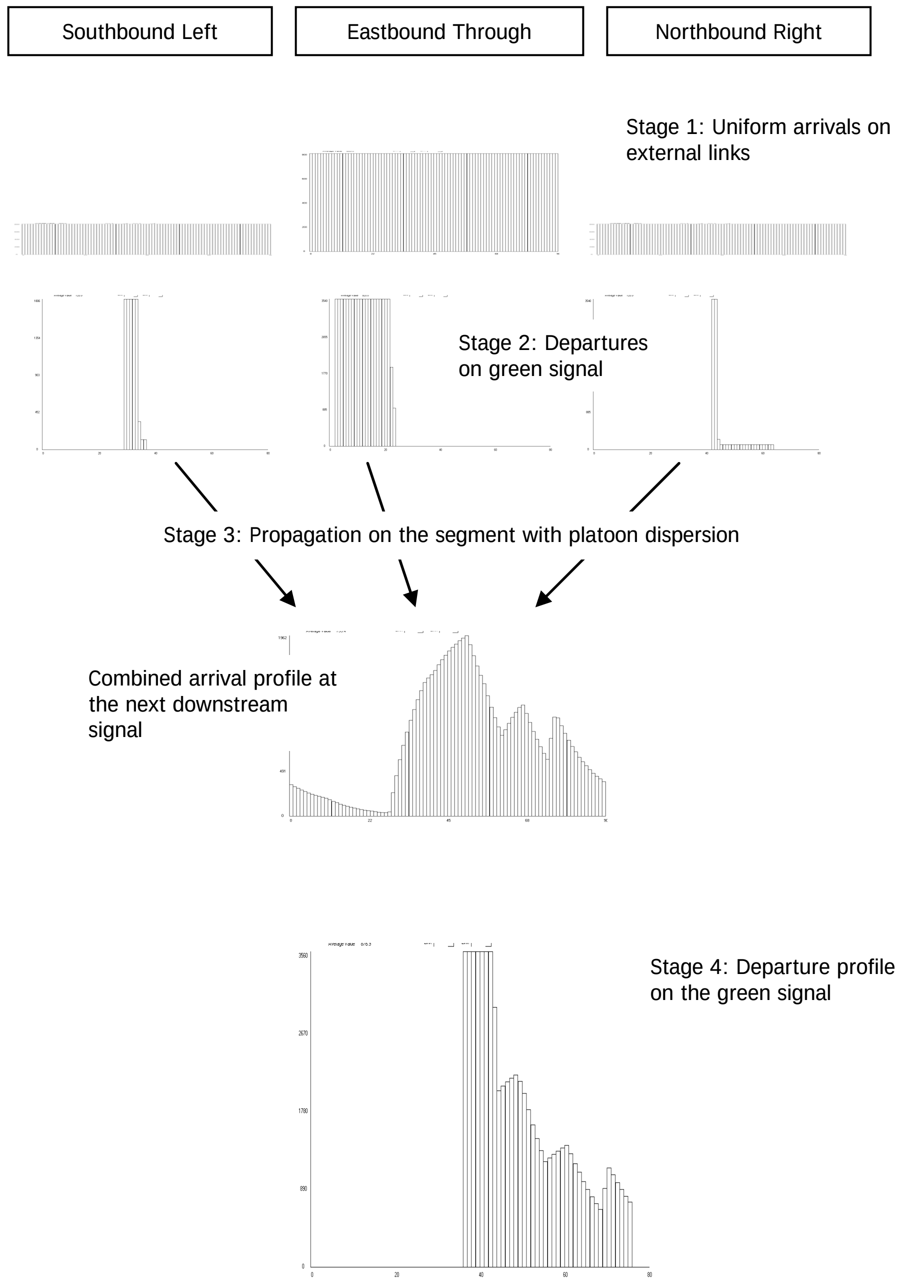
An example illustrating the use of flow profiles is presented in Exhibit 29-13. The eastbound segment between the first and second intersections is depicted in this example. The traffic inputs to this segment come from three independent movements at Intersection 1: southbound left, eastbound through, and northbound right.

Four stages of the progress of traffic into and out of this segment are depicted in the exhibit:

1. *Uniform arrivals on external links*: Each of the three movements entering the segment will arrive with a flow profile that is constant throughout the cycle because of the absence of platoon-forming phenomena on external links.
2. *Departures on the green signal*: Each movement proceeds on a different phase and therefore enters the link at a different time.
3. *Propagation on the segment with platoon dispersion*: Each of the three movements will be propagated downstream to the next signal by using a model that applies the design speed and incorporates platoon dispersion. Arrival of the platoons at the downstream end of the segment: The composite arrival profile is illustrated in the figure. The profile represents the sum of all of the movements entering the link.

4. *Departure on the green signal*: The platoons are regrouped at this point into a new flow profile because of the effect of the signal. The extent of regrouping will depend on the proportion of time that the signal is green. If a continuous green signal were displayed, the output flow profile would match the input flow profile exactly.

