APPENDIX J
Traffic Analysis

Inner Harbor Navigation Canal Lock Replacement Project
Traffic Impact Analysis

Prepared for : US Army Corps of Engineers
GSRC Consultants

Prepared by: Regional Planning Commission
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Table of Contents

I. Introduction

In anticipation of potential impacts that its proposed Inner Harbor Navigation Canal Replacement might have on traffic moving in the Claiborne/ St. Claude corridor, the US Army Corps of Engineers contacted the Regional Planning Commission for Jefferson, Orleans, Plaquemines, St. Bernard, and St. Tammany Parishes (RPC) to discuss ways of determining what the traffic impacts of the project might be and what could be done to mitigate such impacts (See Figure 1 for Study Area). This report is a follow-up analysis that was originally undertaken in 1993.

As a result, the RPC, working with the Corps of Engineers, has undertaken the traffic impact analysis which is the subject of this report. The goals of the study are twofold-

1) To determine, using the RPC's sophisticated travel demand modeling capabilities, the potential impacts of the project on traffic crossing the IHNC during various phases of construction as well as after completion of the project.

2) To devise a set of operational alternatives to mitigate, to the extent possible, the delay, congestion, and accident potential associated with the project.

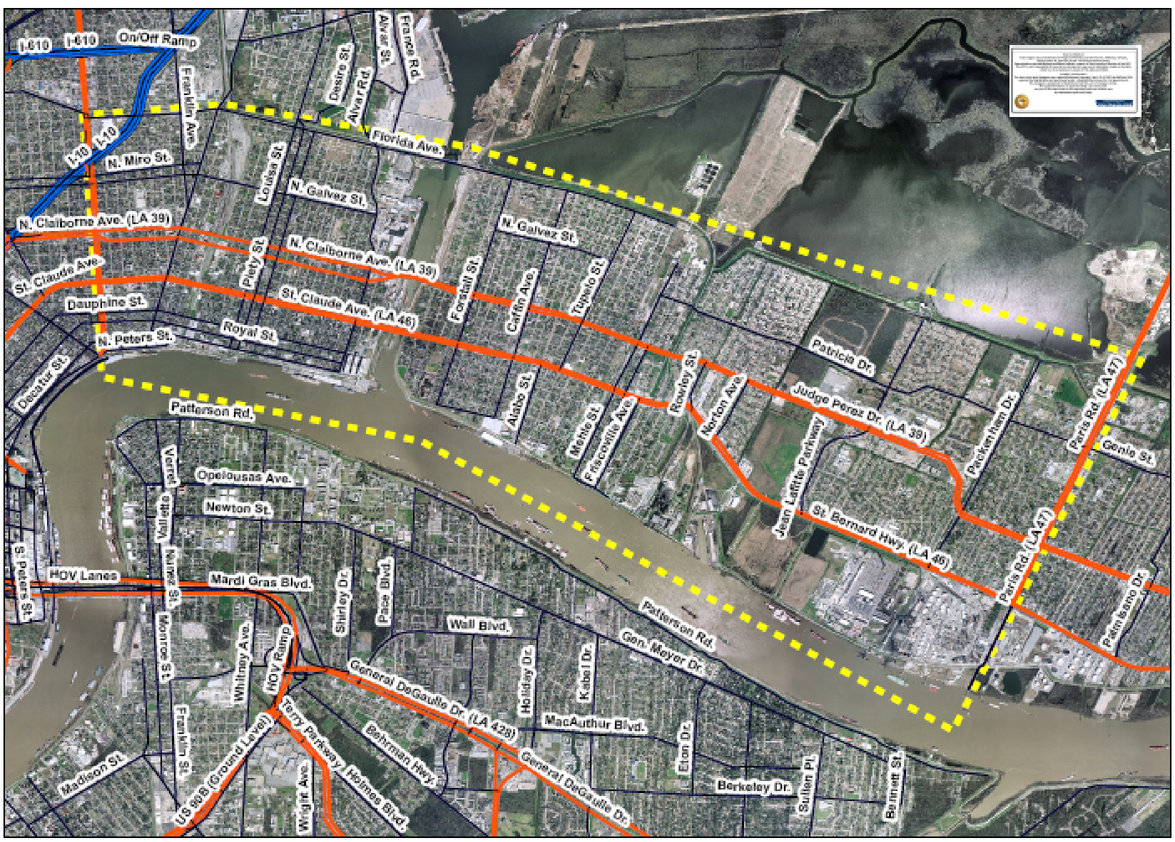

Figure 1: Borders of and major corridors within and near the study area.

Through meetings between the RPC staff and the Corps of Engineers project team, it was decided that the travel demand model runs of the various scenarios should be conducted in roughly chronological order. This approach would allow time for development of future year population projections necessary in the later runs. It would also provide a more natural flow of the steps in the model process such as defining networks for new scenarios by updating from previous versions as the work progressed.

In general terms the scenarios were:

1) A base year validation run
2) The future without project scenarios
3) Construction phase scenarios
4) A horizon year in which all projects in the Metropolitan Transportation Plan for the area are implemented.

The following report contains a discussion of the methods by which these various analyses were carried out, the results of the analyses, and the RPC recommendations with respect to mitigation of traffic impacts of the INHC Lock Replacement Project.

Hurricane Katrina

On August 29, 2005, Hurricane Katrina made landfall twice in the New Orleans metropolitan area, the first in Buras, Louisiana and the second along the border of Louisiana and Mississippi. The ensuing devastation was unprecedented in modern American history. The project area for this study, namely the “upper” Ninth Ward, the “lower” Ninth Ward in Orleans Parish, and St. Bernard Parish as a whole were especially hard hit. Less than one month later (9/24/2005), Hurricane Rita passed to the south of the New Orleans area to make landfall along the border of Louisiana and Texas. While wind damage was minor, hastily reconstructed temporary levees along the Inner Harbor Navigation Canal were again inundated by the surge of the passing Category 5 storm. The inundation of these areas forced the displacement and relocation of ***hundreds of thousands*** of area residents, and has had far-reaching impacts across the nation. The hurricanes have proven not only to be the costliest, but the scale of devastation is unprecedented in modern American experience. Many of these displaced residents have, as of this writing, resettled elsewhere within the region, or out of the New Orleans urbanized area entirely. It is anticipated that many will never return, or may repopulate their former neighborhoods over a long period of time.

Hurricane Katrina and the Impact on Travel Forecasting and Modeling

Estimates of Existing Socio-Economic Conditions

Forecasting the demand for daily household travel depends highly on use of socio-economic data, such as population, income, employment, etc. Moreover, the travel models rely on socio-economic data based on geography not only at the sub parish level, but at the sub-census tract and in some instances, sub-block group level. Since Hurricane Katrina and the diaspora created therefrom, empirical socio-economic data, particularly for this study area, have been extraordinarily difficult to gather, mostly because reliable data simply do not exist at this level of geography.

RPC, as well other organizations, have struggled to ascertain estimates of socio-economic characteristics of the regions' remaining/returning population. RPC has reviewed multiple sources of information- public, proprietary, and those developed by RPC staff- to use as the basis for developing forecast data. RPC has chosen to use a combination of these sources to create datasets that RPC staff believes to be the most accurate based on the best methodologies and the staffs' best professional judgment.

Forecast Data

Most long range transportation planning efforts use empirical socio-economic data observed over many years as a basis to extrapolate trends into the future. Hurricane Katrina has completely changed that paradigm. Population, employment and socio-economic growth trends that were years and decades in the making had accelerated into a timeframe of about three months in some parts of the region immediately after the storm. Some relocatees remained at the location where they originally evacuated. Others returned and set about repairing and rebuilding their homes and properties immediately after the storm. Still others have made slow, incremental progress toward returning to their former residences, or have found other housing near their former residences. The reasons and timeframes for these decisions vary as widely as the number of affected persons.

RPC has used the datasets of existing socio-economic conditions as a basis to develop forecast data over the course of approximately thirty years, to the year 2038. Prior to Hurricane Katrina, growth in the New Orleans Metropolitan Area overall had been extremely slow, at a general rate of 1% per decade for the entire region[1]. RPC's post-Hurricane Katrina population estimates for the study area generally entail faster rates of growth for the near-term (within 10 years to approximately 2018), accounting for persons eventually returning to their communities of origin pre-Hurricane Katrina. Afterward, growth comes much more slowly, mirroring trends that were in place prior to Hurricane Katrina.

II. Demand Analysis Methodology

A. Model Validation

In the course of its responsibilities as the Metropolitan Planning Organization for the Greater New Orleans region, the RPC maintains and applies a set of travel demand models for analysis and testing of projects proposed for inclusion in the region's long range Metropolitan Transportation Plan. The microcomputer highway models applied in this study use socioeconomic information regarding population, housing, income and employment to determine the number and nature of the trips likely to be taken in the corridor, their probable origins and destinations, and mode of travel. These estimated trips are then assigned to a highway network that provides an image of the street system

[1] US Census, New Orleans Urbanized Area SF3 files: 1980, 1990, 2000

available in the corridor. The model accounts for travel time, distance and cost of travel associated with each trip as well as delay due to congestion.

By comparing the results of the model runs under differing network conditions related to the various phases of the project an assessment can be made concerning the type and severity of impacts for alternate project scenarios and schedules. Before such analysis can be undertaken, however, the researcher must determine the reliability of the model's ability to replicate known conditions on the system. Only through such a validation process can confidence be established in the model's ability to predict future outcomes.

The year 2004 was chosen as the base year because of the availability of demographic and socioeconomic data from the decennial census in 2000. Most germane transportation planning products from the census, (namely the Census Transportation Planning Package or CTPP and SF-3) were available and recently updated by 2004. Additionally, the Regional Planning Commission had recently completed an update of the Travel demand model in early 2004, and had validated all components of it as part of the update.

Additionally, RPC used the calibrated 2004 Model update to undertake air quality conformity analyses for the region in 2004. The modeling methodology was thoroughly reviewed by LaDEQ, EPA, and FHWA in 2004 as part of the Air Quality conformity analysis undertaken at that time. EPA and FHWA both concurred with the analysis pursuant to the determination of Air Quality conformity on November 19, 2004.

The availability of this highly reliable data for input into the model recommended 2004 as the ideal base year for this stage of the effort. Available 2004 data was entered into the model input files and reviewed to insure there were no clerical errors.

B. Model Validation for 2008

The validation runs were begun by cleaning and debugging the existing travel demand models' 2004 highway network to correct any obvious errors in the way the system was coded. Particular attention was given to the St. Bernard/Orleans corridor. Roadway and transit improvements since 2004 were added to the network.

Transit data, fares and network configurations were obtained from RTA and SBURT and reviewed for accuracy. Since Hurricane Katrina, transit service in the project area has been significantly curtailed, particularly in the lower Ninth Ward and in St. Bernard Parish. These changes were accounted for in the model runs scenarios.

A complete 2008 run was then undertaken including trip generation, trip distribution, mode split and traffic assignment. A comparison between these traffic assignments and 2008 traffic count data obtained by the Regional Planning Commission as part of its ongoing Congestion Management System database. The general results are as follows-

1) the model's ability to predict the volumes of daily traffic at the three INHC bridge crossings appears to be excellent. The total corridor volume predicted by the model was

within 4% of the combined traffic counts for the three facilities. The error on individual facilities was also well within acceptable parameters, ranging from a low of 3% on N. Claiborne Avenue to a high of -21% on Florida Ave. Table 1 contains a comparative chart of the results and indicates typical error acceptability levels for similar facilities.

Table 1

Roadway	Model Assignment	Traffic Count	Error Observed	Typical Error
N. Claiborne	20,200	19,558	3%	10%
St. Claude	12,239	11,474	7%	14%
Florida	706	976	-28%	40%
Total	33,145	32,008	4%	10%

- Typical error figures from **Calibration of Systems Models** by Dane Ismart, FHWA.

2) model results, though excellent at the screen line created by the IHNC, varied somewhat as the distance from the canal increased. On the West Side of the canal, predicted volumes on Claiborne Ave. seemed somewhat high, while volumes on Galvez appeared low. On the East Side of the canal predicted volumes in St. Bernard as a whole seemed slightly lower than expected.

Overall, the model seems to be accounting for the changes estimated by the new, post-Katrina socio-economic inputs. Table 2 shows some historic perspective of traffic counts and volumes in the corridor. North Claiborne and St. Claude Avenues are both part of RPC's Congestion Management System, and are integral to regional mobility.

Table 2: Comparisons of Actual Traffic Counts: 1993 to 2008

Roadway	1993 Count	2004/05 Counts	2008 Count	% Difference (1993 -2005)	% Difference (2004-2008)
Florida Ave.	14,000	8,906	976	-36%	-89%
N. Claiborne	40,160	37,103	19,558	-8%	-47%
St. Claude	30,190	28,653	11,474	-5%	-60%
Totals	**84,350**	**74,662**	**32,008**	**-11%**	**-57%**

As is evident from the table, volumes were decreasing, albeit slowly, prior to Hurricane Katrina.

On the basis of limited screen line comparisons, the model appears to be adequately validated to undertake the proposed analysis. It should be noted, however, that post-Hurricane Katrina, travel patterns in the region have changed, perhaps significantly. Origins and destinations of travelers have also apparently changed, and that the modes used in the past may also be different.

C. Socioeconomic Forecasts

In order to project traffic for future year scenarios using the travel demand model, socio economic data must be developed to replicate the expected demographic make-up of the study area in the year being analyzed. More difficult yet is that such projections must be made at the level of small area analysis units called traffic analysis zones (TAZ's). RPC used population forecast data received from ***GCR Associates***, in coordination with their ongoing work with the parishes in the New Orleans area. In the New Orleans area, many TAZ's are geographically smaller than census block group data. Thus problems inherent in accuracy at sub-block group level are made much more complicated by the fact that there is very sparse information at the census tract level, or even at that parish level, particularly after Hurricane Katrina. Those reasons were described earlier and won't be repeated here.

Nonetheless, employment data is a key component of the New Orleans travel demand model. In the aftermath of Hurricane Katrina in 2005, the two Louisiana jurisdictions that suffered the greatest flood damage and standing water were Orleans and St. Bernard Parishes. This inundation caused the closing, relocation or downsizing of numerous businesses and other sources of employment. The Regional Planning Commission had previously used a combination of Census Transportation Planning Package (CTPP) journey-to-work data and proprietary business location and employment data gathered for marketing purposes to estimate and project employment figures by Traffic Analysis Zone. With the destruction wrought by the hurricane, much of this data was no longer valid.

To provide a new baseline for employment figures, RPC started with the pre-hurricane distribution of establishment-based employment by Traffic Zone. A detailed analysis of the severity of the flooding in each zone was then undertaken, using information provided to RPC by the LSU Coastal Studies Institute[2] (See Figure 2). File descriptions for data provided to RPC can be found in Appendix A. Factors were then applied to each zone to estimate employment in the first estimation year after the hurricane. RPC then looked at a number of independent sources for projections of retail and non-retail employment in the years to follow. These were then used to estimate employment for 2008 and to project these figures to the years 2013, 2014 and 2038 (See Figures 3, 4, 5). In terms of the New Orleans travel demand model, trip attractions are essentially a function of employment. Nonetheless, employment is not as critical a variable as population. Inherent in the model is the assumption that the trip production model is better that the trip attraction model. Therefore after performing trip generation, the model balances attractions to predicted productions.

[2] LSU Coastal Studies Institute, Water Depth Overlays for the New Orleans area, Dr. Dewitt Braud and Dr. Rob Cunningham, valid September 2, 2005

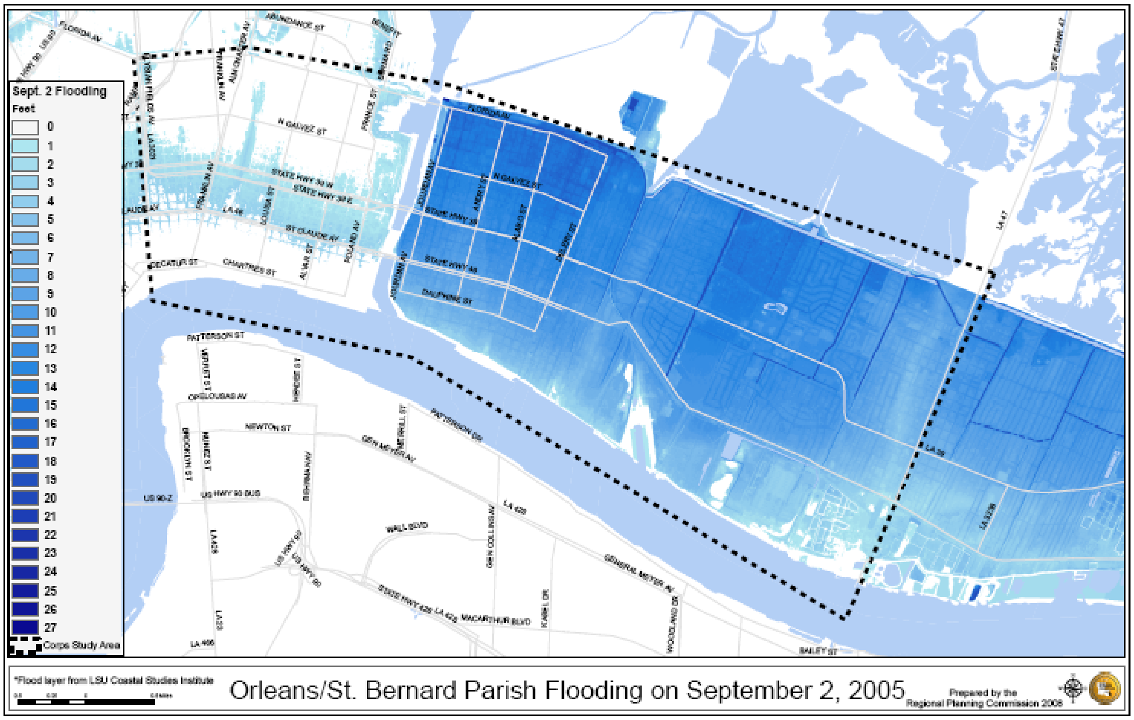

Figure 2: Extent of flooding in the study area on September 2, 2005.

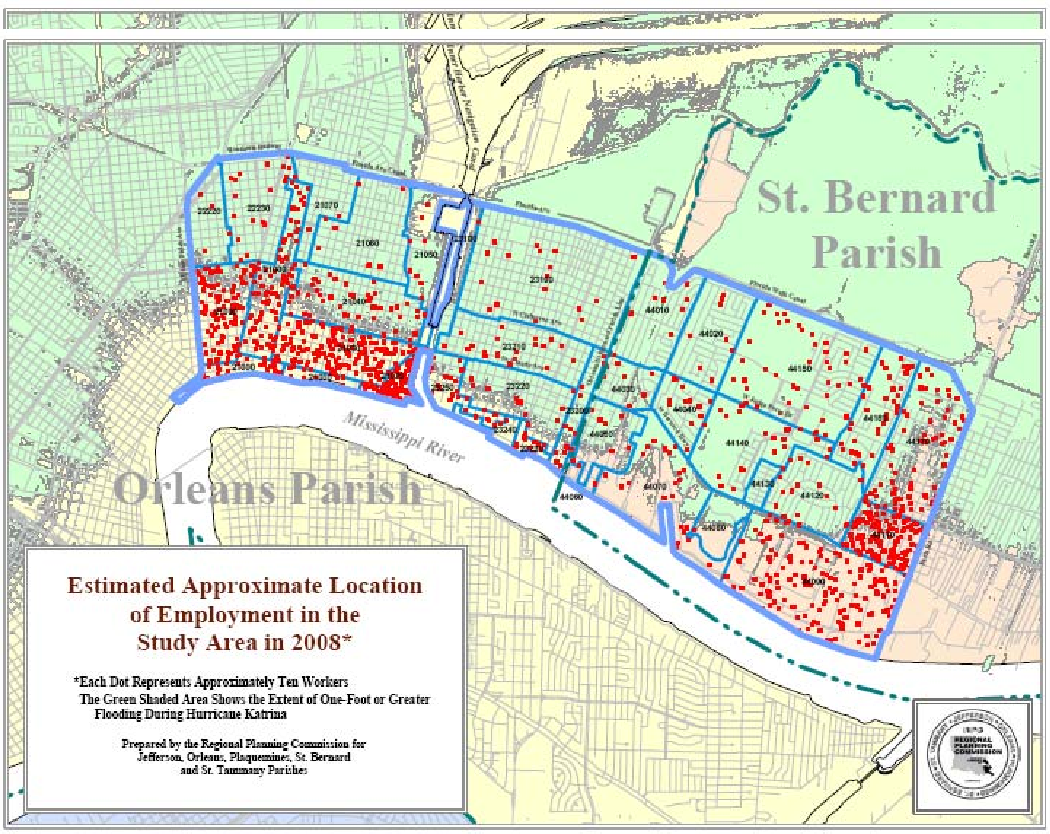

Figure 3: Estimated Employment in the study area in 2008.

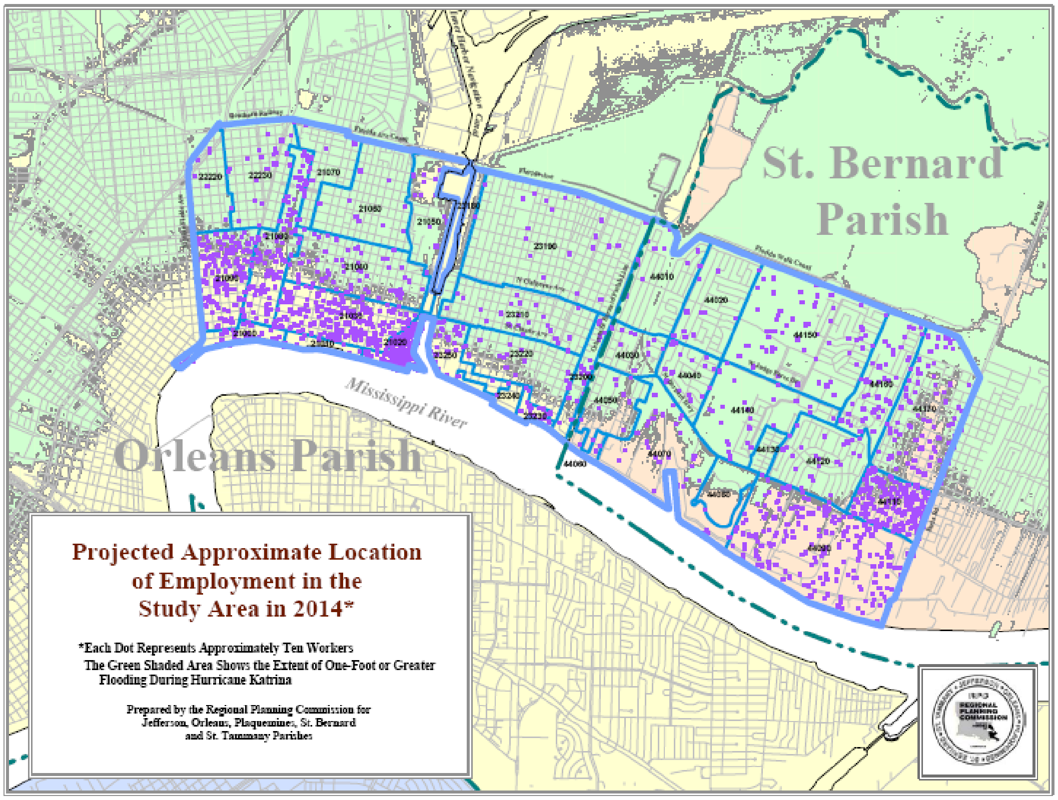

Figure 4: Projected employment in the study area in 2014.

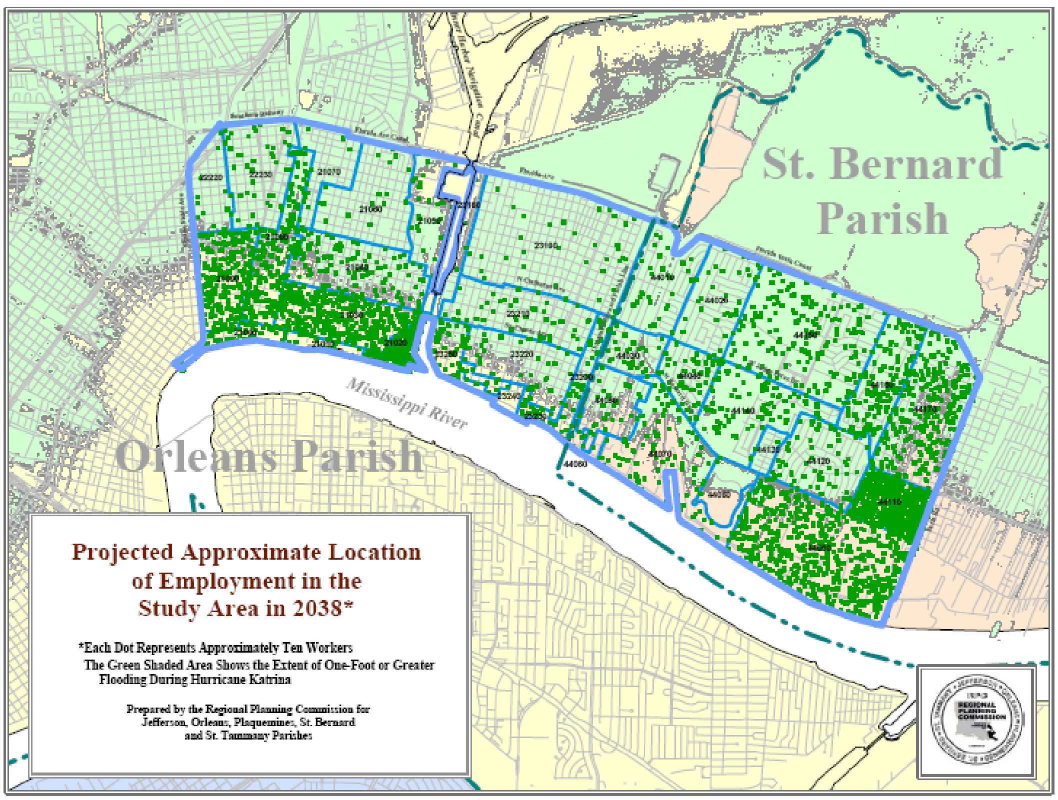

Figure 5: Projected employment in the study area in 2038.

3. Construction Year Scenarios

A) Introduction

One of the most critical elements of the demand analysis was the estimation of impacts during the construction phase of the lock replacement. In addition, changes in water levels under the Claiborne bridge would require heightening the towers on the existing structure, a process that would take the Claiborne bridge out of service for up to six months.

The Corps of Engineers anticipates constructions beginning no sooner than 2013 and being completed no later that 2015. Therefore, an Analysis year of 2014 was chosen for modeling the construction scenarios.

B) Florida Avenue High Rise Bridge:

In the original Lock Project Study in 1993, the Florida Avenue Bridge was part of the evaluation of alternatives, and “was a given in all scenarios evaluated in this study.” The original concept for the bridge included a six lane expressway that intersected with either I-10 or I-610 to the west, and Paris Road to the east. This scenario met with much opposition in the City of New Orleans, while St. Bernard Parish commuters clamored for relief. Since that time, LaDOTD has been working to implement the project as part of the TIMED program, or Transportation Infrastructure Model for Economic Development. After many years, a consensus on the alignment was reached wherein the project was scaled back significantly. The proposed alignment is as follows:

> The bridge over the IHNC would be a four lane high rise bridge. Control of access would begin at Alvar Street on the west side of the IHNC, not I-10 or at I-610. Traffic would be routed north to Alvar Street, toward US 90 and I-10 near Louisa, not west toward Elysian Fields Ave. The proposed roadway would continue as a four lane section, returning to grade at Caffin Street, and proceeding as a four lane section to Tupelo Street. The roadway would taper to two lanes east of Tupelo Street, and proceed over the levee and into the marsh (or the flood side) of the 40 Arpent Levee as a two lane section. Thence, the roadway proceeds east uninterrupted to Paris Road, LA 47, where the project terminates.

LaDOTD has undertaken an Environmental Assessment (EA) pursuant to 23 CFR 771 with the US Coast Guard (USCG) as the lead federal agency for the construction of a new high rise bridge at Florida Avenue[3]. The Finding of No Significant Impact (FONSI) was not issued at the time, primarily because the finding was dependent on the issuance of a USCG Bridge Permit. As part of the bridge permitting process, USCG requires notification of those property owners within a ½ mile radius of the proposed project. LADOTD and USCG worked to develop a database of these property owners and were initiating the notification process during the summer of 2005. On August 29, 2005, before the notification process and issuance of the FONSI, Hurricane Katrina struck, severely damaging the project area. After Hurricane Katrina, LaDOTD and USCG reassessed the project, and both believed an addendum to the EA was needed. Costs of the project were reassessed. The project rose in cost from the initial estimate of $156

[3] *New Florida Bridge over the Inner Harbor Navigational Canal (IHNC) Final Environmental Assessment*,LouisianaDepartment of Transportation and Development, May 2007

Million in 2005, to the latest estimate of $474 Million[4]. Due to the significant rise in the cost of construction in the New Orleans area post-Hurricane Katrina (all construction), LaDOTD has postponed the advancement of the Florida Avenue Bridge Project indefinitely. Had the project remained on its initial timeline from 2005, the Florida Avenue Bridge project could have been implemented within the 2014 build scenario for this evaluation. Since the project has been delayed indefinitely, only the 2038 Horizon year scenario will include the bridge project.

C) Questions to Resolve

Some fundamental questions still need to be answered, such as-

1) What levels of congestion could be expected in the various crossings during each phase of constructions?

2) In general, what level of delay would be associated with the estimated congestion?

3) What system deficiencies need to be addressed when developing an operations plan for the constructions period?

It was determined early on that it would be unreasonable to have both the Claiborne and St. Claude crossings out of service and the same time. Therefore, on the assumption that a temporary four lane bride would be installed before the demolition of the St. Claude bridge, two basic construction year scenarios were developed.

1) Construction year without project.

2) Construction year network with Claiborne bridge out of service.

In order to address various aspects of these scenarios, such as peak and off operation, and derive certain selected data items as requested by the Corps, a total of approximately 10 model runs were necessary.

By comparing the results of the model runs for each of the construction phases to the runs for the no build alternative, the impacts of the construction on traffic in the corridor can be analyzed and the mitigation measures developed. Subsequently, the efficacy of such mitigation measures can also be tested using the model.

It should be noted, however, that the model is a regional and corridor level tool. Although it is useful in determining impacts within a gross order of magnitude, fine details concerning the operational deficiencies of the roadways is beyond the ability of this model. In fact the model assumes that all streets are essentially in good repair, have adequate design, and are significantly free of debris that would impede traffic. As was determined in the field surveys, such assumptions are not always warranted, the details of which will be taken up in the Operational Analysis.

[4] LaDOTD website, http://www.timedla.com/bridge/florida/faqs/

D) Future Year Scenarios

Constructing future year scenarios was somewhat problematic, There were essentially three questions that the Corps of Engineers wanted answered concerning the future scenarios.

1) Would there be any anticipated long term negative impacts associated with traffic crossing the canal or navigating in the near vicinity?

2) Would post project traffic demand at the St. Claude bridge be adequately addressed with a low level bridge, or would construction of a significantly more intrusive midlevel bridge be necessary despite neighborhood objection?

3) Would roadway improvements in the corridor, which the Corps of Engineers anticipated building to facilitate detours during construction have any lasting benefit to traffic in the corridor?

The problem with the model itself is that the improvements contemplated by the Corps of Engineers consist mostly of transportation systems management improvements. They propose better signage, improved signalization, roadway surface repair, etc. Unfortunately, the travel demand model assumes peak operational efficiency of the roadway as a given and has no capacity to evaluate such improvements.

Therefore the following scenarios were modeled:

1) A 2014 no build scenario

2) A 2013 build scenario that closes the N. Claiborne Bridge at the IHNC, with no improvements to the Florida Avenue Bridge.

3) A 2038 build scenario that includes improvements to the Florida Avenue Bridge.

Population projection methods and all other elements of the analysis were consistent with those described in the discussion of the Demand Analysis Methodology.

III. Construction Year Demand Analysis

A. <u>Construction Year Without Project</u>

The first scenario run for the year 2014 was the scenario depicting the Construction Year Without Project. Under this scenario, the current transportation system remains essentially unchanged.

Population and employment in the study area are generally forecast to grow between from approximately 26,000 persons in 2008 to about 31,500 persons in 2014. However, as is evident from **Table 3**, overall traffic growth crossing the IHNC stays flat, although volumes are redistributed differently between N. Claiborne and St. Claude. After a review of the datasets and other model outputs, RPC believes the model could be assigning more trips to areas east of the IHNC than is realistically justified given historic trends. As population and employment grow/ return to St. Bernard Parish and the Lower 9th Ward, trip making in those areas will tend to stay closer to home. As a result, the model results indicate almost no growth in the corridor volumes as a whole.

Table 3: Model Assignment Comparison by Forecast Year

Roadway	2008 Volume	2014 Volume	Change
N. Claiborne	20,200	23,021	0.14
St. Claude	12,240	9,818	-0.20
Florida Ave.	705	776	0.10
Total	**33,145**	**33,615**	**0.01**

While this could be a correct scenario, RPC believed it was not the probable scenario. Therefore, after looking at numerous data sets and outputs of the model, RPC conducted a second evaluation of trip generation and distribution at the screenline. The second evaluation was based on historic, observed traffic count data and other historic data in the area, ITE's Highway Capacity Manual, and ITE's Trip Generation Manual, 7th Edition, and professional judgment. The results are shown in **Table 4**, as follows.

Table 4: Manual Assignment Comparison by Forecast Year

Roadway	2008 Volume	2014 Volume	Change
N.Claiborne	20,200	26,090	29%
St. Claude	12,240	14,100	15%
Florida Ave.	705	910	29%
Total	**33,145**	**41,100**	**24%**

RPC expects the growth of traffic at the IHNC bridges to be roughly equivalent to overall population and employment growth in the study area. Further, RPC forecasts the majority of the growth to occur along the N. Claiborne corridor. RPC bases this forecast on redevelopment initiatives of both St. Bernard Parish[5] and the City of New Orleans[6].

Origin-Destinations

In the 1993 study, the Corps of Engineers project team requested additional information concerning expected origins and destinations of trips likely to suffer delays due to project construction. The Corps Economic Section hoped to use this data to help in deriving income levels of travelers and thus be able to more accurately assign costs to the delay associated with the project. In order to meet this updated objective, a critical link analysis was undertaken and a vehicle trip table was generated showing the origin and destination distribution of vehicle trips that cross the IHNC in the study area or divert around the bridge crossings via Paris Rd. A similar table was created for this effort using

5 Review of redevelopment scenarios per St. Bernard Parish Draft Master Plan, RPC 2008

6 Unified New Orleans Plan (UNOP), Planning Districts 7 (Marigny/Bywater) and 8 (Lower 9th Ward)

the results of the 2008 model runs and provided in **Table 5**. While the table is provided below, it should be viewed with the following caveat:

> Travel patterns in the New Orleans area post-Hurricane Katrina, particularly Origin and Destination (O/D) data, are most likely significantly different than they were pre-Hurricane Katrina, based upon known count data and anecdotal evidence. RPC last conducted O/D transportation surveys in 1999/2000. Neither RPC nor LaDOTD have conducted new O/D transportation surveys post-Hurricane Katrina in the New Orleans area. While these are deemed to be generally accurate, RPC makes this assertion based solely on model outputs.

The resulting zone to zone trip table proved to be somewhat cumbersome for use in the current effort. Therefore a compressed district to district trip table was created and appears in this report as Table 5.

Table 5

IHNC Selected Link Trip Tables by District Year 2008 Run- No Closures

District 1= CBD (CBD) **District 5= St. Bernard (STB)**
District 2= Orleans W of IHNC (OW) **District 6= E. Jeff (EJ)**
District 3= Orleans E of IHNC (OE) **District 7 = W. Jeff (WJ)**
District 4= Other Orleans (OO) **District 8= Ext. Sta. (XST)**

Compressed Distribution for all routes including Paris Road

District	*1 (CBD)*	*2 (OW)*	*3 (OE)*	*4 (OO)*	*5 (STB)*	*6 (EJ)*	*7 (WJ)*	*8 (XST)*	*Total*
1 (CBD)	0	0	721	0	1,504	0	0	744	**2,969**
2 (OW)	0	0	1,012	5	1,139	0	0	250	**2,406**
3 (OE)	719	1,011	0	2,111	0	661	230	374	**5,106**
4 (OO)	0	13	2,115	0	4,573	0	0	1,822	**8,523**
5 (STB)	1,505	1,137	0	4,279	0	1,824	621	816	**10,182**
6 (EJ)	0	0	4,579	0	1,520	0	0	765	**6,864**
7 (WJ)	0	0	0	0	618	0	0	514	**1,132**
8 (XST)	744	254	0	1,821	820	765	511	0	**4,915**
Total	**2,968**	**2,415**	**8,427**	**8,216**	**10,174**	**3,250**	**1,362**	**5,285**	**42,097**

As can be seen from the table, the largest single trip interchange is between St. Bernard and Orleans Parish, but more importantly, St. Bernard is involved in more trip interchanges than any other defined area. This result is not unexpected because St. Bernard represents the bulk of the population on the east of the IHNC that has available automobiles, but it also indicates that a substantial majority of the trips either originate or terminate some distance from the IHNC and would be susceptible to capture on a detour route accessing the Florida Corridor in St. Bernard Parish.

B) Construction Year with Claiborne Ave. Bridge out of Service

The anticipated location of the new lock is north of Claiborne Ave, adjacent to the present Galvez Street wharf. Due to the relocation, the section of the IHNC under the Claiborne bridge will be subject to increased water levels. Due to fluctuations in the river, water levels in the section could be as much as ten feet higher than levels currently found under the bridge. The bridge must therefore be modified in such a way as to allow the bridge to be raised an additional ten feet to provide required clearance for water traffic. The present plan is to raise the towers on the bridge an additional ten feet, a process that will take up to six weeks, during which time the bridge will be out of service to vehicular traffic.

Therefore, a set of runs similar to those previously described were run to determine the likely impacts of the loss of the N. Claiborne bridge. The results are shown in **Table 8**.

Table 8- Highway Link Comparison- 2013

Roadway	No Build	N. Claiborne Closure
N. Claiborne	26,090	-
St. Claude	14,100	28,430
Florida Ave.	910	7,740
Total	**41,100**	**36,170**

The closure of the N. Claiborne Avenue bridge is estimated to divert approximately 12% of total trip in the corridor to Paris Road. The remaining trips, nearly 80% are forecast to use the crossing at St. Claude, while the remaining 20% are forecast to use Florida Avenue. This represents a near *nine-fold* increase in vehicular traffic for that facility compared to current volumes. **Table 9** shows the comparison between traffic volumes under the N. Claiborne bridge closure scenario and pre-Katrina traffic volumes. As is evident from the table, forecast volumes on St. Claude under this scenario would be nearly identical to pre-Katrina observed volumes on the same roadway.

Table 9- Highway Link Comparison: Volume Comparison of N. Claiborne Bridge Closure vs. Pre-Katrina Traffic Volumes

Roadway	2004/2005 Observed	Claiborne Closure
N. Claiborne	37,103	-
St. Claude	28,653	28,430
Florida Ave.	8,906	7,740
Total	**74,662**	**36,170**

C. Delay Estimates for Construction Scenarios

Volume figures do not provide as meaningful a picture of impacts as do delay estimates. By observing all trip interchanges that involve canal crossings, including those diverted to Paris Road, and calculating the gross travel time for those trips, an idea can be gained of the positive or negative impacts of a given scenario in comparison to any other.

Speed and delay estimates were difficult to ascertain for this effort through conventional modeling practice. The model forecast near free flow conditions at all times of day. From a theoretical capacity standpoint, this is correct. **Table 10** shows the comparison between the 2008 Assignment and the 2014 No Build scenario.

Table 10: Volume to Capacity for 2008/2014 No Build Scenarios At INHC Crossing - Model Outputs

2008				2014 No Build			
Roadway	V/C	VHT*	Delay**	Roadway	V/C	VHT*	Delay**
Florida Ave.	0.052	10.7	0.915	Florida Ave.	0.061	12.5	1.07
N. Claiborne	0.2785	709.8	8.841	N. Claiborne	0.313	804.5	10.02
St, Claude	0.1435	256.5	6.416	St. Claude	0.206	368.4	9.215
Totals		977	16.17			1185.4	20.31

* Vehicle Hours Traveled (VHT) expressed in hours over a daily period

** Delay as expressed in hours, per day

As is evident from the table, the models expect little in the way of delay in the no build scenario. This makes intuitive sense given the trends in traffic since 1993, when the last report was undertaken.

Table 11 compares modeled delay between the No Build and Build scenarios for the implementation phase of the project. As is evident from the table, the model forecasts very modest delay across all scenarios.

Table 11- Travel Delay Summary - IHNC Crossings
Comparison of Build vs. No Build Scenarios

Scenario	Total Trips	Vehicle Hours Traveled	Net Delay (Hours)
No Build	41,100	962.8	0.996
Claiborne Bridge Closed	36,170	618.7	4.506

There are, however, caveats that need to be addressed concerning the logic of the model and the way it interprets the network. The model views the transportation system in statistical terms. It analyzes factors such as delay in terms of averages as applied to a smoothly flowing system of adequate design operating at peak efficiency. In reality the street system is operating far below its design expectations in many instances. Reasons for this include road surface conditions, poor traffic signal timing/phasing, school zones, etc. Importantly, the model does not account for the numerous bridge openings that occur at the industrial canal, nor the rail crossings along St. Claude Avenue in Orleans and St. Bernard Highway (LA 46) in St. Bernard Parish. These occur with enough frequency to substantially impact traffic flow throughout the day, but are notoriously difficult to forecast.

Additionally, as traffic volumes begin to reach or exceed the roadway capacity, traffic flow is more often interrupted by accidents, breakdowns, and counterproductive actions by inattentive drivers. These incidents, which are totally unaccounted for in the model, increase geometrically as the level of traffic rises. For these reasons, in addition to the demand analysis performed using the model, the RPC conducted an operational analysis to evaluate identifiable deficiencies in the St. Bernard/ Orleans corridor that might hinder traffic flow during the construction phase of the project.

IV. Operational Analysis

Travel Speed and Delay

As described earlier, the model assumes a peak operational efficiency of the roadway network that typically doesn't exist in reality. This is especially true of the LA 39/LA 46/ Florida Ave. corridor, which is subject to numerous traffic signals, train crossings, and bridge raisings randomly throughout all times of day. Therefore, RPC conducted travel time runs during AM and PM peak traffic hours for the corridors mentioned above to determine the true extent of travel time and delay along this corridor. Delay was ascertained using observed speed and the posted speed limit. These results are shown in **Table 12**. It should be noted that for the Florida Avenue Bridge, the speed limit adjacent to the bridge is 30 mph, but the speed on the bridge itself is 20 mph.

Table 12: Volume to Capacity for 2008
at IHNC Crossing - Observed Data for 2008

2008					
Roadway	Daily V/C*	Daily VHT**	Daily Delay†	Pk Hour Speed#	Level of Service##
Florida Ave.	0.052	29.6	4.87	25	A
N. Claiborne	0.2785	976.5	220.4	25.75	B
St. Claude	0.1435	609.8	244.4	16.2	D
Totals		**1615.9**	**469.67**		

* Vehicle to Capacity Ration estimated for an average day
** Vehicle Hours Traveled, estimated for an average day
† total of all delay experience over an average day, expressed in hours
# Observed average peak hour speed, worst case speed is presented regardless of direction or time of day
As desfine for Type III roadway per ITE Highway Capacity Manual, Table 11-1

Table 12 shows the travel speed, vehicle hours traveled, and net delay for vehicles passing through of the cordon at the IHNC between Florida Avenue, N. Claiborne Avenue, and St. Claude Ave. for the 2008 assignment. As is evident from this table, while capacity along the roadways is well within acceptable operating parameters, delay becomes more apparent.

Using observed data as a basis, RPC proceeded to undertake an analysis of the 2014 no build scenario, with the worst case volumes described in **Table 4.** The additional volumes were entered into the model and estimated travel speeds and delay were re-estimated. **Table 13** summarizes capacity and delay results.

Using similar methodologies, RPC re-accomplished the analysis for the N. Claiborne bridge scenario. The results of the analysis are found in **Table 14**.

Table 13: Volume to Capacity 2014 No Build Scenarios
at IHNC Crossing - 2014 No Build Interpolated Manual Results

2014 No Build						
Roadway	**Daily V/C***	**Pk Hour V/C^**	**Pk Hr. Speed#**	**Daily VHT****	**Daily Delay+**	**Level of Service##**
Florida Ave.	0.061	0.117	25.00	38.00	6.28	A
N. Claiborne	0.313	0.685	24.79	1308.4	514.00	B
St. Claude	0.206	0.335	15.55	698.8	339.62	D
Totals				2045.2	859.90	

* Vehicle to Capacity Ratio estimated for an average day
^ Vehicle to Capacity Ratio estimated for the peak hour of travel (worst case)
** Vehicle Hours Traveled, estimated for an average day
+ Sum total of all delay experience over an average day, expressed in hours
# Forecast average peak hour speed, worst case speed is presented regardless of dire
or time of day.
As defined for Type III Roadway per ITE Highway Capacity Manual, Table 11-1

Table 14: Volume to Capacity 2013 Build Scenario-
N. Claiborne Bridge Closure Scenario

2013 N. Claiborne Closure Scenario						
Roadway	Daily V/C*	Pk Hour V/C^	Pk Hr. Speed#	Daily VHT**	Daily Delay+	Level of Service##
Florida Ave.	0.452	0.557	18.00	449.60	126.49	C
N. Claiborne	-	-	-	-	-	
St. Claude	0.419	0.464	14.69	1555.3	823.63	D/E
Totals				2004.9	950.12	

* Vehicle to Capacity Ratio estimated for an average day
^ Vehicle to Capacity Ratio estimated for the peak hour of travel (worst case)
** Vehicle Hours Traveled, estimated for an average day
+ Sum total of all delay experience over an average day, expressed in hours
# Forecast average peak hour speed, worst case speed is presented regardless of dire
or time of day.
As defined for Type III Roadway per ITE Highway Capacity Manual, Table 11-1

As is evident from Table 14, speeds decrease and delay is forecast to rise significantly in adjacent corridors as the N. Claiborne Bridge is closed. Levels of Service along St. Claude are expected to deteriorate to a "low D." Still, these service levels are relatively common on roadways throughout the metropolitan area.

It should be noted that RPC ***did not*** *forecast operational scenarios that involved a potentially greater number of bridge openings for the N. Claiborne Bridge as a result of occasionally higher water levels per the relocation of the new lock. Travel time and delay could be impacted significantly by any increases in openings resultant of changes in, depending on the amount of marine traffic, of water levels on the Mississippi River, and on how the lock itself is operated. As such, and lacking definitive data on said information, RPC's analysis has assumed bridge openings similar to those experienced at this writing into the future.*

Field Survey and Observations of Roadway Conditions

In addition to travel time and delay analysis, RPC also conducted a field survey of roadway conditions in the corridor. All the approaches to the IHNC crossings, as well as significant intersecting streets, were driven and evaluated. The street network has changed very little since the 1993 study. However, the roadways, if not the moveable bridges, in the area were damaged by floodwaters and wind during the storm.

East of IHNC-

The most likely detour routes for diverting traffic east of the IHNC during construction are Tupelo Street and Caffin Avenue. Both streets are divided streets of concrete construction. The structural condition of these streets is unknown at this time. Nearly the entire lengths of these streets were inundated by the storms, some sections for nearly two months[7]. Both streets appeared to be constructed to arterial standards with Tupelo having the larger capacity, operating as a four lane roadway with an additional parking lane between Florida Avenue and St. Claude. Both streets could serve adequately as detour routes with only minor improvements dealing mostly with signage and signalization, assuming the structural integrity of the street has been maintained post-Hurricane Katrina. Given the length and duration of standing salt-water on the streets and the substantial amount of debris removal in the adjacent neighborhoods, these streets may need to be completely reconstructed prior to implementing a detour plan. The following is a more detailed set of observations and recommendations for operations for roadways east of the IHNC by scenario.

N. Claiborne Bridge Closure

Under this scenario, RPC forecasts significant diversion of trips from N. Claiborne to Florida Avenue. Florida Avenue in this scenario would increase in traffic from a current volume of over 900 vehicles per day to under 7,800 vehicles per day.

Florida Avenue-

East of the IHNC, Florida Avenue is an urban collector street which is concrete surfaced. It is a two lane undivided street including a narrow and ill-defined parking lane. The travel lane and the parking lane are contained within curbs, and roll-over curbs on the right side. The posted speed limit is 30 mph. The speed limit on the bridge itself is 20 mph. A large percentage of current traffic on the roadway is from heavy trucks, servicing either Southern Scrap off of Surekote Rd., or the continued hauling of hurricane related debris from within the study area.

The land uses around Florida Avenue at this location were primarily residential prior to Hurricane Katrina. The area at this time has seen little redevelopment since the storm, and is largely desolate.