Exhibit 29-13
Example Illustrating the Use of Flow Profiles



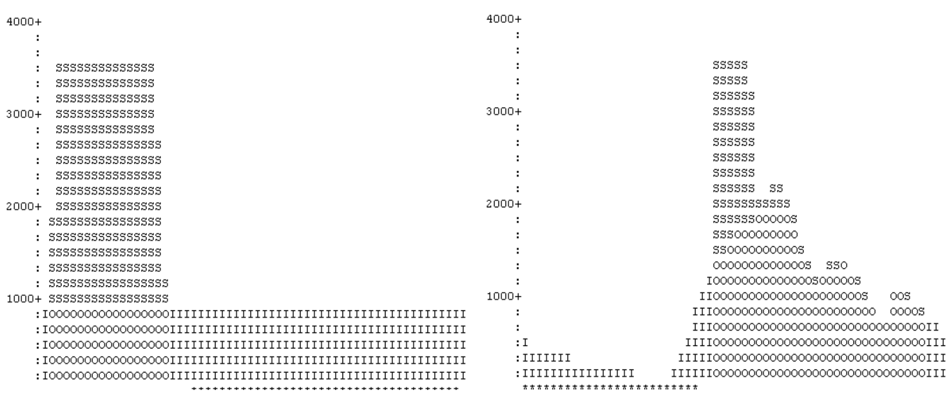
The departure profile for this movement forms one input to the next link and is therefore equivalent to Stage 2 in the list above. The vehicles entering on different phases from the cross street must be added to this movement to form the input to the next segment as the process repeats itself throughout the facility.

The preceding description of the accumulation, discharge, and propagation characteristics of flow profiles is of special interest to this discussion because the same models used by the STPD tool have been adopted by the analysis procedures given in Chapter 17, Urban Street Segments. These procedures are described by Exhibit 30-3 through Exhibit 30-5 in Chapter 30, Urban Street Segments: Supplemental. Therefore the graphical representations given in Exhibit 29-13 should provide a useful supplement to facilitate understanding of the procedures prescribed in Chapter 17.

Composite Flow Profiles

Another form of flow profile graphics is illustrated in Exhibit 29-14. This text-based display offers a composite view of the flow profiles by showing the arrival and departure graphics on the same figure represented by different characters. The uniform arrival pattern from the external link is evident at the upstream intersection, which corresponds to Stages 1 and 2 of Exhibit 29-13. The effect of the platooned arrivals is also evident at the downstream intersection, corresponding to Stages 4 and 5. More details on interpreting the composite flow profiles are given elsewhere (*3*).

Exhibit 29-14
Composite Flow Profiles for the First Eastbound Segment

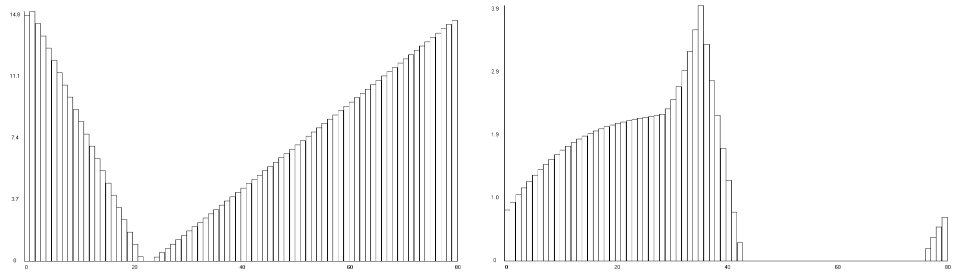


(a) Upstream Intersection (Uniform Arrivals) (b) Downstream Intersection (Platooned Arrivals)

Queue Length Graphics

The accumulation and discharge of queues can also be represented graphically in a manner that is consistent with the analysis procedures of Chapters 16 through 18. An example of graphics depicting the queue length throughout the cycle is presented in Exhibit 29-15. The upstream signal shows the familiar triangular shape that is the basis of the uniform delay equation. The downstream signal shows the effect of platooned arrivals on the length of the queue.

Exhibit 29-15
Variation of Queue Length Throughout the Signal Cycle for the First Eastbound Segment



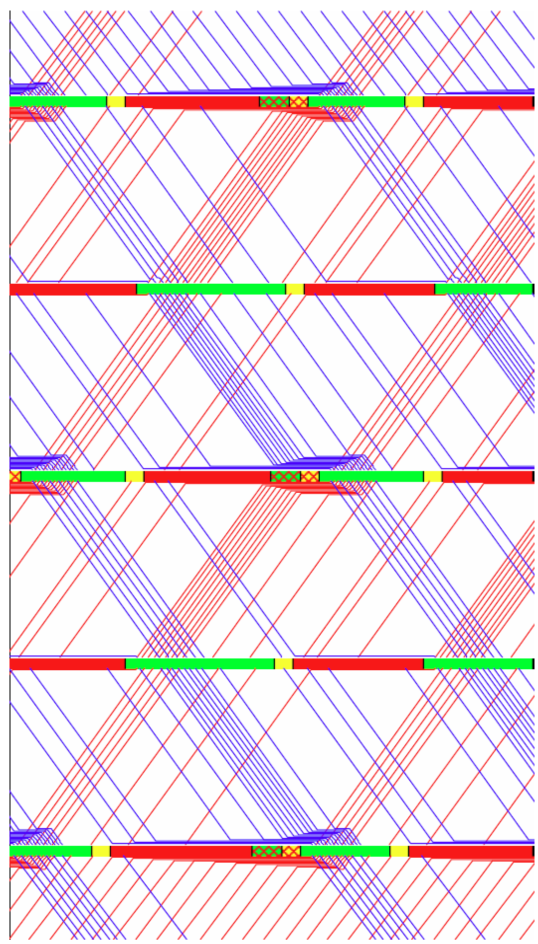
(a) Upstream Intersection Queue Length (Uniform Arrivals)

(b) Downstream Intersection (Platooned Arrivals)

Adding Flows and Queues to the Time–Space Diagram

One useful display superimposes the flow profiles and queuing characteristics on the time–space diagram to give a complete picture of the operation of the facility. An example of this display representing the improved progression design is presented in Exhibit 29-16. The flow rates are represented by the density of the lines progressing between intersections at the design speed. The queues are represented by horizontal lines upstream of each intersection. From this diagram, it is possible to visualize the effect of the design on queue accumulation and discharge and on the propagation of flows between intersections.