The following items are recommended for consideration:

a. Temporary signalization of the intersection with Caffin Avenue

b. Temporary signalization of the intersection with Tupelo Street

d. Operate with no parking along the corridor. Re-stripe the parking lanes and designate them as emergency breakdown lanes only. The operation should be closely monitored initially to determine if any adjustments appear to be necessary in this regard.

e. Removal of said emergency breakdown lanes for a sufficient distance from the major intersecting streets of Caffin and Tupelo to allow for the introduction of right turn storage lanes for increased intersection capacity.

[7] As witnessed by RPC Principal Planner, Jeff Roesel- October 25, 2005

f. Improve lighting and increase the visibility of security patrols along the corridor. Even though the area is forecast to redevelop over the next several years, the area now is largely isolated and desolate, and would be considered by many drivers to be unsafe.

Caffin Avenue-

Caffin Avenue, between St. Claude Avenue and Florida Avenue, is an urban street that is concrete surfaced. It is a two-lane median divided residential street including a wide parking lane. The travel lane and the parking lane are contained within curbs, a barrier curb on the median side and a rollover curb on the right side. The posted speed limit is 30 mph. Caffin Avenue is controlled by traffic signals at St. Claude Avenue, N. Claiborne Avenue and N. Galvez Street, while all way stop signs control traffic at N. Roman, N. Rocheblave, and Urquhart Streets.

At this writing, there is little in the way of redevelopment occurring on Caffin Avenue north of N. Claiborne. However, there are three community facilities on the corner of N. Claiborne and Caffin that could be impacted by a detour plan-

a) EJ Morris Senior Center
b) AP Sanchez Community Center
c) Dr. Martin Luther King Jr. Elementary School

The following items are recommended for consideration:

a. The temporary signalization of the intersection with Florida Avenue.

b. Removing the traffic signal at N. Galvez and replacing with stop control at the N. Galvez Street approaches.

c. Removal of stop signs controlling Caffin Avenue to provide optimum progression to detouring traffic.

d. Operate with parking, initially, providing minimal disruption to the normal habits of the neighborhood. The parking lane design is such that there is ample clearance between the through traffic and parked vehicles, although restriping to clearly delineate the parking and travel lanes will be necessary. The operation should be closely monitored initially to determine if any adjustments appear to be necessary in this regard.

e. Removal of parking for a sufficient distance from the major intersecting streets of St. Claude, N. Claiborne, and Florida Avenue to allow for the introduction of right turn storage lanes for increased intersection capacity.

Tupelo Street-

Tupelo Street between St. Claude Avenue and Florida Avenue is a high design urban street which is concrete surfaced. It is a four lane median divided residential street consisting of two lanes in each direction and includes a wide parking lane, all within raised curbs. There is a barrier curb on the median side of the street and a roll-over curb along the right side of the street. The posted speed limit is 35 mph.

Tupelo Street is controlled by a traffic signal at N. Claiborne Avenue (LA 39) and at St. Claude Avenue (LA 46). All Way stop signs are in use on that portion of Tupelo Street between Claiborne and Florida Avenue at the intersection of N. Prieur Street, N. Galvez Street, N. Tonti Street, and N. Dorgenois Street.

The following items are recommended for consideration:

a. The removal of stop signs to provide the most efficient traffic movement, reducing delay and exhaust pollution.
b. Operate with parking, initially, providing minimal disruption of the normal habits of the neighborhood. The parking lane design is such that there is ample clearance between the through traffic and parked vehicles. The operation with parking should be closely monitored initially to determine adjustments that appear to be necessary.
c. Removal of parking for a sufficient distance from the major intersecting streets of St. Claude, N. Claiborne, and Florida Avenue to allow for the introduction of right turn storage lanes for increased intersection capacity.

Tupelo Street should be the primary route designated to carry the bulk of the detoured traffic between N. Claiborne and Florida Avenue. Because of its design features it should be designated as a truck route.

St. Bernard Parish

Additionally, access to Florida Avenue should be provided for St. Bernard Parish traffic in St. Bernard Parish, a substantial distance from the N.Claiborne bridge. This measure should do much toward reducing delay caused by turning movements from Tupelo Street and Caffin Avenue onto Florida Avenue.

A review of north/south roadways in the Arabi area of St. Bernard Parish indicates that most of the streets are residential in nature and, as such, are not designed to service a large amount of through traffic. Three possibilities were identified as potential candidates to be studied, exclusive of the aforementioned residential streets, regarding their feasibility for providing this necessary connecting service. These include:

a. Angela Ave.
b. Railroad ROW (Right-of-way) along the east side of Aycock St.
c. The undeveloped Meraux tract located immediately east of Arabi.

These will be reviewed as follows:

a. *Angela Avenue*

Angela Ave is a narrow two way street that is located along the eastern boundary of Jackson Barracks. The east side of the street is developed with modest residential homes between the Florida Ave alignment and Judge Perez Dr. (LA 39) and mixed single family residential homes and light commercial developments from that point to St. Bernard Highway (LA 46). Redevelopment of residential properties is evident along the street, particularly between N. Prieur Street to LA 39, and between LA 39 and LA 46.

Angela Avenue ends to the north at a St. Bernard Parish School Bus maintenance and storage facility, just south of the Florida Ave. alignment. There is some undeveloped property to the west of the Angela Avenue alignment that may provide an opportunity to effect a connection. This property is immediately north of Jackson Barracks.

Angela Avenue intersects with Patricia Street, which runs north of and parallel to Judge Perez Drive (LA 39) effectively all the way to LA 47, Paris Road.

In order to properly widen Angela Ave for use as a detour route some small amount of right of way may need to be secured from the Jackson Barracks side of the road.

At a minimum, Angela Avenue should be improved between its intersection with Patricia Street and the end of the road at Mustang Street. North of Mustang, Angela becomes the driveway of the St. Bernard School Board bus barn facility. It may be possible to lease this right of way from the School Board to use as a connection to Florida Avenue for the duration of the project. Florida Avenue would then need to be improved from a dirt road to a concrete roadway between Angela Avenue and DuBreuil Street in Orleans Parish, a distance of about two blocks. Otherwise, the improvement should take in that portion of Angela Avenue from its northern end to LA 46. St. Bernard Highway.

b. *Railroad Right-of-way east of Aycock Street.*

A portion of this ROW, between St. Bernard Hwy. and a point just south of Judge Perez Dr. has been abandoned. The remaining length, however, contains an operating track that circles to the west as it extends northward behind the St. Bernard Parish Bus facility near the extended alignment of the Florida Ave. corridor. Construction of a detour road on that alignment would entail the closure of a large drainage canal that parallels the railroad tracks.

c. *Meraux Track immediately east of Arabi*

This site may be the least objectionable to the local community because it would be away from any residential areas. One objection to those that may have the responsibility of funding the construction would be the additional cost associated with the project.

If this alternative is selected for serious consideration it may be possible to lease the land on a temporary basis. Another possibility is to seek participation by the property owner and/or the Parish in the construction expense of a north/south roadway that would be positioned to be part of future development. This is not a new idea. In a 1987 report to the Parish, just such a roadway was proposed to access the proposed St. Bernard Port expansion. And again in 2005, a roadway connecting the extended Florida Avenue (part of the TIMED Project for the corridor) to a new north-south alignment through the Meraux tract was vetted as part of the EA.

d. *Completion of the Florida Avenue Extension per LaDOTD's TIMED program.*

Per the nearly completed EA for the Florida Avenue Highrise Bridge Project, the extension of Florida Avenue directly to LA 47 (Paris Road) was the consensus choice for the eastern extension of Florida Avenue and the Florida Avenue Highrise Bridge project. LA 47 would become the north-south access roadway for the Florida Corridor. This project would allow the direct bypass of most localized congestion on the LA 39 corridor. This would most likely be the most expensive of the options presented, but may also be the option with the most long-term benefit in the corridor.

Irrespective of what alignment is selected to provide St. Bernard Parish traffic an access to Florida Avenue under this project, consideration should be given to the signalization of their intersections with Judge Perez Dr. and with St. Bernard Highway.

West of IHNC-

West of the IHNC, Florida Avenue is an urban collector street which is concrete surfaced between Elysian Fields Avenue and Desire Street, and with asphaltic concrete between Desire Street and the bridge. It has a provision for parking on the south side of the street, but the area is not utilized for parking. Florida at this location is a two lane undivided street with signalized intersections at Louisa Street, Piety Street, and Desire Street. The travel lanes are contained within curbs, barrier curbs on the north side and roll-over curbs on the south side. The posted speed limit is 30 mph. The speed limit on the bridge itself is 20 mph, and hours of operation are between 6:30 AM and 7:30 PM, daylight hours only.

The land uses around Florida Avenue at this location were primarily residential prior to Hurricane Katrina. The area at this time has seen significant redevelopment since the storm, with the exception of the two public housing projects that were nearby, Desire Housing Project and Florida Housing Project. West of the IHNC, Florida Avenue intersects with several north-south arterials that provide good access to other parts of the city and region.

The following items are recommended for consideration for Florida Avenue west of the IHNC:

a. Improve the westbound to southbound left turn at the Mazant/France Road overpass. Currently, a driver must U-Turn near Alvar Street, and that turn is geometrically inefficient to accommodate either a U-turn or a left turn. There is direct access to Florida Ave. from northbound Poland Avenue, however, westbound traffic on Florida Ave must pass the ramp connection to Poland Avenue south and make a u-turn to get to this ramp. Direct access is blocked by the France Road/Alvar St. Overpass Bridge Piers. A review of this location indicates that there is a possibility to construct a left turn lane in the median on the south side of the bridge piers. Guard rail should be installed to provide protection to the piers
b. Reconstruct or overlay Florida Avenue between Desire Street and the bridge. At this time the roadway is in severe disrepair.
c. Encourage the use of France Road as the north-south detour route to re-enter N. Claiborne, or N. Galvez Street. Florida Avenue provides direct access all the way to Broad Street in New Orleans. However, Florida is frequently blocked by train crossings originating from the Norfolk-Southern intermodal yard near Almonaster Avenue. As such, traffic should be routed either directly back to N. Claiborne or to N. Galvez, both of which are grade separated over said Norfolk-Southern facility.
d. Improve lighting and increase the visibility of security patrols along the corridor

France Road-

At Florida Avenue, France Road is a four lane major arterial roadway that spans the Florida Canal and provides access between Gentilly and the upper 9th Ward in New Orleans. It continues north as Alvar Street, providing access to Port activities and other interstate arterials such as I-10 and US 90.

The following items are recommended for consideration for France Road

a. Reconstruct the at grade approaches between France Road and Florida Avenue south of the Florida Avenue Canal. Roadways are in severe disrepair and do not drain properly.

b. Reconstruct the at grade approaches between France Road and Florida Avenue north of the Florida Avenue Canal. Roadways are in severe disrepair and do not drain properly.

c. Provide better wayfinding signage for westbound drivers on Florida Avenue wishing to go northbound on France Road.

d. Encourage the use of France Road as the north-south detour route to re-enter N. Claiborne, or N. Galvez Street. Florida Avenue provides direct access all the way to Broad Street (US 90) in New Orleans. However, Florida is frequently blocked by train crossings originating from the Norfolk-Southern intermodal yard near Almonaster Avenue. As such, traffic should be routed either directly back to N. Claiborne or to N. Galvez, both of which are grade separated over said Norfolk-Southern facility.

e. Improve lighting and increase the visibility of security patrols along the corridor

Louisa Street North

Louisa St., along with its one way pair, Piety St., provides access to and from the south to the east/west city street of Galvez St. and the primary arterials of N Claiborne Avenue, and St. Claude Avenue two four lane divided roadways. North of Florida Avenue, Louisa Street provides access to Almonaster Avenue and I-10 to the north with a four lane divided roadway configuration. There are All-Way stops located at Abundance Street and at Pleasure Street, and traffic signals are located at Higgins Boulevard, Almonaster Avenue, and the I-10 on ramps. There is also a school zone in place north of Benefit Street.

The following items are recommended for consideration:

a. Construction of a right turn lane on the Florida Avenue east approach to Louisa Street to improve intersection level of service.

b. Installation of a traffic signal at the above reference of a traffic signal at the above referenced intersection and interconnection with the traffic signal on the south side of the canal to improve the intersection operational efficiency.

c. Removal of stop signs to improve traffic progression.

d. Consider the possibility of utilizing dual turns in the design of the geometrics and traffic signalization.

Louisa Street, South

Although Louisa Street north of Florida Avenue is built to arterial standards, the same is not true for Louisa Street south of Florida Avenue. Louisa Street, south of Florida Avenue, operates as a one way pair with Piety located east of Louisa Street. This area is highly residential. With the exception of the first two blocks Louisa Street. is concrete surfaced and is operating as one lane for through traffic with parking on both sides of the street. The first two blocks are asphalt surfaced and in poor condition.

Currently the speed limit is 25 mph and truck traffic is prohibited southbound on Louisa. Traffic controls consist of an All Way stop condition at the N Miro intersection, a traffic signal at N. Galvez Street, and a stop sign control at N. Claiborne Avene. RPC does not recommend using Louisa Street south of Florida Avenue as a detour route.

Piety Street

Piety Street, the northbound portion of the one way pair with Louisa Street is in poor condition between St. Claude Ave and N Claiborne Avenue. The remaining section of the street to the north is generally in good condition but it, Like Louisa, has similar geometrics, is densely residential and has parking on both sides of the street. Left turns are prohibited from St. Claude Avenue. eastbound to Piety Street northbound. Piety Street is not recommended as a detour route.

North Galvez Street

N. Galvez Street, between Poland Avenue and Almonaster Ave., is a two lane two way minor arterial roadway. It is the only east/west roadway that has an overpass over the railroad tracks located between Montegut Street. and St. Ferdinand Street, other than N. Claiborne and St. Claude, and provides access to the north/south arterials of Franklin Avenue, Almonaster Avenue, and Elysian Fields Avenue for access into the CBD. It is generally in good condition.

The following item is recommended for consideration:

a. Wayfinding signage advising drivers coming from southbound Poland Avenue to N. Galvez Street should be installed.
b. At Almonster Avenue, N. Galvez becomes a couplet with N. Miro Street. N. Galvez runs one way eastbound, N. Miro runs one way westbound. Appropriate signage advising westbound drivers to use N. Miro Street should be installed.

Almonaster Avenue

Almonaster Ave is the first major north/south arterial west of Louisa St which is available for conducting detouring traffic to and from the CBD from Florida Ave. It intersects with Franklin Ave. to the south of Florida Ave in the southbound direction where the near side approach is controlled by stop signs. Yield signs control the median crossings.

The following item is recommended for consideration:

a. Signalization of the intersection to provide for efficient movement of the detour traffic to the CBD. Consideration should be given to the possible need for interconnection of this traffic signal with any other nearby traffic signals on Franklin Ave.

General Considerations

In addition to the described specific facility improvements, periodic enforcement patrols along the surface routes should be considered. The detour routes will take traffic through high crime areas. Motorists will need the assurance of a visible enforcement presence to be comfortable with the routes. In addition, the detour routes take traffic through some residential areas with their characteristic heavy parking, pedestrian crossings, and school zones. These conditions combined with the recommended removal of the all way stops on the detour routes makes enforcement of the posted speed limits imperative.

It should also be carefully noted that there are quite a few streets that are available for use in addition to those proposed for designation as detour routes. Although such personal decisions cannot be controlled, advertising designated routes prominently can reduce many negative impacts on the neighborhoods. Proper marketing, combined with enforcement patrols and improved traffic signalization should provide enough preferential treatment to keep the maximum number of motorists on the designated routes.

Signing should be used to specify truck routes and specifically prohibit trucks from some streets. These matters will have to be worked out with the appropriate local authorities. Consideration should also be given to a plan that will employ tow trucks available in the vicinity of the operating bridge crossings, on each side of the Canal, to reduce response time in the event of breakdowns or accidents that contribute disproportionately to the potential delay in any congested corridor. The periodic enforcement patrols previously mentioned will also contribute to early detection of such incidents.

V. FUTURE YEAR DEMAND ANALYSIS RESULTS

A. Projects in the Metropolitan Transportation Plan.

The current Metropolitan Transportation Plan (MTP) in effect for the New Orleans Urbanized area calls for the implementation of several projects that will likely have an impact on traffic crossing the IHNC. While some of these are not in the study area, it is anticipated that their implementation will increase overall throughput in crossing the IHNC, thus providing more reliable and attractive options for drivers traversing the corridor.

1. Florida Avenue Bridge

By far the most significant highway improvement planned for the study corridor is the construction of a Florida Avenue Bridge Project. The project is envisioned in three phases as follows:

Phase 1: Poland/ Alvar-Tupelo Mains Span Bridge Construction
Phase 2: Tupelo to Paris Rd. Construction
Phase 3: Poland/Alvar-Tupelo New Approaches Construction

As described earlier, the scope of the project has changed significantly since the 1993 study. The locally preferred alternative, as described earlier, is as follows-

> The bridge over the IHNC would be a four lane high rise bridge. Control of access would begin at Alvar Street on the west side of the IHNC, not I-10 or at I-610. Traffic would be routed north to Alvar Street, toward US 90 and I-10 near Louisa, not west toward Elysian Fields Ave. The proposed roadway would continue as a four lane section, returning to grade at Caffin Street, and proceeding as a four lane section to Tupelo Street. The roadway would taper to two lanes east of Tupelo Street, and proceed over the levee and into the marsh on the flood side of the 40 Arpent Levee as a two lane section. Thence, the roadway proceeds east uninterrupted to Paris Road, LA 47, where the project terminates.

Many of the alternatives mentioned in the 1993 study were vetted as part of the Environmental Assessment undertaken by LaDOTD in 2005. Because these alternatives were rejected in the EA, only the preferred alternative was evaluated for this effort. Since Hurricane Katrina, DOTD has determined that the preferred alternative for the Florida Avenue project cannot be funded in the near term, and that the project will eventually be “rescoped”. No timetable for said rescoping has been promulgated by DOTD at this writing. Therefore, horizon year evaluations of the roadway network (2038) were undertaken with the alignment promulgated by DOTD as part of the EA described above.

2. LA 46 Improvements: Overpass of Norfolk-Southern Railroad near Mehle Street

This project will significantly improve traffic flow in the LA 46 corridor via the construction of a bridge over the rail operation of the Norfolk Southern railroad near Mehle Street in St. Bernard Parish. Numerous, unscheduled rail crossings at this location effectively diminish the attractiveness of LA 46 as a through route in this area. This is an operational condition that the model does not account for, and is not considered a capacity improvement. This project is slated for implementation in Tier III of the MTP, or after FY 2022.

3. Almonaster Bridge Replacement over IHNC

As originally envisioned, the replacement of the Almonaster Bridge was to be undertaken as a four lane bridge, making Almonaster Boulevard a continuous four lane roadway from Franklin Avenue in the Ninth Ward of New Orleans to I-510 and Old Gentilly Road in far eastern New Orleans. Further, the existing bridge, now nearly 90 years old, suffers from chronic maintenance problems and has been closed to vehicular traffic since Hurricane Katrina. The Almonaster Bridge also serves as the crossing for the CSX railroad between their intermodal yard just east of the IHNC and the NO Public Belt system that serves the extensive port facilities and other Class I railroads in the region. While not within the project limits of the lock project, the replacement of the Almonaster Bridge improves access across the IHNC, and makes the crossing much more reliable, making it a more attractive commuting choice. This project is in Tier III of the MTP, to be implemented after FY 2022.

4. I-10 Widening of the High-Rise Bridge

This project calls for the widening of I-10 at the IHNC to an eight lane section, with breakdown lanes, between the Almonaster exit of I-10 and Crowder Boulevard in eastern New Orleans. While not within the project limits of the lock project, the widening of the I-10 High Rise Bridge improves access across the IHNC, and makes the crossing much more reliable, making it a more attractive commuting choice. This project is in Tier III of the MTP, to be implemented after FY 2022.

B. Model Results of Horizon Year Plan

The projects described above, plus others throughout the region were added to the model network. The horizon year for this effort was determined to be FY 2038. This determination was made as this year was the furthest extent of the socio-economic planning horizon data available to RPC in support of the Metropolitan Transportation Plan (MTP) for the New Orleans Urbanized area. In fact, the horizon year for the current plan is actually FY 2033, but RPC developed additional datasets to support an additional five years to the plan, if required. The model results for the horizon year are shown in **Table 16**:

Table 16: Manual Assignment Comparison by Forecast Year

Roadway	2008 Volume	2014 Volume	2038 Volume	Change 2008-2038
N. Claiborne	20,200	26,090	46,967	232.51%
St. Claude	12,240	14,100	28,921	236.28%
Florida Corridor	705	910	6,423	911.06%
Total	**33,145**	**41,100**	**82,311**	**248.34%**

As is evident from the table, the horizon year forecast calls for a significant growth in vehicular trips over the thirty year time frame of the plan. However, per the model outputs, delay and total vehicle hours traveled are well within acceptable operational parameters. Under these scenarios, the High Rise Florida Bridge project is implemented. . See **Table 17** below:

Table 17: Travel Delay Summary - IHNC Crossings
FY 2038 Horizon Year Analysis

Roadway	**Total Trips**	**Vehicle Hours Traveled**	**New Delay (Hours)**
-	-	-	-
N. Claiborne Bridge	46,967	1,487.4	57.596
St. Claude Bridge	28,921	761.4	24.597
Florida Corridor	6,423	145.1	3.294
Totals	**82,311**	**2,393.9**	**85.487**

In conclusion, horizon year projects in the long range Metropolitan Transportation Plan appear to effectively handle projected growth in population and employment in the corridor.

VI. SUMMARY OF FINDINGS AND RECOMMENDATIONS

Although it is clear from the above analyses that the Inner Harbor Navigation Canal Lock Replacement project will have significant impacts on the movement of vehicular traffic in the St. Bernard/New Orleans corridor, those impacts are not defeating to the project. In fact, the congestion and associated delays predicted by the model during the various bridge closures is considerably less than the reputation of the corridor would have suggested. The demand analysis suggests some of the factors that contributed to this result.

The corridor is presently operating within its design capacity. Peak hour speeds and delays are consistent with other parts of the region and fall within acceptable standards for urban arterial operations. The modest growth rate anticipated in the region should not produce much of a change in this condition, prior to the planned construction year. The corridor's bad reputation obviously springs from traffic delays and motorist frustration associated with the opening of the various bridges for vessels using the IHNC.

The improvements made in the Paris Road corridor assist in mitigating the congestion during bridge construction by allowing traffic to divert around the project without substantial increase in travel delay. Despite these factors, however, public reaction to the loss of access during construction is likely to be hostile, and both the demand and operational analyses lead to the conclusion that there will be hot spots of severe congestion to address. The following provides a brief overview of the anticipated problems and proposed mitigation measures.

A. Reconstruction of Claiborne Bridge.

Although a significant percentage of trips are diverted to Paris Road in this scenario, the Claiborne bridge reconstruction creates the potential for severe delays, particularly during peak hour, with significant traffic impacts on neighborhoods in the vicinity of St. Claude and Florida. Traffic on St. Claude, where signal progression is sub-standard, and between St. Claude and Claiborne, where the neighborhood is highly residential in character, will be particularly bad. This result will be partly due to motorist's tendency to stick to the familiar. Many people will 'try' St. Claude first, to see how bad it is, then attempt to divert north after they have entered, as well as caused, the queue from the bridge.

In order to mitigate these problems, construction of a new north/south roadway accessing Florida Avenue which is discussed below, should be considered. As with St. Claude, significant improvements should be considered with special attention to the portion of the north/south streets between St. Claude and Claiborne.

Installation of high profile signs to encourage early detour would also be beneficial, including the possible installation of an Intelligent Vehicle Highway System that advised motorists of conditions at a sufficiently early point to allow for easy detour. The most cost effective system might simply be a low watt radio station broadcasting travel information relayed from motorists with cell phones or enforcement personnel with radios.

Probably the most important mitigation effort during this phase of the project is an incident management program. Accidents and breakdowns typically account for up to forty percent of the delay on the nations highways. Every minute of incident related shutdown has a spin off of up to six additional minutes of delay due to congestion. The ability to quickly clear a minor incident and restore traffic flow will not only reduce delay, but the high profile presence of enforcement personnel, tow trucks, etc. will reassure motorists and reduce potentially negative reaction to the project.

It should be noted that RPC **did not** forecast operational scenarios that involved a potentially greater number of bridge openings for the N. Claiborne Bridge as a result of occasionally higher water levels per the relocation of the new lock. Travel time and delay could be impacted significantly by any increases in openings resultant of changes in the amount of marine traffic, of water levels on the Mississippi River, and on how the lock itself is operated. As such, and lacking definitive data on said information, RPC's analysis assumed bridge openings similar to those experienced at this writing.

B. Florida Avenue High Level Bridge

Given the amount of uncertainty of bridge openings at N. Claiborne and consistent with regional transportation objectives promulgated over many years, the RPC continues to recommend the construction of a high rise bridge over the IHNC south of the Gulf Intercoastal Waterway. The likeliest location given the enormous amount of planning and consensus building undertaken to date would be the Florida Avenue corridor.

This recommendation is based upon not only day to day vehicular mobility, but incident management and evacuation concerns as well[8]. RPC believes that a high rise bridge is necessary to address those concerns and has been part of regional planning efforts for many years. LaDOTD was provided a constitutional mandate in the late 1980's to undertake improvements specifically at the Florida Avenue crossing of the IHNC.

The timing for implementing the bridge improvement will be an issue of importance. The preferred alternative for the Florida Avenue High Rise Bridge will apparently not happen within the proposed timelines of the lock improvement project and/or will be significantly modified from its current concept. Given existing trends, RPC believes the corridor would not require the high rise bridge for vehicular traffic purposes in the near term if marine traffic operational provisions are enacted, such as strict adherence to curfews and operating the lock to minimize the amount and duration of bridge closures.

C. Construction of a New Roadway to Access Florida Avenue.

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[8] *New Florida Bridge over the Inner Harbor Navigational Canal (IHNC) Final Environmental Assessment*,LouisianaDepartment of Transportation and Development, May 2007, p.II-4

The extension of Florida Avenue into St. Bernard Parish and the construction of an additional connecting north/south roadway is recommended, although not incontrovertibly supported by the analyses in all cases.

During Claiborne bridge reconstruction, an additional route sufficiently deep into St. Bernard Parish to allow traffic to detour early would significantly reduce delay for trips in and out of the study area. This benefit is limited by two factors, however. Trips originating deep in St. Bernard are also those trips most likely to detour via Paris Road. Regardless of the improvements east of the canal, the west approaches, which are difficult to upgrade, will limit the efficiency of the Florida route. From an operational as well as a neighborhood impact standpoint, a new road through the Meraux tract may be the best choice, but the short duration of the Claiborne bridge work suggests that costs associated with the construction of a new roadway should be minimized. The next best option to a new roadway is the upgrade of Angela Street, with the construction of a connection to Florida Avenue.

D. Conclusion

Since the devastation of Hurricane Katrina, travel forecasting in the New Orleans area has been challenging. Given the fluid nature of the recovery effort, the region is only now beginning to realize some stability in population, employment and travel trends. This is especially true for the "upper" and "lower" Ninth Ward areas and St. Bernard Parish, the areas most affected by the storm.

Given the estimates and forecasts on population growth and subsequent travel demand for these areas, RPC concludes the following:

1) There should be sufficient capacity along the existing transportation arteries of Florida Avenue, N. Claiborne Avenue and St. Claude Avenue in the near term (2013-2014) to accommodate the respective short term closure of the N. Claiborne Bridge caused by construction. The crossing of the IHNC at St. Claude Avenue will remain open during reconstruction. A temporary four lane bridge will installed adjacent to the existing structure before the demolition and replacement of the old St. Claude Bridge.
2) The closure of the N. Claiborne bridge during construction will require a significant level of effort to accommodate detouring traffic.
3) RPC **did not** forecast operational scenarios that involved a potentially greater number of bridge openings for the N. Claiborne Bridge as a result of occasionally higher water levels per the relocation of the new lock. Travel time and delay could be impacted significantly by any increases in openings resultant of changes in the amount of marine traffic, of water levels on the Mississippi River, and on how the lock itself is operated. As such, and lacking definitive data on said information, RPC's analysis assumed bridge openings similar to those experienced at this writing.
4) Given the amount of uncertainty of bridge openings at N. Claiborne and consistent with regional transportation objectives promulgated over many years,

> the RPC continues to recommend the construction of a high rise bridge over the IHNC south of the Gulf Intercoastal Waterway. Given existing trends, RPC believes the corridor would not require the high rise bridge for vehicular traffic purposes in the near term (2013-2014) if marine traffic operational provisions are enacted, such as strict adherence to curfews and operating the lock to minimize the amount and duration of bridge closures.

The models used in this analysis are only highly speculative planning tools. Opportunities to collect and quantify real data are still important to understanding the potential problems related to the interruption of traffic across the Inner Harbor Navigation Canal. These data collection efforts are ongoing for N. Claiborne and St. Claude via RPC's Congestion Management System planning process. Information on detour traffic volumes, speeds and vehicle delay should be collected in detail to provide needed insight into possible project impacts, and as "calibration tools" for similar endeavors in the future.

Appendix A

File Descriptions

SPOT Satellite Image Captures: Satellite image was acquired September 2, 2005 and is displayed using visible and infrared band combinations.

Flood Mask: Derived from the SPOT image using spectral sampling of flooded areas for 3 regions to identify inundation.

FEMA LiDAR DEM mosaic: Constructed from the 5 meter LIDAR quarter quads available from the LSU Atlas web site. Note: street pattern, interior roads are low and store water, major roads are usually raised. (0 NAVD 88= -0.8 local MSL).

Water depth overlays: were determined using 3 AOIs for the separately impacted areas of New Orleans, New Orleans East and Arabi-St. Bernard. Water depths were determined from measured heights and high water marks in the area impacted by the 17th Street and London Avenue Canal breaks in New Orleans. New Orleans East and Arabi-St. Bernard flood depths were determined from the intersection (best fit) of the SPOT flood mask and the LiDAR DEMs valid for Sept 2, 2005 only.

Derived by DeWitt Braud and Rob Cunningham.
LSU Coastal Studies Institute

Appendix B

ITE's Highway Capacity Manual, and ITE's Trip Generation Manual, 7th Edition

Chapter 11: Arterial Streets

ARTERIAL STREETS

CONTENTS

I. INTRODUCTION

Urban and suburban arterials are signalized streets that primarily serve through traffic; secondarily, they provide access to abutting properties. For purposes of this manual, they are defined generally as facilities with lengths of at least 1 mi in downtown areas and at least 2 mi in other areas, with a signalized intersection spacing ranging from as little as 200 ft in downtown areas and 400 ft for interchanges and elsewhere to as long as 2 mi, and with turning movements at intersections that usually do not exceed 20 percent of total traffic volume. Roadside development along arterials can be intense, producing friction for through traffic that generally limits a driver's ability to travel at the desired speed.

In the hierarchy of urban highway transportation facilities, arterial streets are ranked between collector and downtown streets on one level and multilane suburban highways and rural roads on another. The difference in ranking is mainly determined by function and by the character and intensity of roadside development.

Collector streets provide both land access and traffic circulation service within residential, commercial, and industrial areas. Their access function is more important than that of arterials, and, unlike arterials, their operation is not always dominated by traffic signals.

Downtown streets are signalized facilities that often resemble arterials. They not only move through traffic but also provide access to local business by passenger cars, transit buses, and trucks. Turning movements at downtown intersections are often greater than 20 percent of total traffic because downtown flow involves a substantial amount of circulatory traffic.

Typical of downtown streets are numerous pedestrian conflicts and lane obstructions caused by stopping or standing taxicabs, buses, trucks, and parking vehicles that cause turbulence in the traffic flow. Downtown street function may change with the time of the day, and for this reason certain strategically located downtown streets are converted to arterial-type operation during peak traffic hours.

Multilane suburban highways and rural roads differ from arterials in the following features: (*a*) roadside development is not as intense, (*b*) density of traffic access points is not as high, and (*c*) signalized intersections are more than 2 mi apart. These conditions result in a smaller number of traffic conflicts, a smoother flow, and a dissipation of the platoon structure associated with arterial traffic.

Urban and suburban arterials include multilane divided arterials; multilane undivided arterials; two-lane, two-way arterials (one travel lane in each direction); and one-way arterials. Federal Highway Administration (FHWA) statistics from the early 1980s indicate the following distribution of urban and suburban arterial miles in urbanized areas of more than 100,000 people: multilane divided arterials constitute approximately 37 percent; multilane undivided arterials total 27 percent; two-lane, two-way arterials make up 33 percent; and one-way arterials represent the remaining 3 percent.

APPLICATIONS

The methodology contained in this chapter can be used by those concerned with the planning, design, and operation of arterials to evaluate the level of service of an existing or proposed facility. The methodology does not address arterial capacity, which is generally determined by the capacity of an arterial's signalized intersections, addressed in Chapter 9. In some cases, special midblock restrictions also limit capacity. In general, the user can best conduct an arterial capacity analysis by analyzing the capacity of the signalized intersections and other such points. It is important to note that capacity analysis of signalized intersections is necessary because when demand exceeds capacity at any point along the arterial, the arterial evaluation methodology based on average travel speed becomes inappropriate.

The methodology of this chapter is oriented toward the evaluation of an existing operations situation or a specific design proposal by a level-of-service (LOS) determination. The person doing such design or operations work will be able to investigate the effect of signal spacing, arterial classification (as defined here), and traffic flow on the arterial level of service. The methodology uses the signalized intersection procedure in Chapter 9 for the lane group containing the through traffic. By redefining lane arrangement (e.g., presence or absence of left-turn lanes, number of lanes), the analyst may influence the projected traffic flow in the through-traffic lane group and the capacity of the lane group. This redefinition, in turn, influences the arterial LOS determination by changing the intersection evaluation and possibly the arterial classification.

Those interested in planning applications may use the entire arterial methodology in a straightforward but somewhat simplified way by computing control delay using certain default values as outlined in Chapter 9. Knowledge of the intended signal timing and quality of progression, however, is vital. If it is lacking or cannot be estimated, no meaningful estimation of arterial level of service is possible, even on a planning level.

LOS criteria can be applied when travel time and delay runs are used to assess the impact of optimizing signal timing or other improvements to the arterial and periodically to evaluate the entire arterial system in an urban area. Arterial level of service also can be estimated by arterial traffic models, provided that

1. Input parameters such as running speeds and saturation flow rates are determined in a manner consistent with the procedures in this manual,
2. The delay calculated or estimated by the model is defined consistent with the definitions in this manual, and
3. The delay outputs from the model are based on the delay equations in this manual or have been validated with field data.

These applications of the methodology always require determination of the level of service and associated measures of effectiveness (i.e., travel time, delay, speed). In certain cases determination of LOS values is the final objective; in other cases LOS values associated with different alternatives are computed, and decisions are made using these values.

CHARACTERISTICS OF ARTERIAL FLOW

The operation of vehicles on arterial streets is influenced by three main factors: arterial environment, interaction among vehicles, and effect of traffic signals. These factors contribute to the capacity of an arterial street and the quality of service offered to its users. They constitute the basic elements of the methodology discussed in Section II of this chapter.

Arterial environment includes the geometric characteristics of the facility and adjacent land uses. Number of lanes and lane

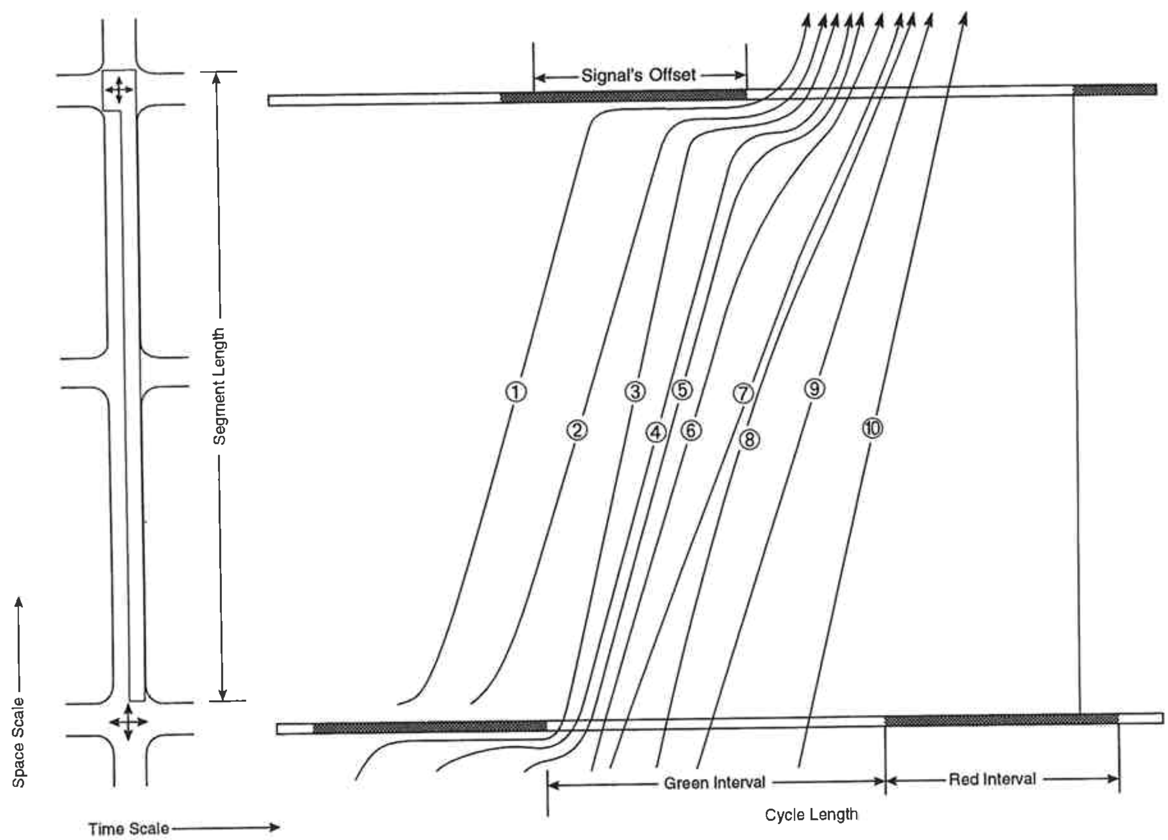


Figure 11-1. Typical time-space trajectories of vehicles on one-lane arterial segment.

width, type of median, driveway-access-point density, and spacing between signalized intersections are among the environmental factors, as are the existence of parking, level of pedestrian activity, speed limit, and population of the city.

The arterial environment affects a driver's notion of safe speed. Even if the effect of the other factors is negligible, the environment restricts a driver's desired speed, that is, the maximum speed at which a driver would like to travel under a given set of environmental conditions. The average desired speed of all drivers on an arterial segment or section is termed *free-flow* speed in this chapter.

Interaction among vehicles is determined by traffic density, the proportion of trucks and buses, and turning movements. This interaction affects the operation of vehicles at intersections and, to a lesser extent, between signals.

Seldom can a driver travel at the desired speed. Most of the time, the presence of other vehicles restricts the speed of a vehicle in motion because desired speeds differ among drivers or because downstream vehicles are accelerating from a stop and have not yet reached their drivers' desired speeds. Therefore, the average speed of a vehicle in motion over a certain length of road, or running speed, is usually lower than the desired speed of its driver because of the effect of vehicle interactions. Likewise, the *average running speed* of all vehicles on an arterial segment is usually lower than the free-flow speed of the segment.

Traffic signals force vehicles to stop and to remain stopped for a certain time, and then release them in platoons. The delays and speed changes caused by traffic signal operation considerably reduce the quality of traffic flow on arterial streets.

The average delay per vehicle depends mainly on the proportion of red time displayed to the arterial segment, the proportion of vehicles arriving on green (or the quality of traffic signal progression), and the traffic volume. The travel speed over an arterial segment (which includes time lost due to intersection effects, including stops and all associated control delay for the through movements) is generally lower than the corresponding running speed. Similarly, the *average travel speed* of all vehicles on the segment is lower than their average running speed unless no vehicles stop.

Figure 11-1 shows simplified time-space trajectories of representative vehicles along one lane of an arterial. Vehicles 1 and 2 turned onto the arterial from side streets, and the rest were discharged from the upstream signal. Vehicles 1, 2, and 3 arrived at the downstream signal approach during the red interval and had to stop. Vehicle 4 could have arrived at the stop line on green but had to stop because it was blocked by Vehicle 3, which was not yet in motion. Vehicles 5, 6, and 7 did not stop but had to reduce their speeds because they were still affected by the stoppages caused by the signal. Vehicle 8 was delayed because its driver's desired speed was higher than that of Vehicle 7's driver. Vehicles 9 and 10 traveled at their drivers' desired speeds. The travel speeds of Vehicles 1, 2, 3, and 4 were lower than their respective running speeds, which in turn were lower than the desired speeds of their drivers. The travel speeds of Vehicles 5, 6, 7, and 8 were equal to their corresponding running speeds, but lower than their drivers' desired speeds. Finally, for Vehicles 9 and 10, whose drivers were traveling at their desired speeds, the three types of speeds have the same value.

ARTERIAL LEVEL OF SERVICE

Arterial level of service is based on *average through-vehicle travel speed* for the segment, section, or entire arterial under consideration. This parameter is the basic measure of effectiveness for Chapter 11. The average travel speed is computed from the running time on the arterial segment or segments and the control delay for through movements at all intersections. To ensure that the arterial is of sufficient length so that average travel speed is a reasonable measure of effectiveness, the arterial's length generally should be at least 1 mi in downtown areas and at least 2 mi in other areas.

Arterial level of service is defined in terms of *average travel speed* of all through vehicles on the arterial. It is strongly influenced by the number of signals per mile and the average intersection control delay. On a given facility, such factors as inappropriate signal timing, poor progression, and increasing traffic flow can substantially degrade arterial level of service. Arterials with medium to high signal densities (more than two signalized intersections per mile) are even more susceptible to these factors, and poor arterial level of service will probably be observed even before substantial intersection problems occur.

The following general statements may be made regarding arterial level of service:

1. *LOS A* describes primarily free-flow operations at average travel speeds, usually about 90 percent of the free-flow speed for the arterial classification. Vehicles are seldom impeded in their ability to maneuver in the traffic stream. Delay at signalized intersections is minimal.
2. *LOS B* represents reasonably unimpeded operations at average travel speeds, usually about 70 percent of the free-flow speed for the arterial classification. The ability to maneuver in the traffic stream is only slightly restricted and delays are not bothersome.
3. *LOS C* represents stable operations; however, ability to maneuver and change lanes in midblock locations may be more restricted than in LOS B, and longer queues, adverse signal coordination, or both may contribute to lower average travel speeds of about 50 percent of the average free-flow speed for the arterial classification.
4. *LOS D* borders on a range in which small increases in flow may cause substantial increases in approach delay and hence decreases in arterial speed. LOS D may be due to adverse signal progression, inappropriate signal timing, high volumes, or some combination of these. Average travel speeds are about 40 percent of free-flow speed.
5. *LOS E* is characterized by significant delays and average travel speeds of one-third the free-flow speed or less. Such operations are caused by some combination of adverse progression, high signal density, high volumes, extensive delays at critical intersections, and inappropriate signal timing.
6. *LOS F* characterizes arterial flow at extremely low speeds, from less than one-third to one-quarter of the free-flow speed. Intersection congestion is likely at critical signalized locations, with long delays and extensive queuing.

Table 11-1 contains the arterial LOS definitions, which are based on average travel speed over the arterial segment being considered (up to and including the entire facility). It should be noted that if demand volume exceeds capacity at any point on the facility, average travel speed may not be a good measure of the arterial level of service. Thus, intersection demand-to-capacity ratios greater than 1.0 will probably result in an unacceptable level of service on the arterial. The arterial classification concept in Table 11-1 is defined as part of the methodology to follow.

II. METHODOLOGY AND PROCEDURES FOR APPLICATION

This methodology provides the framework for arterial evaluation. If field data are available, this framework can be used to determine the level of service of a given arterial without reference to running time and intersection delay estimates. Instead of treating field evaluation as a less desirable method than estimation, the transportation analyst should consider field data a better alternative for arriving at accurate arterial evaluations. If field data are unavailable, arterial traffic models are an alternative that can be used provided certain conditions are met. Input parameters such as running speeds and saturation flow rates must be determined in a manner consistent with the procedures in this manual, the delay calculated or estimated by the model must be defined consis-

TABLE 11-1. ARTERIAL LEVELS OF SERVICE

	ARTERIAL CLASSIFICATION			
	I	II	III	IV
Range of free-flow speeds	45 to 55	35 to 45	30 to 35	25 to 35
Typical free-flow speeds	50	40	33	30
LEVEL OF SERVICE	AVERAGE TRAVEL SPEED			
A	≥42	≥35	≥30	≥25
B	≥34	≥28	≥24	≥19
C	≥27	≥22	≥18	≥13
D	≥21	≥17	≥14	≥9
E	≥16	≥13	≥10	≥7
F	<16	<13	<10	<7

NOTE: Units are miles per hour.

tent with the definitions in this manual, and the delay outputs from the model must be based on the delay equations in this manual or must have been validated with field data.

Note that field data on free-flow speed will help in determining the arterial classification. In cases where the specific arterial does not yet exist, data on free-flow speed at comparable facilities are recommended as an estimate.

The procedure to determine arterial level of service involves seven steps, as shown in Figure 11-2:

1. Establish the location and length of arterial to be considered;
2. Determine the arterial classification using the classification scheme presented here in conjunction with the measurement of free-flow speed;
3. Divide the arterial for the purpose of the evaluation into sections that each contain one or more arterial segments;
4. Compute the arterial running time for each segment, and if any sections are larger than the individual segments, aggregate for the sections;
5. Tabulate the necessary information on each intersection, and compute the control delay for the arterial through movements at each intersection, taking into account intersection parameters for the through movement (C, the cycle length; g/C, the effective green ratio; X, the v/c ratio; and c, the capacity of the through lane group) and the quality of the signal progression;
6. Compute average travel speed (*a*) by section to prepare a speed profile and (*b*) over the entire facility; and
7. Assess the level of service by referring to Table 11-1.

The methodology should be applied twice on two-way arterials if the level of service is to be assessed in each direction.

Steps 4 through 6 can be superseded by field data measurements of the average travel speed by doing travel time and delay studies along the arterial. Appendix I presents the field data collection procedures needed to provide the necessary data. Steps 4 through 6 can also be superseded by arterial traffic model estimates of average travel speeds and control delays for the arterial through movement provided the estimates are either calculated on the basis of procedures in this manual or validated with field data.

Each of the steps is addressed in the remainder of this section.

STEP 1—ESTABLISH ARTERIAL TO BE CONSIDERED

At the start of the analysis, it is useful to define the location and length of the arterial to be considered and identify all relevant physical, signal, and traffic data.

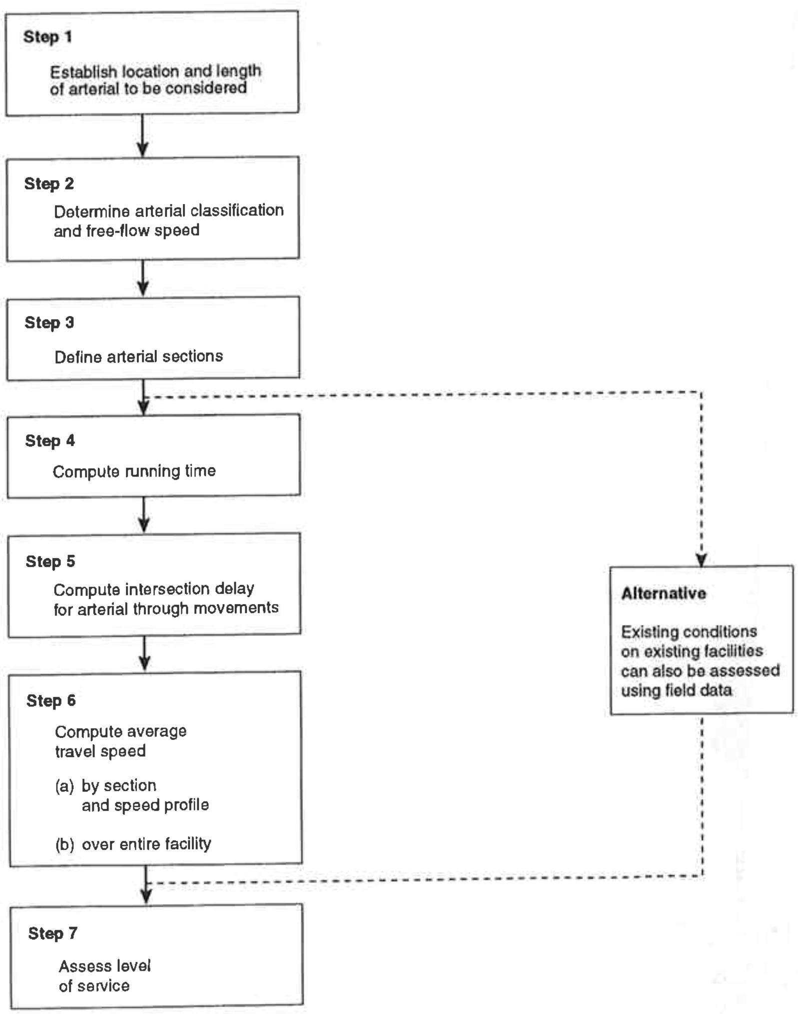


Figure 11-2. Arterial LOS method.

The arterial being analyzed should be at least 1 mi long in downtown areas and at least 2 mi long in other areas. If it is not, the analyst should consider whether to add more sections.