Exhibit 29-16
Time–Space Diagram with Flows and Queues



POTENTIAL IMPROVEMENTS FROM PHASING OPTIMIZATION

The quality of progression in this example was improved from the initial design, but the results leave room for further improvement. There are, for example, minimal arterial through bands. The current design was based on leading phases for all protected left turns. The operation might be improved by the application of lagging left-turn phases on some approaches. The procedures given in Chapter 17 are sensitive to the phase order. These procedures could be applied manually to seek a better operation. The use of STPD tools for this purpose will be demonstrated here because phasing optimization is internalized in the tools as a computational feature.

The phasing optimization process recommended changes at two of the five intersections. The phasing modifications are shown in Exhibit 29-17. Lead-lag phasing was applied at both intersections. As a result of the optimization, the arterial bandwidth increased from 6 to 16 s in both directions. The total signal delay decreased from 220 to 200 s/veh. The arterial speed increased from 22.1 to 23.0 mi/h. Thus, it is clear that the phasing optimization would improve both the progression quality and the operational performance of the route. The progression quality improvement is evident in the time–space diagram presented in Exhibit 29-18.

Exhibit 29-17
Optimized Phasing Modifications

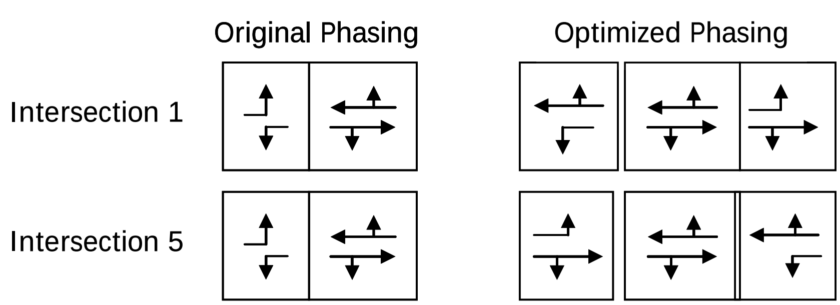
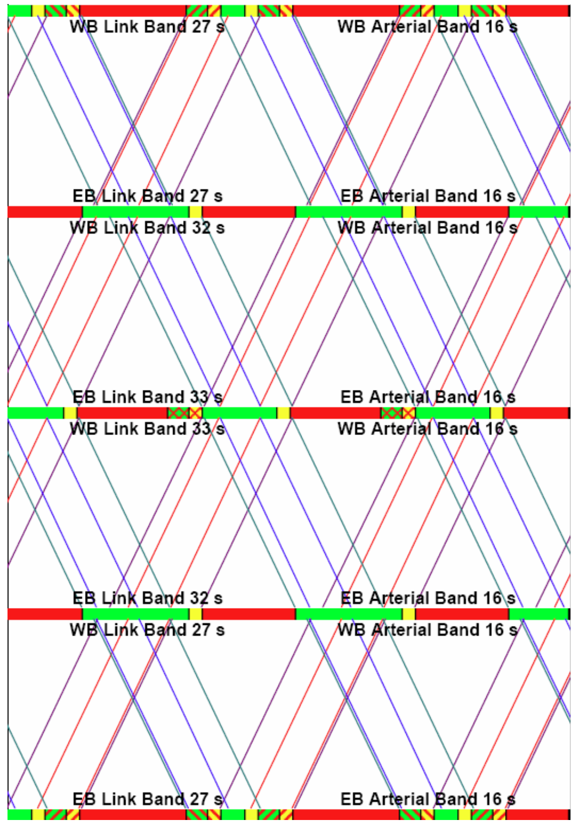


Exhibit 29-18
Time–Space Diagram for the Optimized Phasing Plan



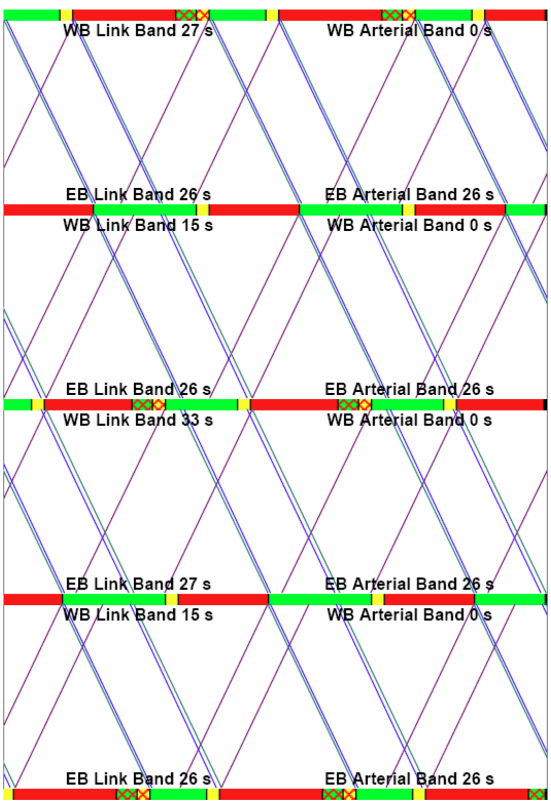
The decision to implement lead-lag phasing involves many factors including safety and local preferences. This discussion has been limited to a demonstration of how STPD tools can be used in the assessment of the operational effects of phasing optimization as one input to the decision process. The suggested modifications will not be implemented in the balance of the examples.