STEP 2—DETERMINE ARTERIAL CLASS AND FREE-FLOW SPEED

Four arterial classifications are defined in this chapter on the basis of arterial function and design. Each classification includes a range of free-flow speeds. In some cases, measurement of free-flow speed is a valuable aid in determining proper arterial classification because of ambiguities in the classification categories.

Both free-flow speed and actual average travel speed can be obtained by arterial travel time studies. Thus, the application of this chapter can be based entirely on field measurements. Appendix I presents the necessary field procedures.

Free-flow speed is the average speed of drivers over the portions of arterial segments that are not close to signalized intersections, as observed during very low traffic volume conditions while drivers are not constrained by other vehicles or by traffic signals. Average free-flow speed should approximate drivers' desired speeds for the facility and its use. Free-flow speeds may be measured by test cars or by spot speed observations away from intersections.

In all cases, the arterial should be classified first by functional category and then by design category. The functional category is either principal or minor arterial.

A *principal arterial* serves major through movements between important centers of activities in a metropolitan area and a substantial portion of trips entering and leaving the area. It also connects freeways with major traffic generators. In small cities (less than 50,000), its importance is derived from the service provided to traffic passing through the urban area. Service to abutting land is subordinate to the function of moving through traffic.

A *minor arterial* is a facility that connects and augments the principal arterial system. Although its main function is still traffic mobility, it performs this function at a somewhat lower level and places more emphasis on land access than does a principal arterial. A system of minor arterials serves trips of moderate length and distributes travel to geographical areas smaller than those served by a principal arterial.

Within the functional classification, the arterial is further classified by its design category. Figure 11-3 shows some typical examples of the four design categories.

Typical high speed design represents an arterial with a low driveway-access-point density, separate left-turn lanes, and no parking. It may be a multilane divided or undivided arterial or a two-lane facility with shoulders. Signals are infrequent and spaced at long distances (no more than two signals per mile). Roadside development is low density, and the speed limits are typically 45 to 55 mph. This design category includes many arterials in nonurban settings.

Typical suburban design represents an arterial with a low driveway-access-point density, separate left-turn lanes, and no parking. It may be a multilane divided or undivided arterial or a two-lane facility with shoulders. Signals are spaced for good progressive movement (one to five signals per mile or signals spaced at even greater distances). Roadside development is of low to medium density, and the speed limits are usually 40 to 45 mph.

Typical intermediate design represents an arterial with a moderate driveway-access-point density. It may be multilane divided, undivided one way, or two lane. It may have some separate or continuous left-turn lanes and some portions with parking permitted. It has a higher density of roadside development than the typical suburban design, and it usually has 4 to 10 signals per mile. Speed limits are normally 30 to 40 mph.

Typical urban design represents an arterial with a high driveway-access-point density. It frequently is an undivided one-way or two-way facility with two or more lanes. Parking is usually permitted. Generally, there are few separate left-turn lanes, and some pedestrian interference is present. The arterial commonly has 6 to 12 signals per mile. Roadside development is densely commercial. Speed limits range from 25 to 35 mph.

In addition to these definitions, Table 11-2 should be used as an aid in the determination of functional and design categories. Once the functional and design categories have been established, the arterial classification may be established by referring to Table 11-3.

As a practical matter, there are sometimes ambiguities in determining the proper categories. Measurement or estimation of free-flow speed is a great aid in this determination because each arterial classification has a characteristic range of free-flow speeds, as shown in Table 11-1. Free-flow speed alone cannot be used to determine arterial classification, but it can be used as an effective check in the arterial classification scheme. Information on arterial classification is used in Steps 4 and 7 of the methodology.

STEP 3—DIVIDE ARTERIAL INTO SECTIONS

The basic unit of the arterial is the *segment*, which is the one-directional distance from one signalized intersection to the next. Figure 11-4 illustrates the segment concept on one- and two-way arterials.

If two or more consecutive segments are comparable in arterial classification, segment length, speed limit, and general land use and activity, the analyst may wish to aggregate these into a *section*. If the segments are aggregated into a section, all results would then focus on the section rather than on the smaller component. When a section is defined, the average segment length may be used in finding the running time per mile in the next step.

STEP 4—COMPUTE ARTERIAL RUNNING TIME

Two principal components make up the total time that a vehicle spends in a section and on the arterial: arterial running time and control delay for the through movement. This step is focused on computing the first of these terms so that it may be used in the denominator of the following equation:

$$ART\ SPD = \frac{3{,}600 * (\text{length})}{[(\text{running time/mile}) * (\text{length}) + (\Sigma \text{ inters. control delay})]} \tag{11-1}$$

where

ART SPD = arterial or section average travel speed (mph),
length = arterial or section length (mi),
running time/mile = total of the running time per mile on all segments in arterial or section (sec), and
inters. control delay = summation of control delays for through movements at all signalized intersections in arterial or section (sec).

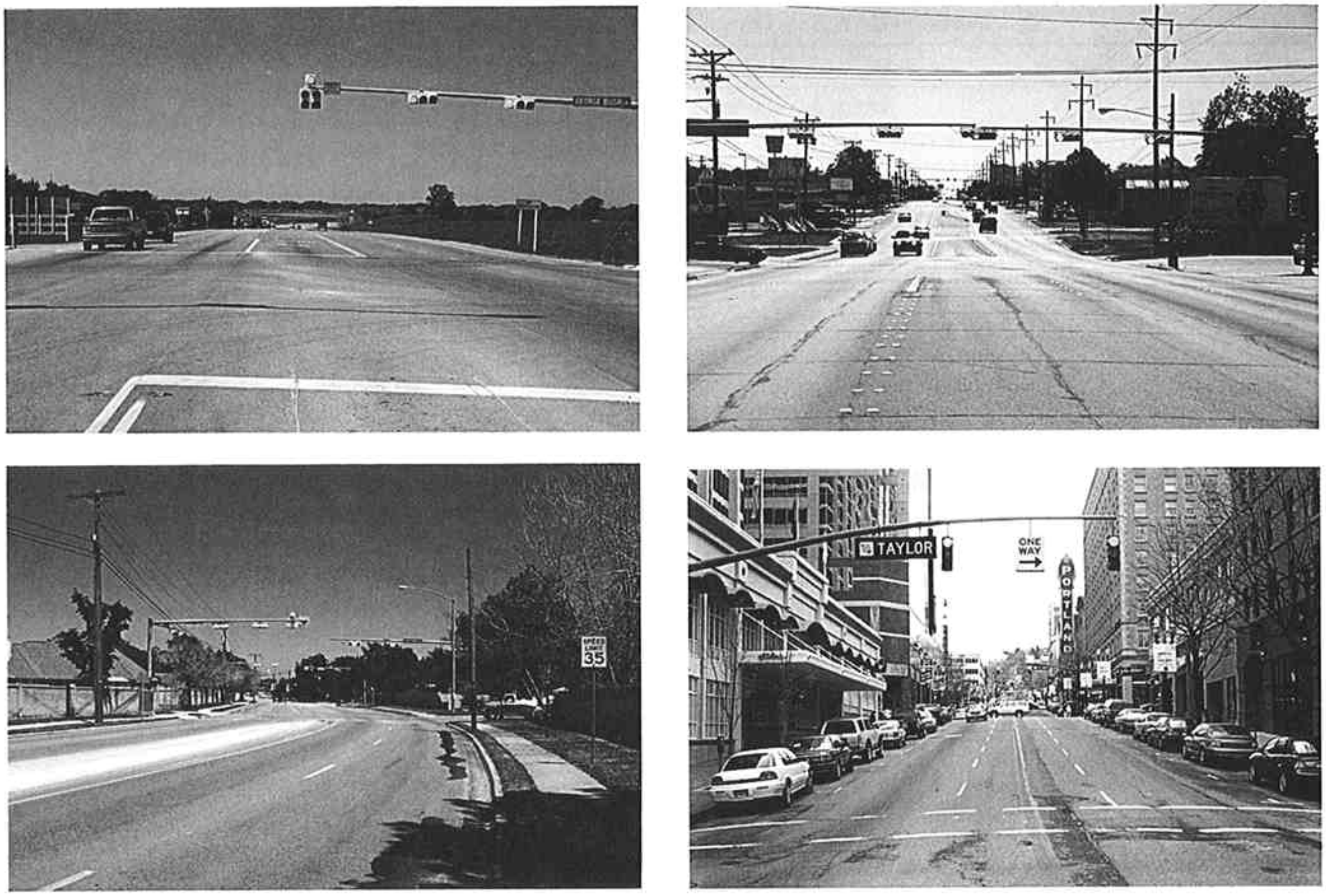


Figure 11-3. Design categories: top left, typical high speed design; top right, typical suburban design; bottom left, typical intermediate design; bottom right, typical urban design.

The 3,600 sec/hr is a conversion factor to compute *ART SPD* in miles per hour.

In special cases, unusual midblock delays may be caused by regular vehicle stops at pedestrian crosswalks. Other such delays may be caused by bus stops or driveway interference. Such delays may be added to the intersection control delay in the denominator of Equation 11-1.

To compute the running time in a segment, the analyst must know

- Arterial classification,
- Segment or section length in miles, and
- Free-flow speed in miles per hour.

The segment running time may then be found by using Table 11-4 (based on research conducted by FHWA and others).

If a section has been defined that encompasses several segments, the average segment length should be used in finding the running time per mile from Table 11-4. Running time per mile is then multiplied by the section length.

In each arterial classification, a number of factors can influence actual free-flow speed and running time per mile. Table 11-4 shows the effect of length directly. In addition, running time per mile may be influenced by such factors as the presence of parking, opportunities for side friction, and local development and street use. In this chapter, these factors are assumed to influence the free-flow speed, so observation of free-flow speed includes the effect of these factors. Once free-flow speed is estimated, the running speed used also reflects the effect of these factors; Table 11-4 contains higher running times for the lower free-flow speeds within each classification.

If it is not possible to observe the free-flow speed on the actual facility or on comparable existing facilities, a note to Table 11-4 gives default values to use; however, a local history of free-flow speeds on different arterial types should be available.

Example: What is the running time on a segment that is 0.20 mi long and has a free-flow speed of 40 mph? The arterial is a principal arterial, suburban design.

Solution: Note that on the basis of Tables 11-2 and 11-3, the arterial falls in Classification II. Table 11-4 estimates the running time per mile at 115 sec, so that the segment running time is 115 × 0.20 = 23 sec.

TABLE 11-2. AID IN ESTABLISHING ARTERIAL CLASSIFICATION

CRITERION	FUNCTIONAL CATEGORY: PRINCIPAL ARTERIALS		FUNCTIONAL CATEGORY: MINOR ARTERIALS	
Mobility function	Very important		Important	
Access function	Generally minor		Substantial	
Points connected	Freeways, important activity centers, major traffic generators		Principal arterials	
Predominant trips served	Relatively long trips between points connected, through trips entering, leaving, going through city		Trips of moderate lengths within relatively small geographical areas	
	DESIGN CATEGORY			
CRITERION	HIGH SPEED DESIGN	SUBURBAN DESIGN	INTERMEDIATE DESIGN	URBAN DESIGN
Driveway access density	Low density	Low density	Moderate density	High density
Cross section	Multilane divided or undivided	Multilane divided; multilane undivided; two lane with shoulders	Multilane divided; multilane undivided; one way; two lane	Undivided one way; two way, two or more lanes
Parking	No	No	Some	Usually
Separate left-turn lanes	Yes	Yes	Usually	Some
Signal per mile	1 to 2	1 to 5	4 to 10	6 to 12
Speed limits	45 to 55 mph	40 to 45 mph	30 to 40 mph	25 to 35 mph
Pedestrian interference	None	Little	Some	Usually
Roadside development	Low density	Low to medium density	Medium/moderate density	High density

TABLE 11-3. ARTERIAL CLASSIFICATION ACCORDING TO FUNCTIONAL AND DESIGN CATEGORIES

DESIGN CATEGORY	FUNCTIONAL CATEGORY: PRINCIPAL ARTERIAL	FUNCTIONAL CATEGORY: MINOR ARTERIAL
High speed design and control	I	Not applicable
Typical suburban design and control	II	II
Intermediate design	II	III or IV
Typical urban design	III or IV	IV

Example: Consider the foregoing case, but with an average 30 sec midblock delay due to a pedestrian crosswalk. How should the analysis be done?

Solution: The analysis should be done as above, but the 30 sec should be added to the third term in the denominator of Equation 11-1 when the computations are done.

Example: Three consecutive segments on a north-south two-lane two-way facility (i.e., one lane in each direction) are 0.15, 0.17, and 0.13 mi long, respectively, all with a free-flow speed of 30 mph. The arterial is a Classification IV principal arterial. What is the northbound running time on the section?

Solution: Note that it is reasonable to define a single section if all necessary conditions are met, including all lengths being within 20 percent of the average segment (see Step 3 of the methodology). From Table 11-4, the running time per mile for a Classification IV arterial with 30 mph free-flow speed is 150 sec for a 0.15 mi segment (the average of the three segment lengths in this section). The actual running time is computed as follows:

$$(150) \times (0.15 + 0.17 + 0.13) = 67.5 \text{ sec}$$

Example: What is the southbound running time for the same section?

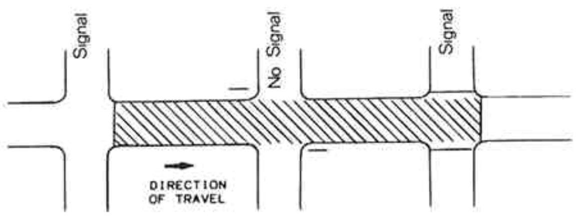
Solution: The southbound running time is found in the same way, and the answer is therefore the same. This example is a useful reminder that frequently two-way arterials should be evaluated in each direction; generally the answers will be different because of the influence of intersection delay (the effect of different signal progression quality in the two directions will contribute to this difference).

As noted in Table 11-4, it is logical that segment running time should depend on traffic flow rate; however, arterial research conducted for FHWA in the early 1980s did not establish a quantitative relation for such a dependence. It logically exists, but is not strong, certainly not as strong as the effect of segment length on segment running time. Nor is it as strong as the substantial variation of intersection control delay with traffic flow rate.

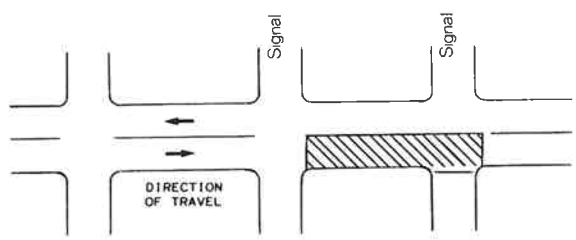
As a practical matter, computation of arterial travel speed for different traffic flow rates is dominated by changes in control delay for the arterial through movements, whether or not the segment running time volume dependence is clearly identified. Thus, the absence of such an explicit factor does not affect the practical result, namely, the computation of arterial travel speed.

STEP 5—TABULATE INTERSECTION INFORMATION AND COMPUTE DELAY

To compute arterial or section speed, the analyst needs to determine individual intersection delays. Because the arterial function is to serve through traffic, the lane group that includes the through traffic is used to characterize the arterial.



(a) Segment on a One-Way Arterial



(b) Segment on a Two-Way Arterial

Figure 11-4. Types of segments.

The correct delay to use in the arterial evaluation is the intersection control delay for the through movement. In general, the analyst has the necessary information because the intersections are evaluated individually as part of the overall analysis. Geometric and traffic delay have already been taken into account in the segment running times in Table 11-4.

The equations for computing average control delay per vehicle are

$$d = d_1 \times PF + d_2 + d_3 \quad (11\text{-}2)$$

$$d_1 = \frac{0.5C\,[1 - (g/C)]^2}{1 - (g/C)\,[\min(X, 1.0)]} \quad (11\text{-}3)$$

$$d_2 = 900T\,[(X - 1) + \sqrt{(X - 1)^2 + 8kIX/Tc}] \quad (11\text{-}4)$$

where

d = control delay (sec/veh),
d_1 = uniform delay (sec/veh),
d_2 = incremental delay (sec/veh),
d_3 = residual demand delay (sec/veh) (see Appendix 9-VI),
PF = uniform delay adjustment for quality of progression,
c = capacity of lane group (veh/hr),
X = v/c ratio for lane group with v representing demand flow rate,
C = cycle length (sec),
g = effective green time for lane group (sec),
T = duration of the analysis period (hr),
k = incremental delay adjustment for actuated control, and
I = incremental delay adjustment for filtering and metering by upstream signals.

TABLE 11-4. SEGMENT RUNNING TIME PER MILE

ARTERIAL CLASSIFICATION	I			II			III		IV		
FREE-FLOW SPEED (MPH)	55	50	45	45	40	35	35	30	35	30	25
SEGMENT LENGTH (MI)	RUNNING TIME PER MILE (SEC/MI)										
0.05										227	265
0.10							145	155	165	180	220
0.15							135	141	140	150	180
0.20				109	115	125	128	134	130	140	165
0.25	97	100	104	104	110	119	120	127	122	132	153
0.30	92	95	99	99	102	110					
0.40	82	86	94	94	96	105					
0.50	73	78	88	88	93	103					
1.00	65	72	80	80	90	103					

NOTES:

1. It is best to have an estimate of free-flow speed based on field observations of the facility or comparable facilities. If an estimate is lacking, however, the analyst can use the table by assuming the following default values:

Classification	*Free-Flow Speed (mph)*
I	50
II	40
III	33
IV	30

2. For very long segment lengths on Classification I or II arterials (1 mi or longer), free-flow speeds may be used to compute running time per mile. These times are shown in the entries for a 1.0-mile segment length.
3. If a Classification I or II arterial has a segment length less than 0.20 mi, the user should (*a*) reevaluate the classification and (*b*) if reevaluation confirms the classification, use the values for 0.20 mi.
4. Likewise, Classification III or IV arterials with segment lengths longer than 0.25 mi should first be reevaluated (i.e., the classification should be confirmed). If necessary, values for a segment of this length can be extrapolated.
5. Although this table does not show segment running time dependent on traffic flow rate, such a dependence is logical; however, the dependence of intersection delay on traffic flow rate is much stronger and thus dominates in the computation of arterial travel speed.

The components of these equations are discussed in the sections that follow.

Uniform Delay (d_1)

Equation 11-3 estimates control delay assuming perfectly uniform arrivals and stable flow. It is based on the first term of Webster's delay formulation and is widely accepted as an accurate depiction of delay for the idealized case of uniform arrivals. Note that values of X higher than 1.0 are not used in the computation of d_1.

Incremental Delay (d_2)

Equation 11-4 estimates the incremental delay due to nonuniform arrivals and individual cycle failures (random delay) as well as that caused by sustained periods of oversaturation (oversaturation delay). It is sensitive to the degree of saturation of the lane group (X), the duration of the analysis period of interest (T), the capacity of the lane group (c), the type of signal control, as reflected by the control type parameter (k), and the upstream filtering/metering parameter (I). The incremental delay term is valid for all degrees of saturation (X), including highly oversaturated lane groups. The equation assumes that no unmet demand causes residual queues at the start of the analysis period (T).

Residual Demand Delay (d_3)

When demand from a previous time period causes a residual queue to occur at the start of the analysis period (T), additional delay is experienced by the vehicles arriving in the analysis period because the residual queues must first clear the intersection. A procedure for determining residual demand delay is described in detail in Appendix 9-VI. This procedure is also used to analyze delay over multiple time periods, each having a duration (T), in which a residual demand may be carried from one time period to the next.

Progression Adjustment Factor (PF)

One of the most critical traffic characteristics that must be quantified to complete an operational analysis of an arterial or a signalized intersection is the quality of the progression. The parameter that best describes this characteristic is the arrival type for each lane group. This parameter is a general categorization that represents the quality of progression in an approximate manner. Six arrival types are defined for the dominant arrival flow as follows:

- Arrival Type 1: Dense platoon containing more than 80 percent of the lane group volume and arriving at the start of the red phase. This arrival type is representative of arterials that experience very poor progression quality as a result of conditions such as lack of overall network signal optimization.
- Arrival Type 2: Moderately dense platoon arriving in the middle of the red phase or dispersed platoon containing 40 to 80 percent of the lane group volume arriving throughout the red phase. This arrival type is representative of unfavorable progression quality on a two-way arterial.
- Arrival Type 3: Random arrivals in which the main platoon contains less than 40 percent of the lane group volume. This arrival type is representative of operations characterized by highly dispersed platoons at isolated and noninterconnected signalized intersections. It may also be used to represent coordinated operation in which the benefits of progression are minimal.
- Arrival Type 4: Moderately dense platoon arriving in the middle of the green phase or dispersed platoon containing 40 to 80 percent of the lane group volume arriving throughout the green phase. This arrival type is representative of favorable progression quality on a two-way arterial.
- Arrival Type 5: Dense to moderately dense platoon containing more than 80 percent of the approach volume and arriving at the start of the green phase. This arrival type is representative of highly favorable progression quality, which may occur on routes that have a low to moderate number of side street entries and receive high priority in the signal timing plan design.
- Arrival Type 6: This arrival type is reserved for exceptional progression quality on routes with nearly ideal progression characteristics. This arrival type is representative of very dense platoons progressing over a number of closely spaced intersections with minimal or negligible side street entries.

Arrival type is best observed in the field, but can be approximated by examining time-space diagrams for the arterial or street in question. The arrival type should be determined as accurately as possible because it has a significant impact on delay estimates and LOS determination. Although no definitive parameters precisely quantify arrival type, the following ratio is a useful value:

$$R_p = P \times \left(\frac{C}{g}\right) \qquad (11\text{-}5)$$

where

R_p = platoon ratio,
P = proportion of all vehicles in movement arriving during green phase,
C = the cycle length (sec), and
g = effective green time for movement (sec).

P may be estimated or observed in the field, while C and g are computed from the signal timing. When P is estimated, note that its value may not exceed 1.0. As shown in Table 11-5, the approximate ranges of R_p are related to arrival type, and default values are suggested for use in subsequent computations.

Good signal progression results in the arrival of a high proportion of vehicles on the green. Poor signal progression results in the arrival of a low percentage of vehicles on the green. The progression adjustment factor, PF, applies to all coordinated lane groups, including both pretimed control and nonactuated lane groups in semiactuated arterial control systems. Progression primarily affects uniform delay, and for this reason, the adjustment is applied only to d_1. The value of PF may be determined by

$$PF = \frac{(1 - P)f_p}{(1 - g/C)} \qquad (11\text{-}6)$$

where g/C = effective green time ratio, and f_p = supplemental adjustment factor for platoon arriving during the green.

The default values for f_p are 0.93 for Arrival Type 2, 1.15 for Arrival Type 4, and 1.0 for all other arrival types.

As mentioned previously, the value of P may be measured in the field or estimated from the arrival type. If field measurements are carried out, P should be determined as the proportion of vehicles in the cycle that arrives at the stop line or joins the queue

TABLE 11-5. RELATIONSHIP BETWEEN ARRIVAL TYPE AND PLATOON RATIO (R_p)

ARRIVAL TYPE	RANGE OF PLATOON RATIO (R_p)	DEFAULT VALUE (R_p)	PROGRESSION QUALITY
1	≤0.50	0.333	Very poor
2	>0.50 and ≤0.85	0.667	Unfavorable
3	>0.85 and ≤1.15	1.000	Random arrivals
4	>1.15 and ≤1.50	1.333	Favorable
5	>1.50 and ≤2.00	1.667	Highly favorable
6	>2.00	2.000	Exceptional

NOTE: $R_p = P \times (C/g)$.

TABLE 11-6. UNIFORM DELAY (d_1) PROGRESSION ADJUSTMENT FACTOR (PF)

GREEN RATIO	ARRIVAL TYPE (AT)					
(g/C)	AT 1	AT 2	AT 3	AT 4	AT 5	AT 6
0.20	1.167	1.007	1.000	1.000	0.833	0.750
0.30	1.286	1.063	1.000	0.986	0.714	0.571
0.40	1.445	1.136	1.000	0.895	0.555	0.333
0.50	1.667	1.240	1.000	0.767	0.333	0.000
0.60	2.001	1.395	1.000	0.576	0.000	0.000
0.70	2.556	1.653	1.000	0.256	0.000	0.000
Default, f_p	1.00	0.93	1.00	1.15	1.00	1.00
Default, R_p	0.333	0.667	1.000	1.333	1.667	2.000

NOTES: 1. $PF = (1 - P)f_p/(1 - g/C)$.
2. Tabulation is based on default values of f_p and R_p.
3. $P = R_p * g/C$ (may not exceed 1.0).
4. PF may not exceed 1.0 for AT 3 through AT 6.

(stationary or moving) while the green phase is displayed. The value of PF may be computed from measured values of P using the default values for f_p. Alternately, Table 11-6 may be used to determine PF as a function of the arrival type based on the default values for P (i.e., $R_p \times g/C$) and f_p associated with each arrival type. If PF is estimated by Equation 11-6, its calculated value may exceed 1.0 for Arrival Type 4 with extremely low values of g/C. As a practical matter, PF should be assigned a maximum value of 1.0 for Arrival Type 4. This constraint has already been taken into consideration in the values shown in Table 11-6.

Application of the progression adjustment factor requires detailed knowledge of offsets, travel speeds, and intersection signalization. When delay is estimated for future situations involving coordination, particularly when alternatives are analyzed, it is advisable to assume Arrival Type 4 as a base condition for coordinated lane groups, in which case P may be estimated from Table 11-5 and Equation 11-5 as $R_p \times g/C$. Arrival Type 3 should be assumed for all uncoordinated lane groups.

Movements made from exclusive left-turn lanes on protected phases are not usually provided with good progression. Thus, Arrival Type 3 is usually assumed for coordinated left turns. When the actual arrival type is known, it should be used. When the coordinated left turn is part of a protected-permitted phasing, only the effective green for the protected phase should be used to determine the PF since the protected phase is normally associated with platooned coordination. When a lane group contains movements that have different levels of coordination, a flow-weighted average of P should be used in determining the PF.

Actuated Control Adjustment Factor (k)

The incremental delay adjustment term k in Equation 11-4 incorporates the effect of controller type on delay. For pretimed signals, a k-value of 0.50 is used. This value is based on a queuing process with random arrivals and uniform service equivalent to the lane group capacity. Actuated controllers, on the other hand, have the ability to adjust the green time to the cyclic demand, thus reducing the overall incremental delay component. The delay reduction depends in part on the controller's unit extension and the prevailing degree of saturation. Recent research indicates that lower unit extensions result in lower values of k and d_2; however, when the degree of saturation approaches 1.0, an actuated controller behaves similarly to a pretimed controller, resulting in k-values of 0.50 at $X \geq 1.0$. Table 11-7 illustrates recommended k-values for pretimed and actuated lane groups with different unit extensions and degrees of saturation.

Upstream Filtering/Metering Adjustment Factor (I)

The incremental delay adjustment term I in Equation 11-4 incorporates the effects of metering arrivals from upstream signals. For isolated signals, an I-value of 1.0 is used. This value is based on a queuing process with random arrivals such that the ratio of the variance to mean arrivals per cycle is equal to 1.0. Upstream signals decrease the variance by metering arrivals at the downstream intersection, thus reducing the ratio of the variance to mean arrivals per cycle. The I-value and the resultant delay reduction depend on the through movement's degree of saturation at the upstream intersection and the amount of entering and exiting traffic between the two intersections. Table 11-8 illustrates recommended values of I for different upstream degrees of saturation at the upstream intersection.

Example

Delay is a complicated variable that is sensitive to a variety of local and environmental conditions. The procedures provided here

TABLE 11-7. RECOMMENDED k-VALUES FOR LANE GROUPS UNDER ACTUATED AND PRETIMED CONTROL

UNIT EXTENSION (UE) (SEC)	DEGREE OF SATURATION (X)					
	≤0.50	0.60	0.70	0.80	0.90	≥1.0
≤2.0	0.04	0.13	0.22	0.32	0.41	0.50
2.5	0.08	0.16	0.25	0.33	0.42	0.50
3.0	0.11	0.19	0.27	0.34	0.42	0.50
3.5	0.13	0.20	0.28	0.35	0.43	0.50
4.0	0.15	0.22	0.29	0.36	0.43	0.50
4.5	0.19	0.25	0.31	0.38	0.44	0.50
5.0	0.23	0.28	0.34	0.39	0.45	0.50

NOTES: 1. $k = 0.50$ for nonactuated lane groups.
2. For a given UE and its k_{min} value at $X = 0.5$, $k = (1 - 2k_{min})(X - 0.5) + k_{min}$.
3. For UE > 5.0, extrapolate to find k, keeping $k \leq 0.5$.

TABLE 11-8. RECOMMENDED I-VALUES FOR LANE GROUPS WITH UPSTREAM SIGNALS

	DEGREE OF SATURATION AT UPSTREAM INTERSECTION (X_u)						
	0.40	0.50	0.60	0.70	0.80	0.90	≥1.0
I	0.922	0.858	0.769	0.650	0.500	0.314	0.090

NOTE: $I = 1.0 - 0.91X_u^{2.68}$ and $X_u \leq 1.0$.

present reasonable estimates for delays expected for average conditions. They are most useful when used to compare operational conditions for various geometric or signalization designs. When existing conditions are evaluated, it is advisable to measure delay in the field. Appendix III of Chapter 9 contains guidelines for intersection delay measurements using lane occupancy and volume counts.

Description: Consider an arterial segment with a through lane group with two lanes, a demand volume of 1,500 veh/hr, and peak hour factor of 0.91. Further, a pretimed signal has a cycle length of 90 sec, the g/C ratio is 0.60, and X or the v/c ratio is 0.90. Vehicles arrive as a dense platoon at the beginning of the green. What is the estimated control delay for the through lane group?

Solution: To use Equations 11-3 and 11-4 to compute control delay, it is necessary to know C, g/C, X, and c. The last term must be computed.

The adjusted demand flow rate is

$$v = (1{,}500/0.91) = 1{,}648 \text{ veh/hr}$$

Once it is known that X or the v/c ratio is 0.90, c can be calculated as

$$c = v/(v/c)$$

$$c = 1{,}648/(0.90) = 1{,}831 \text{ veh/hr}$$

The intersection control delay is computed as $d = 17.8 + 3.5 = 21.3$ sec/veh.

From the description of the arriving vehicles, the arrival type is 5. On the basis of a pretimed signal, a g/C ratio of 0.60, and Arrival Type 5, Tables 11-6, 11-7, and 11-8 are consulted to find $PF = 0.0$, $k = 0.5$, and $I = 1.0$, respectively. Thus, the estimated control delay = 3.5 sec/veh.

The computations must be done for each signalized intersection or obtained from the results of Chapter 9 evaluations. Figure 11-5 is a summary worksheet for intersection delay computations. An additional blank worksheet may be found in Appendix II to this chapter.

STEP 6—COMPUTE AVERAGE TRAVEL SPEED

The average speed is to be computed by segment and over the entire arterial. It is recommended that the user also prepare a speed profile of the facility and supplement the LOS assessment with insights gained from the speed profile and the levels of service of the individual intersections.

Figure 11-6 shows a worksheet, with some illustrative data filled in, which is provided to ease the task of assembling the information.

Equation 11-1 is used to compute the arterial speed for each segment and for the overall facility. Performing these computations results in the speed profile shown in Figure 11-7. For segments 1 and 9, the running time per mile for a segment 0.10 mi long is used, but is multiplied by the actual segment lengths.

Sample Computation. Fourth Avenue is a principal arterial of intermediate design with a 35-mph free-flow speed (Figure 11-6). From Table 11-3, it is arterial Classification III. In Section 2 of the arterial, the average segment length is 0.20 mi. From Table 11-4, the running time per mile is 128 sec.

The total running time in the section is given by

$$128 \times (0.20 + 0.20 + 0.20) = 76.8 \text{ sec}$$

The control delay for the arterial through movements at the three intersections in Section 2 is given in Figure 11-6 as 5.0 + 7.0 + 10.0 = 22.0 sec, so the total travel time is 76.8 + 22.0 = 98.8 sec.

The arterial speed in the section is 3,600 x 0.60/98.8 = 21.9 mph.

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: ____________________ ____________________ - bound

File or Case # ____________________ Date: ____________________

Prepared by: ____________________

Seg-ment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1^a \star PF^b$	Filtering/ Metering Factor I^c	Incre-mental Delay d_2^d	Control Delay d^e(sec)	Through Move-ment LOS[f]
1										
2										
3										
4										
5										
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal).
[f] Table 9-1.

Figure 11-5. Worksheet for summary of arterial intersection delay estimates.

STEP 7—ASSESS LEVEL OF SERVICE

A distinct set of arterial LOS criteria has been established for each arterial classification. These sets of criteria are based on the differing expectations drivers are judged to have for the different classes of arterials.

In the arterial LOS definitions, both the free-flow speed of the arterial classification and the intersection LOS definitions are taken into account. In general, the arterial LOS criteria are based on the smooth and efficient movement of through traffic along an entire arterial. Therefore, it is necessary to expect less delay per segment than the corresponding intersection level of service.

Table 11-1 gives the arterial LOS criteria for each of the four arterial classifications. The lower the arterial classification (i.e., the larger the classification number), the lower the driver's expectations while driving on that facility and the lower the speed associated with a given level of service. Thus, a Classification III arterial provides LOS B at a lower speed than does a Classification I arterial.

The analyst should be aware of this relationship in explaining before-and-after assessments of arterials when upgrading is involved. If reconstruction results in upgrading a facility from Classification II to Classification I, it is possible that the level of service will not change (or may even technically degrade), despite higher average speed and other improvements, because expectations are higher.

Note that the concept of an overall arterial level of service is generally meaningful only when all segments on the arterial are of the same classification. If different arterial classifications are represented, the LOS criteria are different.

COMPUTATION OF ARTERIAL LOS WORKSHEET

Arterial: Fourth Avenue North -bound

File or Case #: 001 Date: 10/20/93

Prepared by: ______

$$ART\ SPD = \frac{3600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	0.08	III	35	1	11.6	10.0					
2	0.12	III	35	1	17.4	13.0		77.5	0.30	13.9	
3	0.10	III	35	1	14.5	11.0					
4	0.20	III	35	2	25.6	5.0					
5	0.20	III	35	2	25.6	7.0		98.8	0.60	21.9	
6	0.20	III	35	2	25.6	10.0					
7	0.10	III	35	3	14.5	8.0		22.5	0.10	16.0	
8	0.15	III	35	4	20.2	4.0		24.2	0.15	22.3	
9	0.05	III	35	5	7.2	6.0		13.2	0.05	13.6	
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = 236.2

Grand Sum of Length (y) = 1.20

3600×(y)/(x) = 18.3

Arterial LOS = ______

Figure 11-6. Worksheet for computation of arterial level of service.

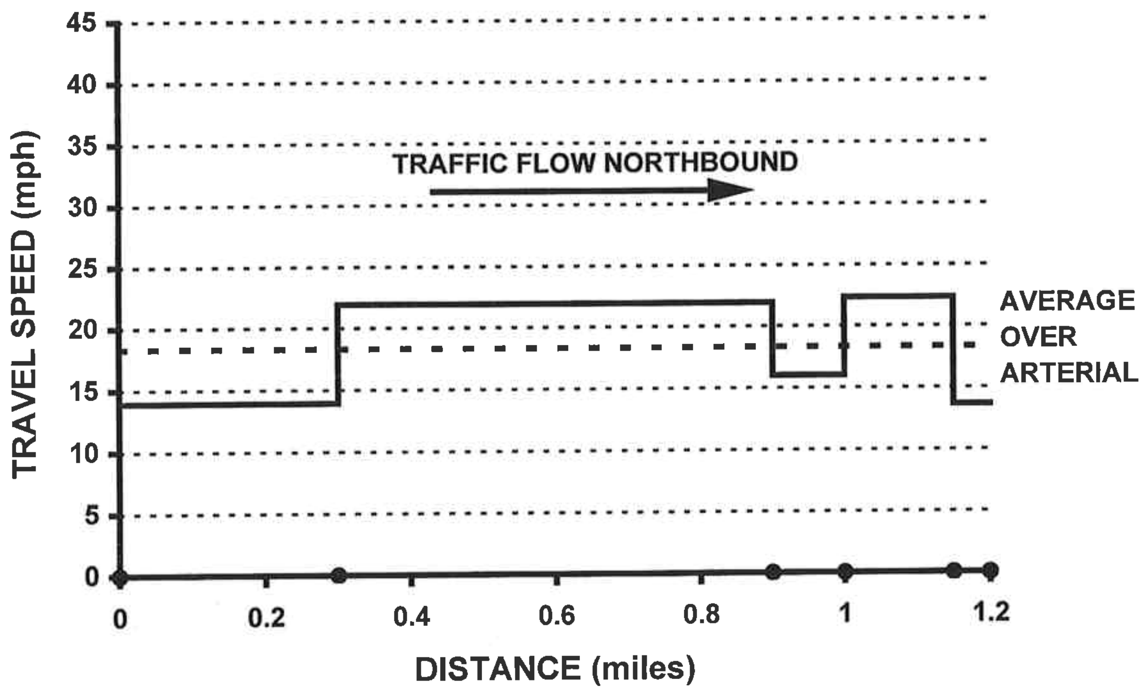


Figure 11-7. Speed profile by arterial section.

III. PLANNING APPLICATIONS

OBJECTIVES

The objective of an arterial LOS analysis at a planning level is to approximate the operating conditions of the facility. A major use for this type of analysis is related to growth management issues. The accuracy of the planning LOS analysis is largely dependent upon the degree of generalization of input data and should not be used for design or operational analyses. The planning method is most applicable when

1. LOS estimates are desired,
2. Field data are lacking,
3. Relatively long planning horizons are used, and
4. Individuals with limited transportation planning experience are involved.

A major difference between the planning analysis of signalized intersections and that of arterials is the treatment of turning vehicles. Whereas the purpose of a signalized intersection is to move vehicles (including turning vehicles) past a point, the purpose of an arterial is to move (through) vehicles over a reasonable length of roadway. Because the emphasis of an arterial is on through movement, the major simplifying assumption in this planning application is that left turns are accommodated by providing left-turn bays at major intersections and controlling the left-turn movement with a separate phase that is properly timed. With this simplifying assumption, many of the inputs and complexities of intersection analyses can be handled abstractly as default values, allowing a relatively easy-to-use planning application; however, as a result of this assumption, planning application results should not be used for intersection design or traffic operations analyses.

DATA REQUIREMENTS

To conduct a planning analysis, traffic, roadway, and signal input values or assumed defaults are needed for the following characteristics:

Traffic characteristics:
- Annual average daily traffic (*AADT*),
- Planning analysis hour factor (*K*),
- Directional distribution factor (*D*),
- Peak hour factor (*PHF*),
- Adjusted saturation flow rate,
- Percentage of turns from exclusive lanes;

Roadway characteristics:
- Number of through lanes (*N*),
- Free-flow speed,
- Arterial classification,
- Medians,
- Left-turn bays or exclusive left-turn lanes;

Signal characteristics:
- Arrival type,
- Signal type,
- Cycle length (*C*),
- Effective green ratio (*g*/*C*).

Some of these characteristics are discussed in the remainder of this section.

Planning Analysis Hour Factor (*K*)

The planning analysis hour factor represents the percentage of *AADT* occurring in the peak hour. For planning purposes many possible peak hours may be appropriate. *K*30 (the 30 highest hour volumes of the year) is widely accepted as the design hour in nonurban areas. *K*100 approximates the typical weekday peak hour during the peak season in developed areas and is frequently used in long-range urban transportation models. *K*200 to 400 is a better representation of a typical peak hour of the year. In many urban areas, general ranges for *K*30, *K*100, and *K*200 to 400 are 8.5 to 11.0 percent, 8.0 to 10.0 percent, and 7.0 to 9.0 percent, respectively. The analyst needs to determine the appropriate peak hour.

Adjusted Saturation Flow Rate

Numerous factors affect the saturation flow rate per lane (see Chapter 9). For a planning analysis, these adjustments may reasonably be combined and multiplied by the ideal saturation flow rate to determine an adjusted saturation flow rate. On the basis of an ideal saturation flow rate of 1,900 passenger cars per hour of green time per lane (pcphgpl), a reasonable range for urban arterials during the peak hour is 1,750–1,850 pcphgpl.

Percentage Turns from Exclusive Lanes

Turns from exclusive lanes represent the percentage of vehicles performing left- or right-turn movements at signalized intersections from lanes dedicated solely to turning movements. The planning methodology assumes that left turns are accommodated by separate lanes and phases so that they have minimal effect on through vehicles. Where a separate right-turn lane exists, it is reasonable to add the percentage of right turns to the percentage of left turns (assuming a left-turn bay or lane) to determine the percentage of turns from exclusive lanes.

Number of Through Lanes

Because significant delays seldom occur in midblock locations, a parameter of importance is the number of through and shared right-turn lanes at signalized intersections; however, when significant midblock delays occur or reasonable lane continuity between intersections is not maintained, caution should be used in strictly applying the concept of the number of such lanes.

Free-Flow Speed

For planning purposes, an arterial's free-flow speed should be based on actual studies of the road or on studies of similar roads and should be consistent with arterial classifications. The actual or probable posted speed limit may be used as a surrogate for free-flow speed if comparable roadway free-flow studies do not exist.

Medians

Medians are painted, raised, or grassed areas that separate opposing midblock traffic lanes and that are wide enough to serve as bays for turning vehicles. For planning purposes, the adjusted saturation flow rate may be reduced 5 percent for roadways that do not have medians.

Left-Turn Bays or Exclusive Left-Turn Lanes

Left-turn bays or lanes are storage areas at signalized intersections to accommodate left-turn movements. These bays or lanes must be long enough to accommodate left turns without impeding the through movement. For planning purposes, the saturation flow rate should be reduced 20 percent for roadways that do not have left-turn bays at major intersections. (This value is a 15 percent additional reduction for a roadway that does not have a median.)

Effective Green Ratio (g/C)

The parameter g/C is the ratio of the time allocated for the through traffic movement (red clearance minus the startup lost time minus effective green time) to the cycle length (C). An arterial's through g/C for each intersection is desirable; however, for broad planning purposes a weighted g/C may be appropriate. The weighted g/C of an arterial is the average of the critical-intersection through g/C and the average-intersection through g/C. For example, if an arterial section has three signalized intersections with effective green ratios of 0.4, 0.7, and 0.7, the critical intersection has a g/C of 0.4 (the lowest g/C); the average intersection has a g/C of 0.6 [(0.4 + 0.7 + 0.7)/3], and the weighted g/C is 0.5 [the average of the critical g/C and the average g/C, (0.6 + 0.4)/2]. Thus, the weighted g/C takes into account the adverse impact of the critical intersection and the overall quality of flow for the arterial length. Average weighted effective green ratios for arterials vary by road purposes and by areas.

COMPUTATIONAL STEPS

The calculation process for determining arterial level of service is illustrated in Figure 11-8 and consists of the following steps:

1. Convert daily volumes to the planning analysis hour by an appropriate planning analysis hour factor (*K*).
2. Multiply *K* by the directional distribution factor (*D*) to obtain hourly directional volumes.
3. Adjust the hourly directional volumes based on *PHF* and turns from exclusive lanes to yield estimated through volumes for 15-min service flow rates.
4. Calculate the running time on the basis of arterial classification, intersection spacing, and free-flow speed.
5. Calculate the intersection control delay on the basis of adjusted saturation flow rates, number of lanes (*N*), arrival type, signal type, cycle length (*C*), and g/C for each intersection using Equations 11-3 and 11-4.
6. Calculate the average travel speed using running time and intersection control delay.
7. Obtain arterial level of service on the basis of the average travel speed.

Calculation 8 in Section IV of this chapter illustrates the computational steps in a planning analysis.

Frequently in a planning analysis, however, the level of service may be given and the desired outcome is a volume—hourly direc-

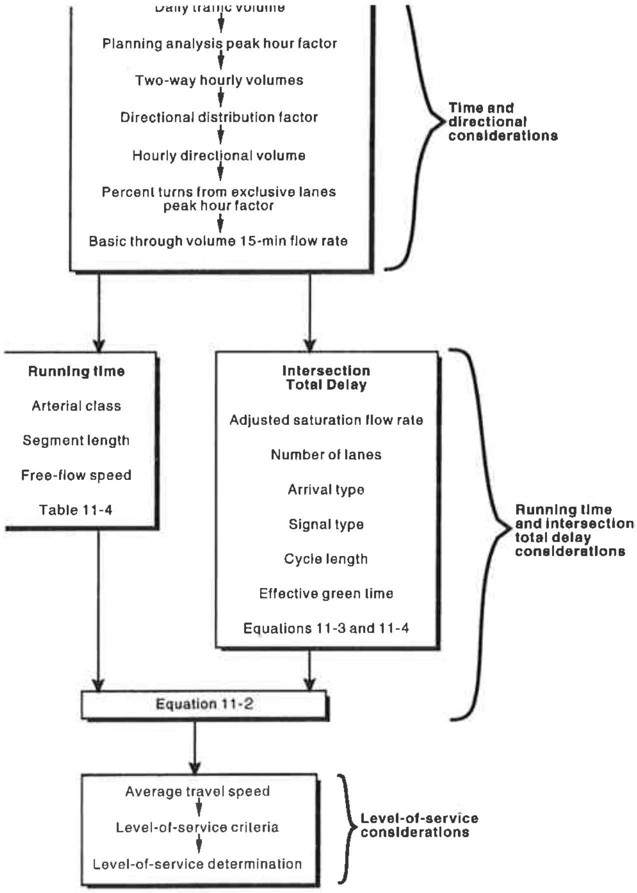


Figure 11-8. Arterial LOS calculation process.

tional, hourly nondirectional, or daily. For such applications, the calculation process is essentially reversed, as follows:

1. Select the level of service and the corresponding average travel speed (range or minimum) based on arterial type and free-flow speed as selected from Table 11-1.
2. Compute the total section running time for the given arterial type, number of intersections, free-flow speed, and section length.
3. Calculate the control delay at all intersections (d) using Equation 11-1 and Steps 1 and 2.
4. Compute v by using the value of average control delay, given number of lanes, adjusted saturated flow rate, arrival type, C, and weighted g/C in Equations 11-2 to 11-4.
5. Determine the hourly directional volume for the design hour using the percentage of turns from exclusive lanes, basic through 15-min volumes, and the *PHF*.
6. Calculate the two-way hourly directional volume for the design hour using the hourly directional volume and the directional distribution factor.
7. Determine *AADT* using the two-way hourly directional volume and the applicable K factor.

INTERPRETATION OF RESULTS

Planning analysis results range from a rough estimate of level of service to an operational analysis, depending primarily on the degree to which default values are used as input. For example, using statewide defaults for appropriate traffic, roadway, and signal characteristics results in rough LOS estimates. Using area- or roadway-specific data but treating all signal characteristics the same (e.g., using a weighted g/C approach) should provide more accurate LOS estimates. Using specific traffic, roadway, and signal data for each road segment and traffic signal should provide an even more accurate LOS estimate. The next level of precision is a detailed treatment of turning movements and signal timing, which is essentially an operational analysis except that projected rather than actual traffic volumes are used.

IV. SAMPLE CALCULATIONS

CALCULATION 1—ARTERIAL CLASSIFICATION

1. *Description*: An arterial with three lanes in each direction and signal spacing of 0.15 mi passes through an area with moderate roadside development. Virtually all of the traffic passes through the area; there is very little pedestrian activity. Identify the arterial classification.

2. *Solution*: To determine the arterial classification, it is necessary to decide the design and functional categories of the arterial and then to use Table 11-3 to specify the arterial classification. The statement that "virtually all of the traffic passes through the area" defines the functional category: the roadway is a principal arterial. Table 11-2 can be used to assist in determining the design category. Note that the arterial is a multilane undivided facility with approximately seven signals per mile (based on 0.15-mi spacing), moderate roadside development, and very little pedestrian activity. The design category is therefore intermediate.

Referring to Table 11-3, one concludes that the arterial is Classification III. This information is used in determining the LOS definitions to be used in evaluating the arterial. Further, lacking more specific information, one can expect a free-flow speed on the order of 33 mph (refer to the top of Table 11-1), with a range of 30 to 35 mph.

CALCULATION 2—COMPUTATION OF ARTERIAL LEVEL OF SERVICE

1. *Description*: A multilane divided facility functions as a principal arterial. There is significant access control, no parking, and a signal spacing of approximately 0.30 mi between pretimed signals. The arterial has little roadside development, two lanes in each direction, and a measured free-flow speed of 39 mph.

Detailed information on the intersection parameters and the arterial segments for the southbound flow is contained in Figures 11-9 and 11-10. The progression is excellent in the southbound direction.

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 2* *South* - bound

File or Case #: Date: *03/21/97*

Prepared by:

Segment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1^a * PF^b$	Filtering/ Metering Factor f^c	Incremental Delay d_2^d	Control Delay d^e(sec)	Through Movement LOS^f
1	*70*	*0.60*	*0.583*	*1800*	*5*					
2	*70*	*0.60*	*0.611*	*1800*	*5*					
3	*70*	*0.60*	*0.611*	*1800*	*5*					
4	*70*	*0.60*	*0.611*	*1800*	*5*					
5	*70*	*0.60*	*0.597*	*1800*	*5*					
6	*70*	*0.60*	*0.593*	*1800*	*5*					
7	*70*	*0.60*	*0.593*	*1800*	*5*					
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal).
[f] Table 9-1.