4. EFFECT OF MIDSEGMENT PARKING ACTIVITIES

HCM procedures recognize some midsegment activities such as cross-street entry between signals and access-point density. A procedure is provided in the methodology for estimating the delay due to vehicles turning left or right into an access-point approach. No procedures are included for estimating the delay or stops due to other causes such as pedestrian interference and parking maneuvers. Alternative tools must be used to assess these effects.

This section will demonstrate the use of a typical microscopic simulation tool (*4*) to assess the effects of midsegment parking maneuvers on the performance of an urban street facility. The signal timing plan example from the previous section of this chapter will be used for this purpose. The offsets will be modified first to create "ideal" progression in the eastbound direction at the expense of the westbound flow. The investigation will focus on the eastbound flow. The offsets and time–space diagram depicting this operation are shown in Exhibit 29-19. Offset 1 is referenced to the first phase for arterial through movements. Offset 2 is referenced to Phase 1. Their values will differ because of leading left-turn phases at some intersections. Different tools require different offset references.

Exhibit 29-19
Time–Space Diagram Showing Ideal Eastbound Progression



Signal	Offset 1	Offset 2
1	0	0
2	35	47
3	63	68
4	23	35
5	57	56

The treatment of parking maneuvers by the selected simulation tool is described in the tool's user guide (*4*). The following parameters must be supplied for each segment that contains on-street parking spaces:

- Beginning of the parking area with respect to the downstream end of the segment,

- Length of the parking area,
- Mean duration of a parking maneuver, and
- Mean frequency of parking maneuvers.

The occurrence and duration of parking maneuvers are randomized around their specified mean values. The parameters that will be used in this example are shown in Exhibit 29-20.

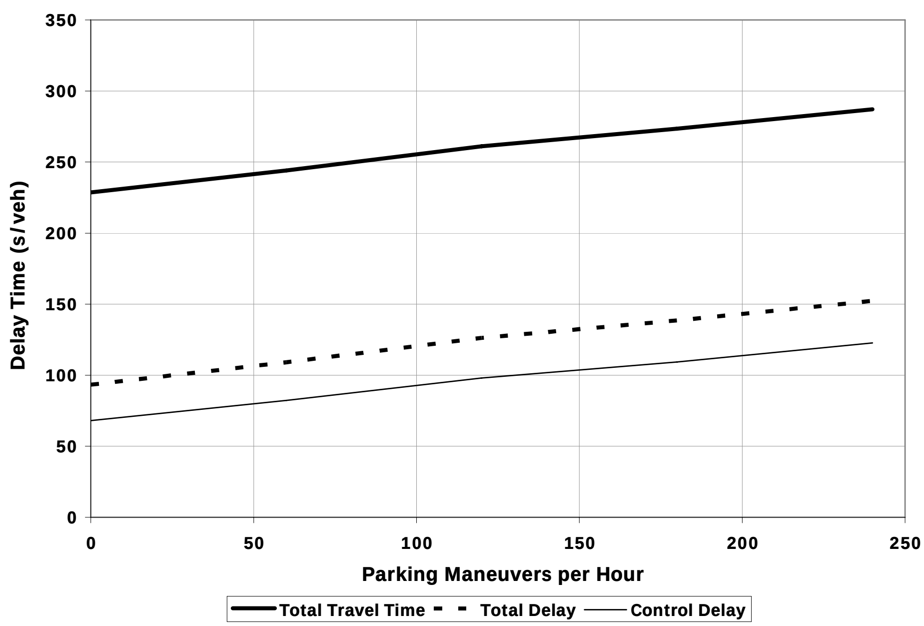
Exhibit 29-20
Parameters for the Parking Example

Parameter	Value
Beginning of the parking area	200 ft from the downstream intersection
Length of the parking area	1,600 ft (leaving 200 ft to the upstream intersection)
Mean duration of a parking maneuver	30 s
Mean frequency of parking maneuvers	0 veh/h (no parking maneuvers) 60 veh/h 120 veh/h 180 veh/h 240 veh/h Represents a range of approximately 15 min to 60 min average parking duration

The simulation runs covered 80 cycles of operation. Separate runs were made for each level of parking frequency. The default simulation parameters of the selected tool were used.

The effect of the parking activity on travel time and delay is presented in Exhibit 29-21, which shows the total travel time for the facility as well as the two delay components of travel time (total delay and control delay). Each of the values represents the sum of the individual segment values. The graphs demonstrate that all of the relationships were more or less linear with respect to the parking activity level.

Exhibit 29-21
Effect of Parking Activity Level on Travel Time and Delay



The effect of the parking activity on stops is presented in Exhibit 29-22. For this example, the average percentage of stops for all eastbound vehicles increased from slightly more than 40% to slightly less than 60% throughout the range of parking activity levels. Both of these exhibits indicate that the simulation tool was able to extend the capability for analysis of urban street facilities beyond the stated limitations of the methodology presented in Chapter 16.

Exhibit 29-22
Effect of Parking Activity Level on the Percentage of Stops

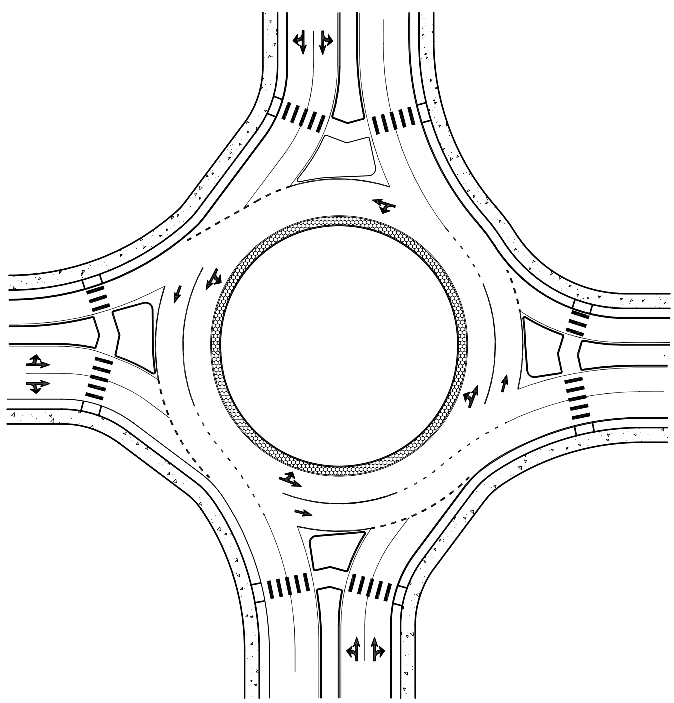
5. EFFECT OF PLATOONED ARRIVALS AT A ROUNDABOUT

Chapter 21, Roundabouts, describes a procedure for analyzing the operation of an isolated roundabout. The procedure does not deal with a roundabout operating within a signalized arterial, so no such treatment has been incorporated into the urban street segments analysis procedure. Therefore, the analysis of a roundabout as a part of a coordinated traffic control system is likely better accomplished with alternative tools. The alternative deterministic tools described earlier in this chapter do not deal explicitly with roundabouts in coordinated systems. Most simulation tools offer some roundabout modeling capability, although the level of modeling detail varies among tools.

This section describes the use of a typical simulation tool (*5*) in analyzing a roundabout within the arterial configuration of the previous example in this chapter. For this purpose, Intersection 3 at the center of the system will be converted to a roundabout with two lanes on each approach. To simplify the discussion, a basic symmetrical configuration will be used, because the discussion will be limited to the effect of platooned arrivals on the operation. The design aspects of roundabouts are covered in Chapter 21, Roundabouts, with more details provided in Chapter 33, Roundabouts: Supplemental, and elsewhere (*6*). The default traffic modeling parameters of the simulation tool will be applied.

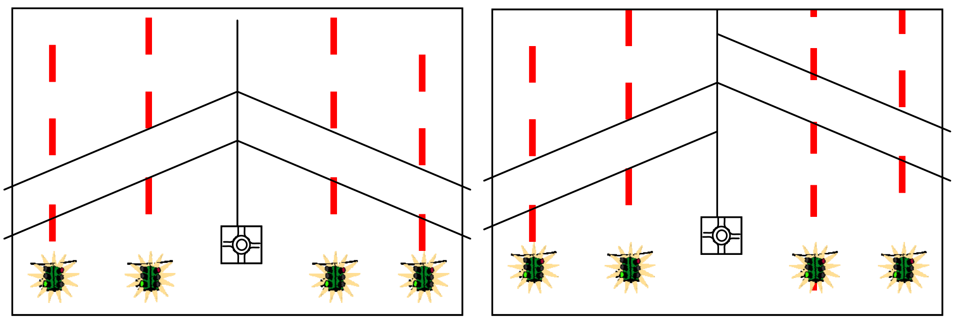
The roundabout configuration is shown schematically in Exhibit 29-23.

Exhibit 29-23
Roundabout Configuration for Intersection 3



This example will examine two STPDs that create substantially different platoon arrival characteristics on the arterial approaches to the roundabout. The time–space diagrams representing the two designs are shown in Exhibit 29-24. The first design provides simultaneous arrival of the arterial platoons from both directions. The second creates a situation in which one platoon will arrive in the first half of the cycle and the other will arrive during the second half. The two cases will be described as "simultaneous" and "alternating" platoon arrivals.

Exhibit 29-24
Time–Space Diagrams Showing Simultaneous and Alternating Platoon Arrivals at the Roundabout



(a) Simultaneous Platoon Arrivals (b) Alternating Platoon Arrivals

The platoon arrival characteristics can only be expected to influence the operation of a roundabout with relatively free-flowing traffic. While a two-lane roundabout could accommodate the demand volumes used in the previous examples in which the intersection was signalized, the initial simulation runs indicated enough queuing on all approaches to obscure the effect of the progression design. Since the focus of this example is on the effect of the adjacent signal timing plan, the demand volumes on the cross-street approaches to the roundabout will be reduced by 100 veh/h (approximately 17%) to provide a better demonstration of that effect.

Ten simulation runs were performed for both progression designs, and the average values of the performance measures were used to compare the two designs. The performance measures illustrated in Exhibit 29-25 include delay and stops on all approaches to the roundabout and travel times on individual link segments and on the route as a whole.

Exhibit 29-25
Performance Comparison for Simultaneous and Alternating Platoon Arrivals at a Roundabout

Movement	Alternate	Simultaneous	Difference	Percent
Delay				
Major-street approaches	15.81	14.18	1.64	10.34
Minor-street approaches	19.36	19.88	–0.52	–2.69
Stops				
Major-street approaches	0.59	0.52	0.08	12.71
Minor-street approaches	0.88	0.89	–0.01	–0.57
Average Travel Times				
Through vehicles traveling the full route	250.60	237.74	12.86	5.13
Approach links	58.06	56.30	1.76	3.03
Exit links	50.76	45.66	5.10	10.05

As a general observation, the simultaneous design performed noticeably better than did the alternating design on the major street, with a slight degradation to the cross-street performance. Travel times for vehicles traveling the full length of the facility were improved by about 5%. Travel times on the arterial segments entering and leaving the roundabout were improved by 3% and 10%, respectively.