Figure 11-9. Calculation 2, description: using worksheet for summary of arterial intersection delay estimates.

Determine the arterial level of service by segment and for the entire facility. Do not aggregate the segments.

2. *Solution*: This solution proceeds according to the steps outlined in Figure 11-2. In some applications, it may not be necessary to perform all steps, or it may be easier to do certain steps before others. For instance, if the intersection evaluations have been done previously (or if the summary information is available), that information may be entered on the appropriate worksheet (Figure 11-5) and the control delay computed before the arterial running times are computed.

Step 1. Establish Arterial To Be Considered

This step has been performed in the preceding statement.

Step 2. Determine Arterial Classification

The functional category, principal arterial, is given. The design category may be established by referring to Table 11-2 and noting the following characteristics:

- Multilane divided,
- Significant access control,
- No parking,
- Little roadside development,
- Seven signals in 2.1 mi (three signals per mile).

The facility clearly belongs to the suburban design category.

On the basis of a functional category of principal arterial and a design category of suburban, the facility is found to be a Classification II arterial by referring to Table 11-3.

Step 3. Define Arterial Sections

Step 3 may be skipped because the instructions in the description were not to aggregate the segments. Nonetheless, note that some sections could be aggregated on the basis of average segment lengths and volume pattern. For instance, the following aggregations could be made:

COMPUTATION OF ARTERIAL LOS WORKSHEET

Arterial: *Sample Calculation 2* *South* -bound

File or Case #________ Date: *03/21/97*

Prepared by: ________

$$ART\ SPD = \frac{3600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Control Delay[b] (sec)	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	*0.20*		*39*	*1*							
2	*0.20*		*39*	*2*							
3	*0.30*		*39*	*3*							
4	*0.30*		*39*	*4*							
5	*0.30*		*39*	*5*							
6	*0.40*		*39*	*6*							
7	*0.40*		*39*	*7*							
8											
9											
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = ________

Grand Sum of Length (y) = ________

3600×(y)/(x) = ________

Arterial LOS = ________

Figure 11-10. Calculation 2, description: using worksheet for computation of arterial level of service.

Segment	*Section*
1	1
2	1
3	2
4	2
5	2
6	3
7	3

If the volume differences make the user uncomfortable with this aggregation, it could be checked after the intersection delay has been estimated.

Step 4. Compute Running Time

The arterial is Classification II with a free-flow speed of 39 mph, which establishes the relationship to be used for the running time computation (see Table 11-4).

Consider Segment 1. For a Classification II arterial and a segment length of 0.20 mi, Table 11-4 indicates a running time per mile of 115 sec for a free-flow speed of 40 mph and 125 sec for a free-flow speed of 35 mph. It may be interpolated that for 39 mph the running time would be $117 \times 0.20 = 23.4$ sec. This information is entered on the computation of arterial LOS worksheet.

Step 5. Compute Intersection Delay

Figure 11-11 shows the arterial intersection delay estimates for Calculation 2. Note that because this computation is an arterial evaluation, the information must be for the lane group containing the principal part of the through movement. This information is generally available for the desired lane group from evaluations of individual intersections based on procedures described in Chapter 9.

Equations 11-3 and 11-4 are used to compute the uniform delay (d_1) and the incremental delay (d_2), which can then be entered on the summary worksheet.

The selection of the arrival type for the approaching vehicles is a special consideration. In this case, it is straightforward because of the information given in the description that progression is excellent in the southbound direction. When this information is matched with the arrival type definitions, Arrival Type 5 is selected because it is defined as a dense platoon arriving at the beginning of the green phase with a highly favorable progression quality.

Table 11-6 shows the progression factors (*PF*s) for the pretimed signals and arrival types given. As shown in Figure 11-11, all the intersections have g/C ratios of 0.60, so a *PF* of 0.00 is used for all of them.

The results of the intersection computations are shown transferred to the arterial LOS worksheet in Figure 11-12.

Step 6. Compute Average Travel Speed

With the running time from Step 4 and the delay time from Step 5, the computations may be summarized using the arterial LOS worksheet. The completed worksheet is shown in Figure 11-12, with the calculation for each section (in this case, each segment) identical in form to that shown on the bottom of the worksheet for the entire arterial.

Figure 11-13 shows the speed profile for the arterial. This drawing is a valuable depiction of the operation and should be constructed as part of each evaluation.

Step 7. Assess Level of Service

With all of the preliminary work done, the final determination of LOS values is straightforward. The speeds computed in the summary arterial LOS worksheet can be compared with the definitions for the appropriate arterial classification (in this case, Classification II, as established in Step 2) given in Table 11-1. These are entered on the arterial LOS worksheet in Figure 11-12 and, together with the intersection levels of service determined previously, on the speed profile in Figure 11-13.

As stated in Section I, intersection LOS values are generally better than the arterial LOS values. This difference is logical, because an intersection with less than 5 sec of delay per vehicle is certainly LOS A, whereas an arterial on which one could travel at a speed of 39 mph but instead has to travel at 30 to 35 mph is somewhat less than LOS A.

CALCULATION 3—COMPUTATION OF ARTERIAL LEVEL OF SERVICE

1. *Description*: The northbound side of the arterial described in Calculation 2 has intersection traffic as shown in Figure 11-14 and very poor progression, with virtually the entire northbound platoon arriving in the middle of the red at each intersection.

Determine the arterial level of service by segment and for the entire facility. Do not aggregate the segments.

2. *Solution*: The calculations for this solution are identical in form and sequence to those of Calculation 2 and will not be repeated; however, certain key points must be highlighted:

- The evaluation of an arterial is *by direction*, and a two-way arterial usually requires two evaluations, one for each direction, just as was required in Calculation 2.
- The arrival types in the two directions are generally different because the progression of the signal timing is often set to favor one direction over the other. This difference has a major impact on the intersection delay estimates.
- It is useful to include the segment numbers in the speed profile (as shown in Figure 11-13), to make the final presentation clear. It is also useful to mark the direction of travel clearly.
- The intersections analyzed are those at the input and output ends of each segment.

The results of the computations are shown in Figures 11-15 and 11-16, and the speed profile is shown in Figure 11-17, which for comparative purposes also shows the southbound speed profile as well as the intersection and arterial levels of service for both directions.

One additional point stands out: the determination of arrival type so that correct *PF*s may be selected. The description states that there is ''very poor progression, with virtually the entire northbound platoon arriving in the middle of the red at each intersection.'' It is important to note that this situation is not the worst condition: a careful reading of the arrival type descriptions makes it clear that Type 2 covers the present case, whereas the worst case—Type 1—is reserved for a dense platoon arriving at the *beginning* of the red phase.

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 2* ____ *South* - bound

File or Case # ____ Date: *03/21/97*

Prepared by: ____

Seg-ment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1^a * PF^b$	Filtering/ Metering Factor I^c	Incre-mental Delay d_2^d	Control Delay d^e(sec)	Through Move-ment LOS[f]
1	70	0.60	0.583	1800	5	0.0	0.786	1.1	1.1	A
2	70	0.60	0.611	1800	5	0.0	0.786	1.2	1.2	A
3	70	0.60	0.611	1800	5	0.0	0.757	1.2	1.2	A
4	70	0.60	0.611	1800	5	0.0	0.757	1.2	1.2	A
5	70	0.60	0.597	1800	5	0.0	0.757	1.1	1.1	A
6	70	0.60	0.593	1800	5	0.0	0.772	1.1	1.1	A
7	70	0.60	0.593	1800	5	0.0	0.776	1.1	1.1	A
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal).
[f] Table 9-1.

Figure 11-11. Calculation 2, solution: using worksheet for summary of arterial intersection delay estimates.

CALCULATION 4—EFFECT OF TRAFFIC FLOW RATE ON ARTERIAL LEVEL OF SERVICE

1. *Description*: An arterial with two lanes in each direction and a 35-mph free-flow speed has been found to be a Classification III arterial. Ten signals are spaced 0.20 mi apart. The intersections all have pretimed signals with a 60-sec cycle length and g/C of 0.50. The progression is excellent.

For a range of adjusted traffic demand from a flow rate of 600 to 1,600 veh/hr, plot the arterial segment speed and find the arterial level of service, as well as the intersection levels of service.

2. *Solution*: The relationships shown in this chapter for arterial running time do not depend explicitly on arterial volume or flow rate (see Note 5, Table 11-4). The arterial speed is sensitive to traffic volume because the intersection delay is dependent on that volume. Recall that the basic relation is Equation 11-1, which is repeated here for convenience:

$$ART\ SPD = \frac{3{,}600 * (\text{length})}{[(\text{running time/mile}) * (\text{length}) + (\Sigma \text{ inters. control delay})]} \quad (11\text{-}1)$$

For the stated situation, the segment running time per mile is found from Table 11-4 to be 128 sec for a segment length of 0.20 mi. The running time in the segment is therefore 128 × 0.20 = 25.6 sec.

The intersection control delay is based on Equations 11-3 and 11-4 and the application of the PF. Two parameters are given (C = 60 sec and g/C = 0.50). The other two, arterial lane group capacity (c) and v/c ratio (X), are not directly given.

Without specific information on the lane group capacity, it is both possible and necessary to compute c = 1,600 × 2 × 0.50 = 1,600 veh/hr, for all segments. If the g/C differed from segment to segment, the computed value would also differ. When this relationship is used for a specific site, the evaluation becomes highly approximate; however, this sample calculation is for a typical or representative arterial.

In the information given, the adjusted demand flow rate varies from v = 600 veh/hr to v = 1,600 veh/hr. For each value of v, the corresponding value of $X = v/1{,}600$, because c = 1,600 veh/hr was just computed above.

The arrival type is 5 because the progression is excellent. The PF is selected from Table 11-6 for Arrival Type 5 and a g/C of 0.50. The results of the computations are given in Table 11-9. The

COMPUTATION OF ARTERIAL LOS WORKSHEET

Arterial: *Sample Calculation 2* *South* -bound

File or Case #________ Date: *03/21/97*

Prepared by: ________

$$ART\ SPD = \frac{3600(sum\ of\ length)}{sum\ of\ time}$$

Seg-ment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	*0.20*	*II*	*39*	*1*	*23.4*	*1.1*		*24.5*	*0.20*	*29.4*	*B*
2	*0.20*	*II*	*39*	*2*	*23.4*	*1.2*		*24.6*	*0.20*	*29.2*	*B*
3	*0.30*	*II*	*39*	*3*	*31.1*	*1.2*		*32.3*	*0.30*	*33.5*	*B*
4	*0.30*	*II*	*39*	*4*	*31.1*	*1.2*		*32.3*	*0.30*	*33.5*	*B*
5	*0.30*	*II*	*39*	*5*	*31.1*	*1.1*		*32.2*	*0.30*	*33.5*	*B*
6	*0.40*	*II*	*39*	*6*	*39.1*	*1.1*		*40.2*	*0.40*	*35.8*	*A*
7	*0.40*	*II*	*39*	*7*	*39.1*	*1.1*		*40.2*	*0.40*	*35.8*	*A*
8											
9											
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = *226.3*

Grand Sum of Length (y) = *2.10*

3600×(y)/(x) = *33.4*

Arterial LOS = *B*

Figure 11-12. Calculation 2, solution: using worksheet for computation of arterial level of service.

estimated control delay is the uniform delay multiplied by the *PF* plus the incremental delay.

The levels of service are identified by referring to Table 11-1 for a Classification III arterial and to Table 9-1 for the intersections. Note that the intersection level-of-service is based on control delay and is shown for the lane group containing the through traffic.

Figure 11-18 is a plot of arterial segment speed as a function of arterial volume for the stated condition of a 0.20-mi segment length. Note that the intersection approach delay ranges from 13 to 32 per-

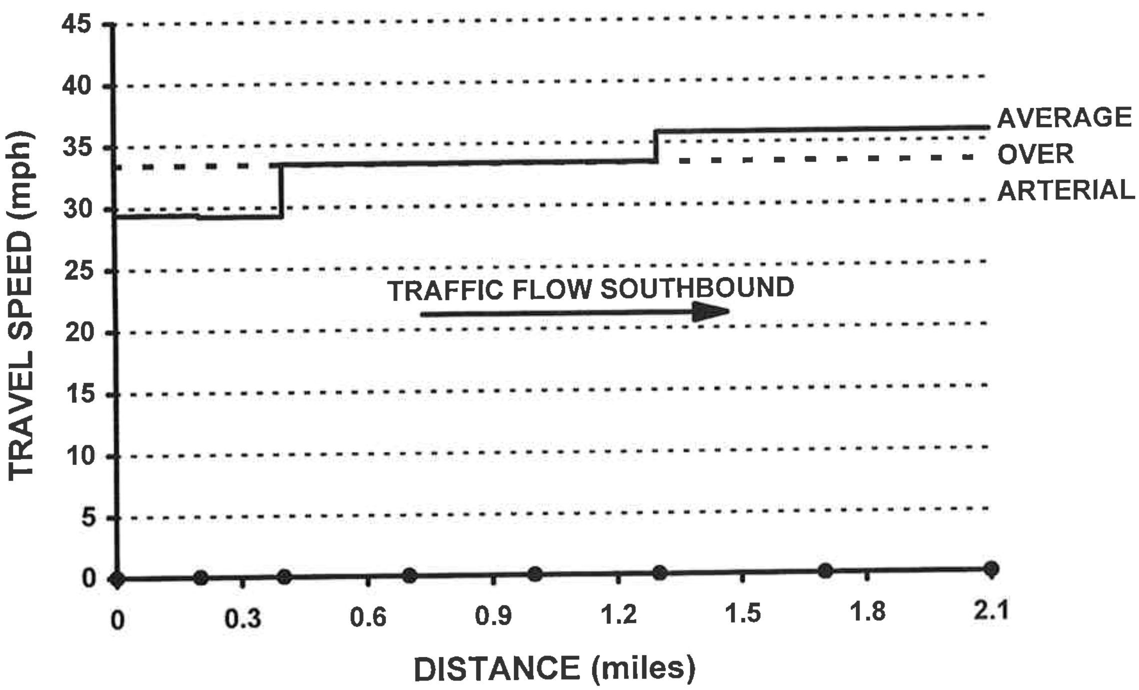


Figure 11-13. Speed profile for Calculation 2, southbound traffic.

cent of the total time spent on the segment, depending on the traffic flow rate. Figure 11-19 is a plot of arterial segment average travel speed as a function of arterial flow rate for a 0.10-mi segment length. For comparative purposes, the plot for a 0.20-mi segment length is also shown. The facts that the speeds are much lower and that the arterial level of service is now significantly lower than the intersection level of service deserve attention.

First, it is necessary to observe that with the 0.10-mi segment, the intersection delay per mile has increased (relative to that in Calculation 4) because there are now more intersections per mile: 5 intersections per mile for the 0.20-mile segment have become 10 intersections per mile for the 0.10-mi segments. Thus, a delay of 8.0 sec/veh per intersection now contributes 10 × 8.0 = 80 sec/mi to the arterial travel time, whereas it was 5 × 8.0 = 40 sec/mi in the previous computation. Thus, two radically different arterials are being compared.

The driver's expectation is more demanding for an arterial than for an individual intersection. With 10 signals per mile, very little delay per intersection is required to degrade the quality of flow for through traffic; however, any intersection with less than 5.0 sec of stopped delay is operating rather well (i.e., LOS A is a realistic evaluation of such an intersection).

CALCULATION 5—EFFECT OF TRAFFIC FLOW RATE AND LENGTH ON ARTERIAL LEVEL OF SERVICE

1. *Description*: Reevaluate Calculation 4 using a signal spacing of 0.10 mi. All other information is the same as that given in Calculation 4, including the arterial classification.

2. *Solution*: Numerically, the computations are the same as in Calculation 4, and all the introductory remarks are the same. The results of the computations are given in Table 11-10.

The levels of service are again identified by referring to Table 11-1 for a Classification III arterial and to Table 9-1 for the intersections.

Table 11-10 illustrates the following point: because of the close signal spacing and the control delay per unit length, it is possible for arterial level of service to be two or even three levels worse than that of a typical intersection. (As will be shown in Calculation 7, it is also possible for the arterial level of service to be better than the intersection level of service when the segment is very long.)

Note that in this calculation, the intersection delay ranges from 20 to 45 percent of the total time spent on the segment, depending on the traffic flow rate. In Calculation 4, the range was 13 to 32 percent.

CALCULATION 6—EVALUATION BASED ON FIELD DATA

1. *Description*: On a given multilane two-way divided arterial with left-turn bays and good access control, the free-flow speed is measured along its length as 45 mph. The following data are collected along its eight eastbound segments, using the field data procedures of Appendix I:

Segment	*Length (mi)*	*Average Travel Time (sec)*	*Average Control Delay (sec/veh)*
1	0.20	28.3	3.4
2	0.15	19.2	1.7
3	0.15	21.8	3.6
4	0.20	29.4	5.3
5	0.25	49.7	17.6
6	0.25	40.6	0.5
7	0.25	35.2	6.2
8	0.20	28.1	3.2

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES										
	Arterial: *Sample Calculation 3*								*North* - bound	
	File or Case #					Date:				
	Prepared by:									
Segment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1{}^a \star PF^b$	Filtering/ Metering Factor I^c	Incremental Delay $d_2{}^d$	Control Delay d^e(sec)	Through Movement LOS[f]
1	*70*	*0.60*	*0.417*	*1800*	*2*					
2	*70*	*0.60*	*0.417*	*1800*	*2*					
3	*70*	*0.60*	*0.417*	*1800*	*2*					
4	*70*	*0.60*	*0.361*	*1800*	*2*					
5	*70*	*0.60*	*0.361*	*1800*	*2*					
6	*70*	*0.60*	*0.306*	*1800*	*2*					
7	*70*	*0.60*	*0.306*	*1800*	*2*					
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal).
[f] Table 9-1.

Figure 11-14. Calculation 3, description: using worksheet for summary of arterial intersection delay estimates.

These data are based on an appropriate number of travel time runs that include both the running time and the intersection control delay.

Find the arterial level of service, by segment and for the entire facility, as well as the intersection levels of service.

2. *Solution*: To determine the arterial classification, consult Tables 11-2 and 11-3 and note that

- The facility is multilane divided,
- Access control is good,
- There are eight signals in 1.65 mi, or about five signals per mile.

It is likely that the design category would be suburban on the basis of Table 11-2. Because the facility is a principal arterial, Table 11-3 leads one to determine that it belongs in arterial Classification II. The ranges of free-flow speed shown in Table 11-4 indicate that a measured free-flow speed of 45 mph is consistent with arterial Classification II.

The field data can also be used to compute the arterial speed by segment and for the entire facility without any need to use Table 11-4. The computations of the arterial speed are shown in the summary of calculations on the completed arterial LOS worksheet in Figure 11-20. The speed calculations are straightforward; for instance, for Segment 1, $ART\ SPD$ = 3,600 × 0.20/28.3 = 25.4 mph.

The LOS determination is made by referring to Table 11-1 for arterial Classification II and applying the definitions; for instance, Segment 1 with a computed speed of 25.4 mph is LOS C.

Figure 11-21 shows the speed profile for the arterial and graphically demonstrates where the problem occurs. Note that the overall

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 3*   *North* - bound

File or Case #______   Date: ______

Prepared by: ______

Segment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1{}^a * PF^b$	Filtering/ Metering Factor I^c	Incremental Delay $d_2{}^d$	Control Delay d^e(sec)	Through Movement LOS[f]
1	70	0.60	0.417	1800	2	10.2	0.913	0.7	10.9	B
2	70	0.60	0.417	1800	2	10.2	0.913	0.7	10.9	B
3	70	0.60	0.417	1800	2	10.2	0.913	0.7	10.9	B
4	70	0.60	0.361	1800	2	9.7	0.913	0.5	10.2	B
5	70	0.60	0.361	1800	2	9.7	0.941	0.5	10.2	B
6	70	0.60	0.306	1800	2	9.2	0.941	0.4	9.6	A
7	70	0.60	0.306	1800	2	9.2	0.962	0.4	9.6	A
8										
9										
10										
11										
12										
13										
14										
15										

[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal).
[f] Table 9-1.

Figure 11-15. Calculation 3, solution: using worksheet for summary of arterial intersection delay estimates.

level of service does not highlight the problem as well as the speed profile or the set of segment levels of service does.

The field data also allow a direct determination of intersection levels of service, based on measured control delay. With the LOS definitions of Table 9-1, the determination is straightforward:

Segment	*Intersection LOS*	*Measured Control Delay (sec/veh)*
1	A	3.4
2	A	1.7
3	A	3.6
4	A	5.3
5	B	17.6
6	B	10.5
7	A	6.2
8	A	3.2

These levels of service are also shown in Figure 11-21.

CALCULATION 7—ARTERIAL WITH LARGE SIGNAL SPACINGS

1. *Description*: Route 25 is a suburban arterial with a free-flow speed of 51 mph measured in field studies. It is an undivided facility, with two lanes in each direction and left-turn bays, and is dominated by its signals. A pretimed set of signals is used on the portion of the facility of interest. The following information is available for westbound traffic for the period being studied:

Segment	*Length (mile)*	*C (sec)*	*g/C*	*X*	*c (veh/hr)*
1	0.7	70	0.60	0.89	1,800
2	0.6	70	0.57	0.97	1,710
3	0.7	70	0.60	0.94	1,800
4	0.7	70	0.60	0.94	1,800
5	0.7	70	0.60	0.94	1,800

COMPUTATION OF ARTERIAL LOS WORKSHEET

Arterial: *Sample Calculation 3* *North* -bound

File or Case #________ Date: *03/21/97*

Prepared by: ________

$$ART\ SPD = \frac{3600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	*0.20*	*I*	*39*	*1*	*23.4*	*10.9*		*34.3*	*0.20*	*21.0*	*D*
2	*0.20*	*I*	*39*	*2*	*23.4*	*10.9*		*34.3*	*0.20*	*21.0*	*D*
3	*0.30*	*I*	*39*	*3*	*31.1*	*10.9*		*42.0*	*0.30*	*25.7*	*C*
4	*0.30*	*I*	*39*	*4*	*31.1*	*10.2*		*41.3*	*0.30*	*26.1*	*C*
5	*0.30*	*I*	*39*	*5*	*31.1*	*10.2*		*41.3*	*0.30*	*26.1*	*C*
6	*0.40*	*I*	*39*	*6*	*39.1*	*9.6*		*48.7*	*0.40*	*29.5*	*B*
7	*0.40*	*I*	*39*	*7*	*39.1*	*9.6*		*48.7*	*0.40*	*29.5*	*B*
8											
9											
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = *290.7*

Grand Sum of Length (y) = *2.10*

3600×(y)/(x) = *26.0*

Arterial LOS = *C*

Figure 11-16. Calculation 3, solution: using worksheet for computation of arterial level of service.

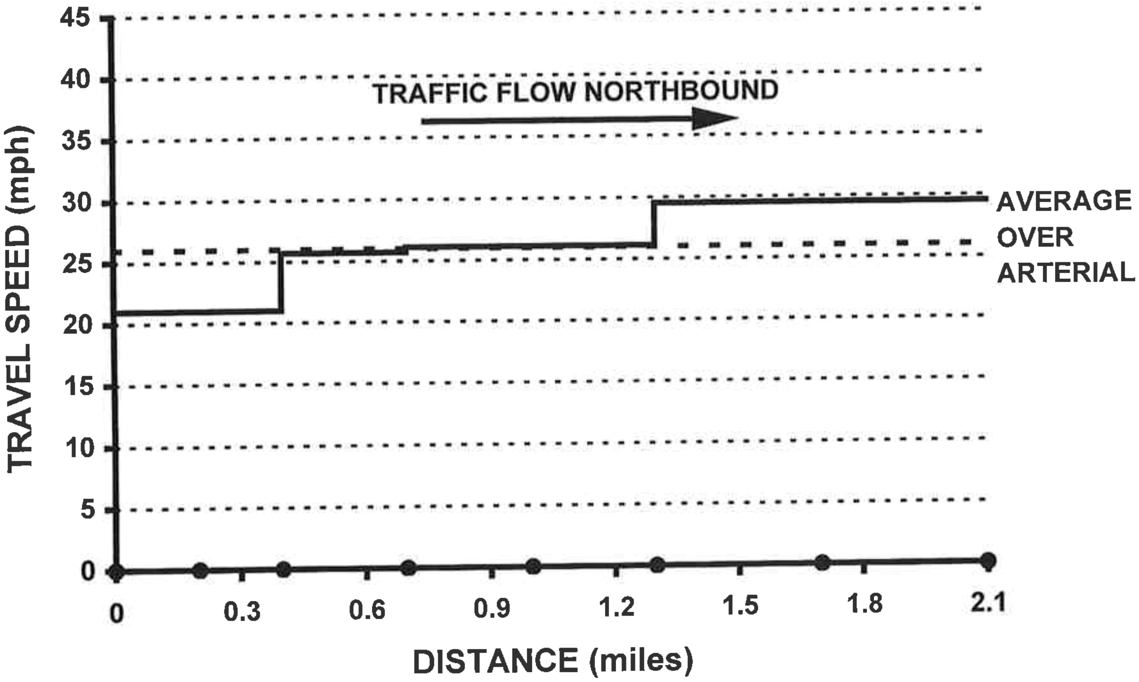


Figure 11-17. Speed profile for Calculation 3, northbound traffic.