This example has demonstrated that the simulation tool was able to describe the effect of two signal progression schemes on the performance of a roundabout within a coordinated arterial signal system. The next example will deal with the same basic arterial layout except that the roundabout will be replaced by a two-way STOP-controlled (TWSC) intersection. The platoon arrival types can be expected to have a greater influence on the TWSC operation than the roundabout because the effect is much more direct. Major-street vehicles always have the right-of-way over minor-street vehicles. Simultaneous platoons arriving from both directions will provide more opportunity for gaps in the major-street flow. Alternating platoons will keep major-street vehicles in the intersection for a greater proportion of time, thereby restricting cross-street access.

The effect at a roundabout is much more subtle because minor-street vehicles have the right-of-way over major-street vehicles once they have entered the roundabout. With simultaneous arrivals, platoons from opposite directions assist each other by keeping the minor-street vehicles from entering and seizing control of the roadway. When there is no traffic from the opposite direction, as in the case of alternating arrivals, a major-street movement is more likely to encounter minor-street vehicles within the roundabout. This phenomenon explains the 10% improvement in performance for simultaneous arrivals in the roundabout example as indicated in Exhibit 29-25.

6. QUEUE LENGTH ANALYSIS BASED ON VEHICLE TRAJECTORIES

The HCM's segment-based chapters provide deterministic procedures for estimating the extent of queue backup on either signalized or unsignalized approaches. Most of the procedures are sensitive to some degree to platoon formation from adjacent signals. Most provide estimates of the average back of queue (BOQ) and the expected BOQ at some level of probability.

One additional queuing measure that can be derived from simulation is the proportion of time that the BOQ might be expected to extend beyond a specified point. This measure can be obtained directly from the analysis of individual vehicle trajectories by using the procedures set forth in Chapter 7, Interpreting HCM and Alternative Tool Results, and Chapter 24, Concepts: Supplemental. Those procedures will be applied in this example to examine the queuing characteristics on the minor-street approach to a TWSC intersection operating within a signalized arterial system. The criteria and procedures prescribed in Chapter 24 for identifying the onset and release from the queued state will be used.

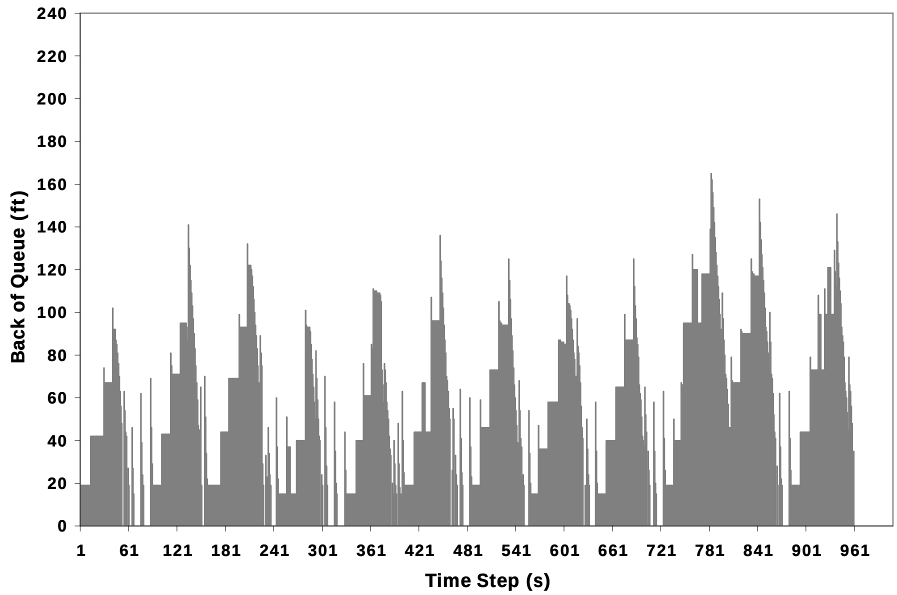
The same urban street configuration will be used for this purpose. The center intersection that was converted to a roundabout in the previous example will now be converted to TWSC. Because of the unique characteristics of TWSC, a few changes will have to be made to the configuration. Because TWSC capacities are lower than those of signals or roundabouts, it will be necessary to reduce the minor-street demand volumes. The two-lane approaches will be preserved, but the additional left-turn bay will be eliminated. The same two platoon arrival configurations (simultaneous and alternating) will be examined to determine their effect on the minor-street queuing characteristics. The signal timing plans from the roundabout example, as illustrated in Exhibit 29-24, will also be used here. Twelve cycles covering 960 s will be simulated for each case to be examined, and the individual vehicle trajectories will be recorded.

QUEUING CHARACTERISTICS

The first part of this example will demonstrate TWSC operation with an idealized scenario to provide a starting point for more practical examples. Two intersecting streams of through movements with completely uniform characteristics will be simulated. As many of the stochastic features of the simulation model as possible will be disabled. This is a highly theoretical situation with no real practical applications in the field. Its purpose is to provide a baseline for comparison.

The formation of queues under these conditions is illustrated in Exhibit 29-26, which shows the instantaneous BOQ at all time steps in the simulation. The cross-street entry volume was 600 veh/h in each direction, representing approximately the capacity of the approach. The cyclical operation is quite evident here, with 12 discernible cycles observed. Each cycle has a similar appearance. The differences among cycles are due to embedded stochastic features that could not be disabled.

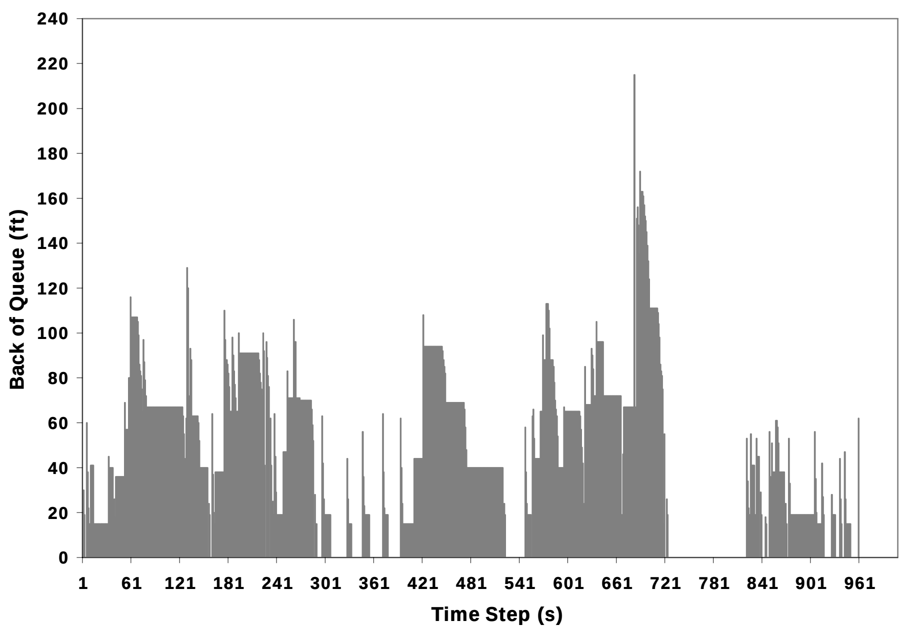
Exhibit 29-26
Queuing Results for the Theoretical Example



The signal timing plan with simultaneous platoon arrivals should produce the most cyclical operation that could actually be observed in the field. This configuration was simulated by loading the minor street to near capacity levels as determined experimentally. The entry volume was 350 veh/h.

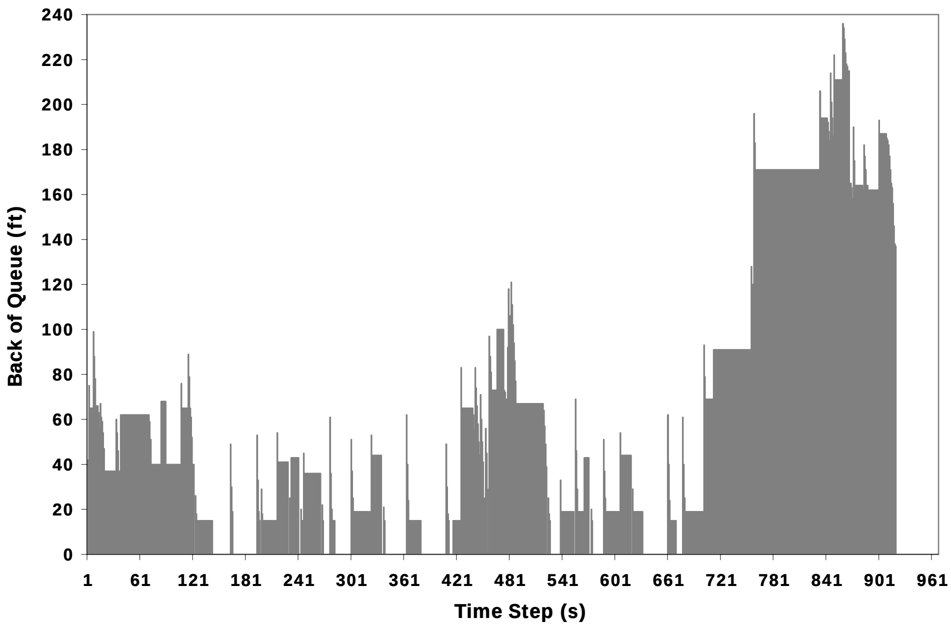
The queuing results are shown in Exhibit 29-27. Some cyclical characteristics are still evident here, but they are considerably diminished from the idealized case. The loss of cyclical characteristics results from cross-street turning movements entering the segments at their upstream intersections and from the general stochastic nature of simulation modeling.

Exhibit 29-27
Queuing Results for Simultaneous Platoons



The operation was simulated next with alternating platoon arrivals. Again the demand volumes were set to the experimentally determined approach capacity, which was 270 veh/h, or about 25% lower than the capacity with simultaneous platoons. The results are presented in Exhibit 29-28. Some further loss of cyclical properties due to the spreading of entry opportunities across a greater proportion of the cycle is observed here.

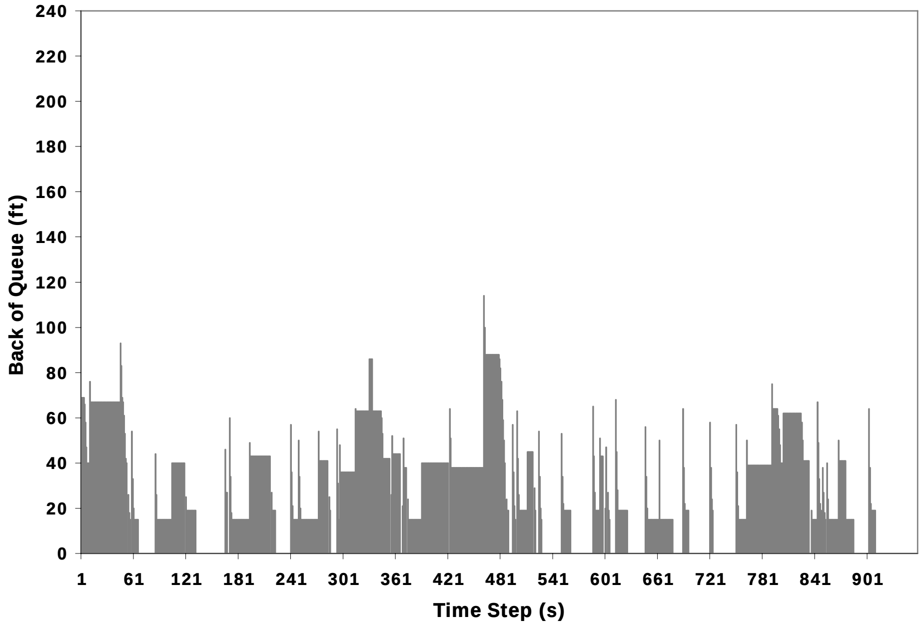
Exhibit 29-28
Queuing Results for Alternating Platoons



The least cyclical characteristics would be expected from simulation of a completely isolated operation. The 2,000-ft link lengths were retained for this case, but no adjacent intersections existed. All other parameters remained the same, including the entry volume because the entry capacity for isolated operation was found to be the same as the case with alternating platoons.

The results are presented in Exhibit 29-29. There are no cyclical characteristics here because there is no underlying cycle in the operation. Also, even with the same entry volume as the alternating platoon case, the peak BOQs are much lower. This is because the entry opportunities are distributed randomly in time instead of being concentrated at specific points in the cycle.

Exhibit 29-29
Queuing Results for Isolated TWSC Operation



BOQ ASSESSMENT

The discussion to this point has focused on instantaneous BOQs in an effort to understand the general nature of queuing under the conditions that were examined. With knowledge of the instantaneous BOQ values available from simulation, it is possible to produce useful performance measures related to queuing from simulation. One such measure is the proportion of time that a queue would be expected to back up beyond a specified point. This concept is different from the probability of backup to that point normally associated with deterministic tools. The balance of the discussion will deal with proportion of time with queue backup (PTQB) beyond a specified point.

The three cases examined in this example were simulated with cross-street demand volumes of 80, 160, 240, 320, and 400 veh/h, and the PTQB characteristics were determined by simulation for each case. The results were plotted for a specified distance of 100 ft from the stop line as shown in Exhibit 29-30. Each case is represented by a separate line that shows the percentage of time that the queue would be expected to back up beyond 100 ft from the stop line for each cross-street entry volume level. The simultaneous platoon case showed the lowest BOQ levels, starting with no time with BOQ beyond 100 ft below 240 veh/h, and reached a value of 80% of the time at the maximum volume of 400 veh/h. Predictably, the isolated case was the most susceptible to queue backup, and the alternating platoon case fell somewhere in between.

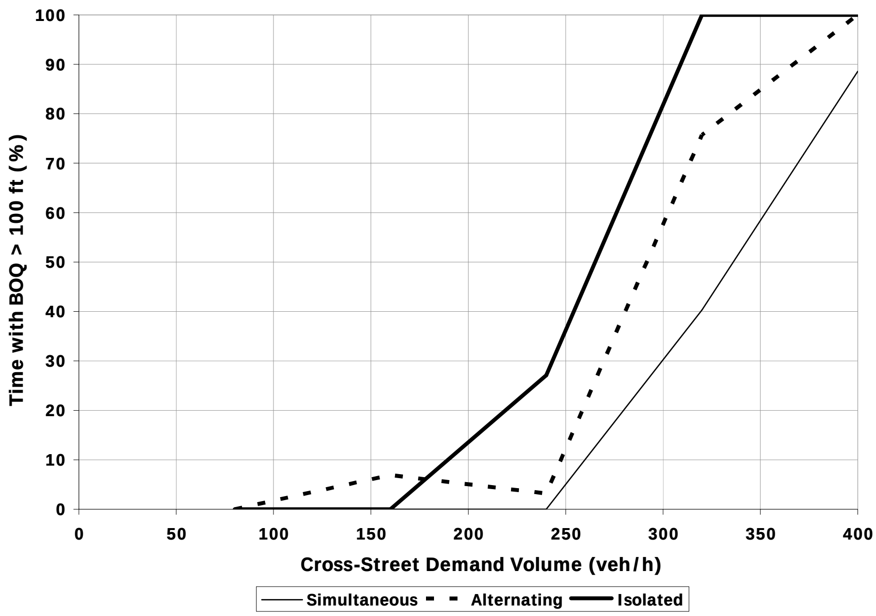


Exhibit 29-30
Effect of Cross-Street Demand Volume on Queue Backup Beyond 100 ft from the Stop Line

This example has demonstrated the use of simulation to produce potentially useful queuing measures based on the analysis of individual vehicle trajectories. It has also demonstrated how simulation can be used to assess the queuing characteristics of a minor-street approach to a TWSC intersection operating in a coordinated signal environment.

7. REFERENCES

1. Koonce, P., L. Rodegerdts, K. Lee, S. Quayle, S. Beaird, C. Braud, J. Bonneson, P. Tarnoff, and T. Urbanik. *Traffic Signal Timing Manual.* Report FHWA-HOP-08-024. Federal Highway Administration, Washington, D.C., June 2008.
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4. *Corridor-Microscopic Simulation Program (CORSIM) Version 6.1 User's Guide.* University of Florida, Gainesville, 2008.
5. *VISSIM 5.10 User Manual.* PTV Vision, Karlsruhe, Germany, 2008.
6. Robinson, B. W., L. Rodegerdts, W. Scarbrough, W. Kittelson, R. Troutbeck, W. Brilon, L. Bondzio, K. Courage, M. Kyte, J. Mason, A. Flannery, E. Myers, J. Bunker, and G. Jacquemart. *Roundabouts: An Informational Guide.* Report FHWA-RD-00-067. Federal Highway Administration, Washington, D.C., June 2000.