TABLE 11-9. COMPUTATIONS FOR SAMPLE CALCULATION 4

DIST = 0.20 MILES, CYCLE LENGTH = 60 SECONDS, g/C = 0.50

FLOW (VPH)	CAPACITY (VPH)	v/c RATIO X	UNIFORM DELAY d_1	PF	I	INCREMENTAL DELAY d_2	INTERSECTION CONTROL LEVEL OF SERVICE	INTERSECTION APPROACH CONTROL DELAY	SEGMENT RUN-NING TIME	SUM TIME (SEC)	AVERAGE TRAVEL SPEED (MPH)	ARTERIAL LEVEL OF SERVICE
600	1,600	0.38	9.2	0.33	0.93	0.6	A	3.7	25.6	29.3	24.6	B
700	1,600	0.44	9.6	0.33	0.90	0.8	A	4.0	25.6	29.6	24.3	B
800	1,600	0.50	10.0	0.33	0.86	1.0	A	4.3	25.6	29.9	24.1	B
900	1,600	0.56	10.4	0.33	0.81	1.2	A	4.6	25.6	30.2	23.8	C
1,000	1,600	0.63	10.9	0.33	0.74	1.4	A	5.0	25.6	30.6	23.5	C
1,100	1,600	0.69	11.4	0.33	0.67	1.6	A	5.4	25.6	31.0	23.2	C
1,200	1,600	0.75	12.0	0.33	0.58	1.9	A	5.9	25.6	31.5	22.8	C
1,300	1,600	0.81	12.6	0.33	0.48	2.3	A	6.5	25.6	32.1	22.4	C
1,400	1,600	0.88	13.3	0.33	0.36	2.7	A	7.2	25.6	32.8	22.0	C
1,500	1,600	0.94	14.1	0.33	0.23	3.5	A	8.2	25.6	33.8	21.3	C
1,600	1,600	1.00	15.0	0.33	0.09	6.8	B	11.8	25.6	37.4	19.3	C

The signal progression is good, with less than 10 percent of the through traffic stopping.

Determine the arterial level of service by segment and for the entire facility.

2. *Solution*: On the basis of the free-flow speed, the facility is arterial Classification I. Refer to Table 11-1 or Table 11-4.

The intersection delay may be computed using Equations 11-3 and 11-4, with the computations summarized on the worksheet for summary of arterial intersection delay estimates (Figure 11-22).

On the basis of *PF* descriptions in this chapter, the arrival type is 5—a dense platoon arriving at the beginning of the green phase, a highly favorable progression. This judgment is based on the given condition that the signal progression is good, with less than 10 percent of the through traffic stopping.

Table 11-6 indicates the following *PF* values for pretimed control and Arrival Type 5:

g/C Ratio	*Progression Factor,* PF
0.50	0.333
0.60	0.000

From these values, the *PF* for a *g/C* ratio of 0.57 may be interpolated as 0.099.

Given the free-flow speed of 51 mph and the relatively long signal spacing, the free-flow speed can be used as the arterial speed in computing the running time:

Segment running time = 3,600 × (segment length)/(*ART SPD*)

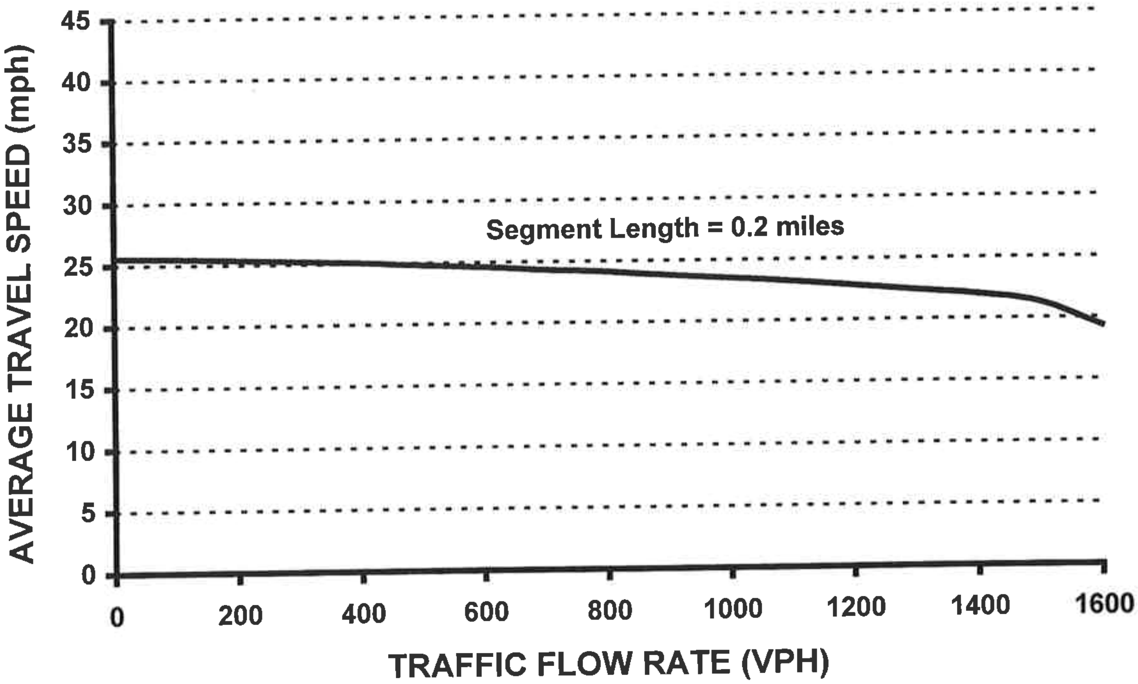


Figure 11-18. Sample calculation speed as a function of arterial flow rate.

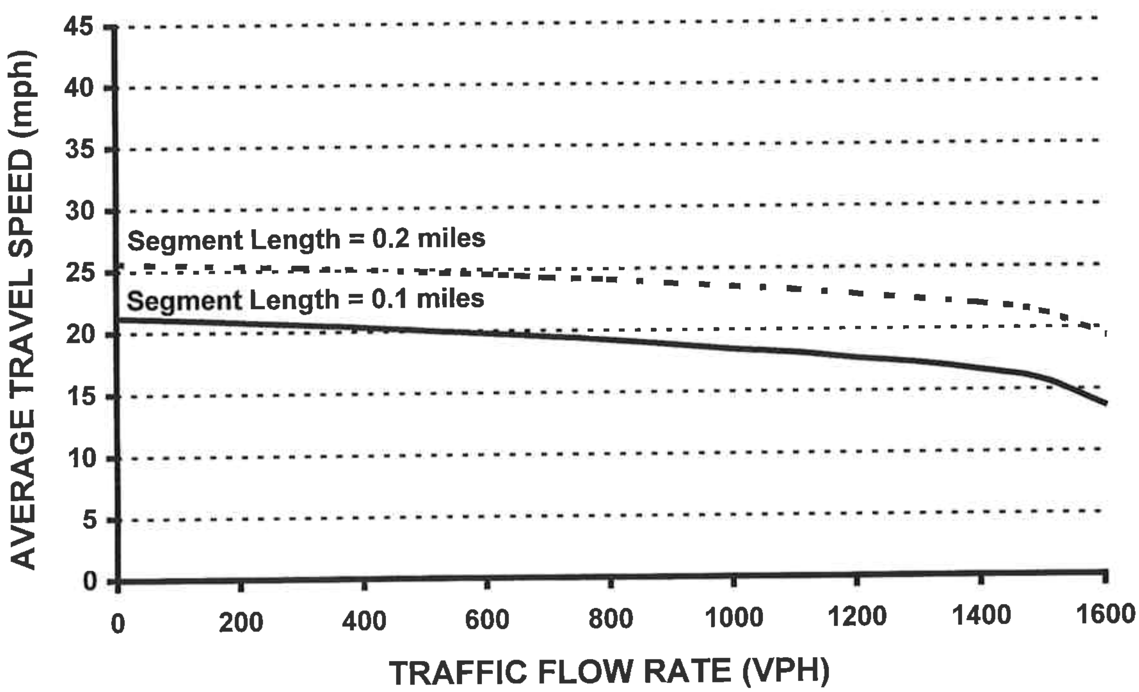


Figure 11-19. Calculation 5 speed as a function of arterial flow rate on two different segment lengths.

For instance, in Segment 1,

Segment running time = 3,600 × (0.70)/(51 mph) = 49.4 sec

To this computed running time is added the intersection delay time in the usual way, as shown in Figure 11-23.

If Table 11-4 is inspected carefully, a more precise estimate of the computed running time can be generated. For instance, for a segment length of 0.50 mi and a free-flow speed of 50 mph, the segment running time is (78/72) or 1.08 times the value of a 1.0-mi segment. Thus, more precise estimates for such segment lengths as 0.60, 0.80, and 0.90 mi could be generated for a free-flow speed of 51 mph by similar logic. However, the better and more accurate approach would be to rely on field data for such an arterial.

Figure 11-23 also indicates the level of service for each arterial

TABLE 11-10. COMPUTATIONS FOR SAMPLE CALCULATION 5

							DIST = 0.10 MILES, CYCLE LENGTH = 60 SECONDS, g/C = 0.50					
							INTERSECTION					
FLOW (VPH)	CAPACITY (VPH)	v/c RATIO X	UNIFORM DELAY d_1	PF	I	INCREMENTAL DELAY d_2	CONTROL LEVEL OF SERVICE	APPROACH CONTROL DELAY	SEGMENT RUNNING TIME	SUM TIME (SEC)	AVERAGE TRAVEL SPEED (MPH)	ARTERIAL LEVEL OF SERVICE
600	1,600	0.38	9.2	0.33	0.93	0.6	A	3.7	14.5	18.2	19.8	C
700	1,600	0.44	9.6	0.33	0.90	0.8	A	4.0	14.5	18.5	19.5	C
800	1,600	0.50	10.0	0.33	0.86	1.0	A	4.3	14.5	18.8	19.2	C
900	1,600	0.56	10.4	0.33	0.81	1.2	A	4.6	14.5	19.1	18.8	C
1,000	1,600	0.63	10.9	0.33	0.74	1.4	A	5.0	14.5	19.5	18.4	C
1,100	1,600	0.69	11.4	0.33	0.67	1.6	A	5.4	14.5	19.9	18.1	C
1,200	1,600	0.75	12.0	0.33	0.58	1.9	A	5.9	14.5	20.4	17.6	D
1,300	1,600	0.81	12.6	0.33	0.48	2.3	A	6.5	14.5	21.0	17.2	D
1,400	1,600	0.88	13.3	0.33	0.36	2.7	A	7.2	14.5	21.7	16.6	D
1,500	1,600	0.94	14.1	0.33	0.23	3.5	A	8.2	14.5	22.7	15.8	D
1,600	1,600	1.00	15.0	0.33	0.09	6.8	B	11.8	14.5	26.3	13.7	E

segment, based on the fact that the arterial is Classification I, and referring to the LOS boundaries in Table 11-1. Figure 11-24 depicts the speed profile for the arterial and also indicates the arterial and intersection levels of service based on the average travel speed and control delay values, respectively.

Note that with the large signal spacings on such an arterial, one can expect the intersections to provide the driver with poorer levels of service than the arterial, simply on the basis of LOS criteria for arterials and signalized intersections. Even on a Classification I arterial, LOS A can be achieved with a speed of 42 mph or greater; however, more than 5.0 sec of stopped delay per vehicle removes an intersection from LOS A (refer to Table 9-1).

CALCULATION 8—PLANNING APPLICATION: DETERMINING LEVEL OF SERVICE

1. *Description*: The following information has been determined about an arterial section for the predominant directional flow:

Traffic characteristics:

- $AADT$ = 30,000,
- $K100$ = 0.091,
- D = 0.568,
- PHF = 0.925,
- Adjusted saturation flow = 1,850 pcphgpl,
- Percentage of turns from exclusive lanes = 12;

Roadway characteristics:

- Through lanes = 4 (2 through lanes in each direction),
- Arterial classification = II,
- Free-flow speed = 45 mph,
- Section length = 2 mi,
- Median = yes,
- Left-turn bays = yes;

Signal characteristics:

- Signalized intersections = 4 (thus, average segment length = 0.5 mi),
- Arrival type = 3,
- Signal type = actuated,
- C = 120 sec,
- Weighted g/C = 0.42.

Find the following:

1. Two-way hourly volume for the planning analysis hour,
2. Hourly directional volume based on the predominant directional flow,
3. Basic through-volume 15-min flow rate,
4. Running time,
5. Control delay,
6. Average travel speed, and
7. Level of service for the arterial section.

2. *Solution*: The solution is reached with the following steps:

Step 1. The two-way hourly volume for the planning analysis hour is 2,730 (AADT × K = 30,000 × 0.091).

Step 2. The hourly directional volume for the planning analysis hour is 1,550 (two-way hourly volume × D = 2,730 × 0.568).

Step 3. The basic through-volume 15-min flow rate is 1,475 or the hourly directional volume divided by the product of the PHF and the quantity 1 minus the percentage of turns from exclusive lanes [1,550/0.925 × (1 − 0.12)].

Step 4. The running time of 88 sec/mi is obtained directly from Table 11-4 with arterial Classification II, a segment length of 0.5 mi, and a free-flow speed of 45 mph as entries.

Step 5. The control delay (d) for all the intersections of 140.0 sec is obtained using Equations 11- 2, 11-3, and 11-4, the number of signalized intersections, and the following inputs: adjusted saturation flow rate, number of through and through/right lanes, arrival type, signal type, C, g/C, progression adjustment factor (PF), and incremental delay adjustment factors (k,I). The 140.0 sec is calculated from Equations 11-2, 11-3, and 11-4 with the inputs that follow.

$$d = d_1 \times PF + d_2 + d_3$$

$$d_1 = \frac{0.5C\left[1 - (g/C)\right]^2}{1 - (g/C)\ [\min(X, 1.0)]}$$

$$d_2 = 900T\ [(X - 1) + \sqrt{(X - 1)^2 + 8kIX/Tc}]$$

where

$d_1 = 33.6$ sec,
$d_2 = 3.9$ sec,
$d_3 = 0$ sec,
$d = 33.6 + 3.9 + 0 = 37.5$ sec,

and $\Sigma d = 37.5 \times 4 = 140.0$ sec, $c = 1{,}850 \times 2 \times 0.42 = 1{,}554$; the v/c ratio (X) = 1,475/(1,850 × 2 × 0.42) = 0.949; PF = 1.00 (Table

COMPUTATION OF ARTERIAL LOS WORKSHEET

Arterial: *Route 25* *East* -bound

File or Case # *Sample Calc. 6* Date: *03/21/97*

Prepared by:

$$ART\ SPD = \frac{3600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	0.20	I	45					28.3	0.20	25.4	C
2	0.15	I	45					19.2	0.15	28.1	B
3	0.15	I	45					21.8	0.15	24.8	C
4	0.20	I	45					29.4	0.20	24.5	C
5	0.25	I	45					49.7	0.25	18.1	D
6	0.25	I	45					40.6	0.25	22.2	C
7	0.25	I	45					35.2	0.25	25.6	C
8	0.20	I	45					28.1	0.20	25.6	C
9											
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = *252.3*

Grand Sum of Length (y) = *1.65*

3600×(y)/(x) = *23.5*

Arterial LOS = *C*

Figure 11-20. Calculation 6, solution: using worksheet for computation of arterial level of service.

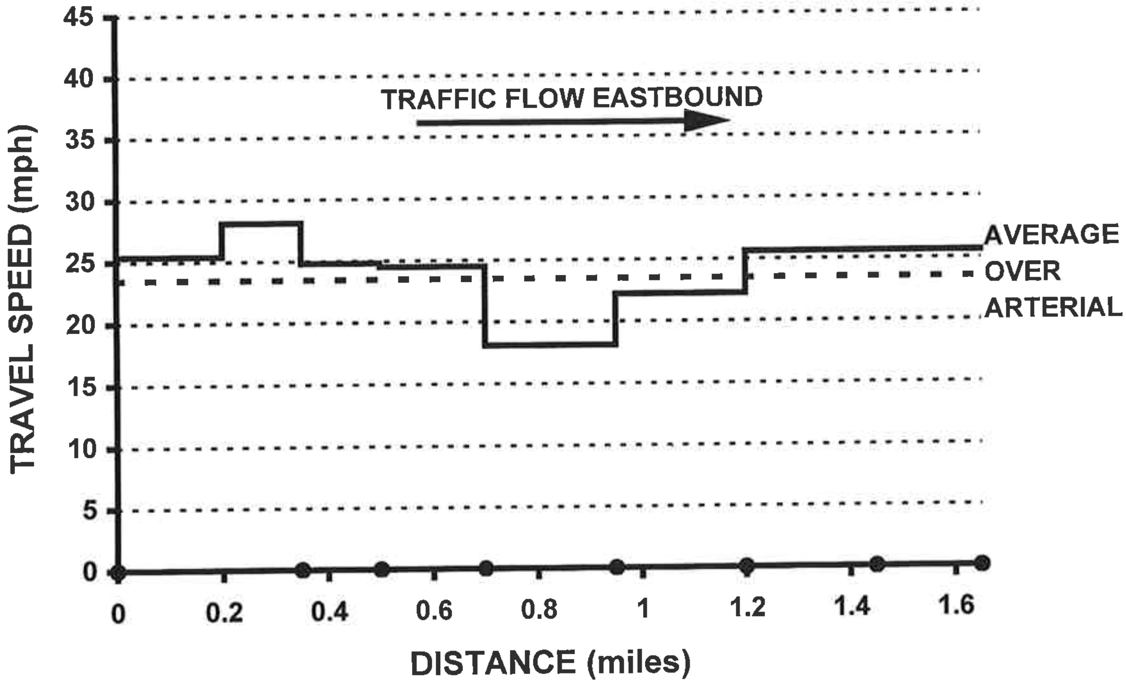


Figure 11-21. Speed profile for Calculation 6.

11-6); T = 0.25 hr; k = 0.5 (for planning purposes); I = 0.209 (Table 11-8); and the number of signalized intersections = 4.

Step 6. The average travel speed is 22.8 mph, which is calculated by applying Equation 11-1 with running time and control delay for all signalized intersections as inputs.

$$ART\ SPD = (3{,}600 \times 2)/[(88 \times 2) + (D)]$$
$$= 7{,}200/(176 + 140.0)$$
$$= 7{,}200/316.0$$
$$= 22.8 \text{ mph}$$

Step 7. On the basis of an average travel speed of 22.8 mph and the criteria in Table 11-1 for a Classification II arterial, the arterial's level of service is C.

CALCULATION 9—PLANNING APPLICATION: DETERMINING VOLUMES

1. *Description*: In preliminary design it is desired to know the maximum volume of vehicles that a six-lane facility could handle at LOS C given the following traffic, roadway, and signal characteristics:

Traffic characteristics:
- $K30$ = 0.095,
- D = 0.55,
- PHF = 0.95,
- Adjusted saturation flow = 1,750 pcphgpl,
- Percentage of turns from exclusive lanes = 12;

Roadway characteristics:
- Through lanes = 6 (3 through lanes in each direction),
- Arterial classification = II,
- Free-flow speed = 40 mph,
- Section length = 2 mi,
- Median = yes,
- Left-turn bays = yes;

Signal characteristics:
- Signalized intersections = 6 (thus, segment length = 0.33 mi),
- Arrival type = 5,
- Signal type = semiactuated,
- C = 120 sec,
- Weighted g/C = 0.42.

Find the following:

1. The lowest acceptable average travel speed for LOS C,
2. The maximum acceptable hourly directional volume based on the predominant directional flow,
3. The maximum acceptable two-way hourly directional volume, and
4. The maximum acceptable $AADT$.

2. *Solution*: The solution is found as follows:

Step 1. From Table 11-1, the lowest acceptable average travel speed for arterial Classification II and LOS C is 22 mph.

Step 2. A running time of 100 sec/mi is obtained by interpolation from Table 11-4 with arterial Classification II, a segment length of 0.33 mi, and a free-flow speed of 40 mph as entries.

Step 3. The control delay for all the intersections of 127.3 sec is calculated by applying Equation 11-1 with average travel speed and running time as inputs:

$$22 = (3{,}600 \times 2)/[(100 \times 2) + d]$$

Solving for d: d = 127.3 sec/veh control delay.

Step 4. The average control delay per vehicle per intersection of 21.2 sec is calculated as follows:

$$d = 127.3/6$$

Solving for d: d = 21.2 sec/veh.

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: *Sample Calculation 7* *West* - bound

File or Case #______ Date: ______

Prepared by: ______

Segment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1^{a} * PF^{b}$	Filtering/ Metering Factor I^{c}	Incremental Delay d_2^{d}	Control Delay d^{e}(sec)	Through Movement LOS[f]
1	70	0.60	0.89	1800	5	0.0	0.334	2.6	2.6	A
2	70	0.57	0.97	1710	5	1.7	0.334	7.4	9.0	A
3	70	0.60	0.94	1800	5	0.0	0.161	2.3	2.3	A
4	70	0.60	0.94	1800	5	0.0	0.229	3.2	3.2	A
5	70	0.60	0.94	1800	5	0.0	0.229	3.2	3.2	A
6										
7										
8										
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11										
12										
13										
14										
15										

[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal).
[f] Table 9-1.

Figure 11-22. Calculation 7, solution: using worksheet for summary of arterial intersection delay estimates.

Step 5. The basic 15-min flow rate of 2,156 is obtained by using Equations 11-2, 11-3, and 11-4; the average control delay of 21.2 sec/veh; *PF*s from Table 11-6; and the following inputs: adjusted saturation flow rate, number of through and through/right lanes, arrival type, signal type, *C*, *g/C*, *k*, and *T*.

The capacity of the lane group (c) = 1,750 × 3 × 0.42 = 2,205; PF = 0.508 (Table 11-6); T = 0.25 hr (15-min period); k = 0.5 (for planning purposes).

Solving for *v/c* ratio: X = 0.978.

Solving for the flow rate: v = 2,156.

Step 6. The hourly directional volume for the design hour is 2,328, the product of the basic 15-min through volume and the *PHF* divided by the quantity 1 minus the percentage of turns from exclusive lanes [2,156 × 0.95/(1 − 0.12)].

Step 7. The two-way hourly directional volume for the design hour is 4,232, the hourly directional volume divided by the directional distribution factor (2,328/0.55).

Step 8. The *AADT* based on the design hour and LOS C is 44,545, the two-way hourly directional volume divided by design hour factor (4,232/0.095).

It should be noted that this example applies to a preliminary design problem, and the planning application and results obtained should not be used beyond preliminary design. A more detailed planning analysis could have used signal-specific effective green ratios, variable turning movements, variable lengths between signalized interesections, and other traffic, roadway, and signal characteristics. The use of 12 percent for turns from exclusive lanes and the application of a weighted effective green ratio of 0.42 to all signalized intersections in this problem are broad planning assumptions and are inappropriate for design and operational analyses.

CALCULATION 10—STOP CONTROL ON ARTERIAL

1. *Description*: Humboldt Boulevard is a minor arterial of intermediate design with a free-flow speed of 30 mph. It is an undivided facility with two lanes per direction and the layout and intersection spacing given in Figure 11-25. The intersections of Keefe Avenue, Locust Street, Center Street, and North Avenue are signal con-

COMPUTATION OF ARTERIAL LOS WORKSHEET

Arterial: *Sample Calculation 7* *West* -bound

File or Case #_____ Date: *03/21/97*

Prepared by: _____

$$ART\ SPD = \frac{3600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	*0.70*	*I*	*51*	*1*	*49.4*	*2.6*		*52.0*	*0.70*	*48.5*	*A*
2	*0.60*	*I*	*51*	*2*	*42.4*	*9.0*		*51.4*	*0.60*	*42.0*	*A*
3	*0.70*	*I*	*51*	*3*	*49.4*	*2.3*		*51.7*	*0.70*	*48.7*	*A*
4	*0.70*	*I*	*51*	*4*	*49.4*	*3.2*		*52.6*	*0.70*	*47.9*	*A*
5	*0.70*	*I*	*51*	*5*	*49.4*	*3.2*		*52.6*	*0.70*	*47.9*	*A*
6											
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14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = *260.4*

Grand Sum of Length (y) = *3.40*

3600×(y)/(x) = *47.0*

Arterial LOS = *A*

Figure 11-23. Calculation 7, solution: using worksheet for computation of arterial level of service.

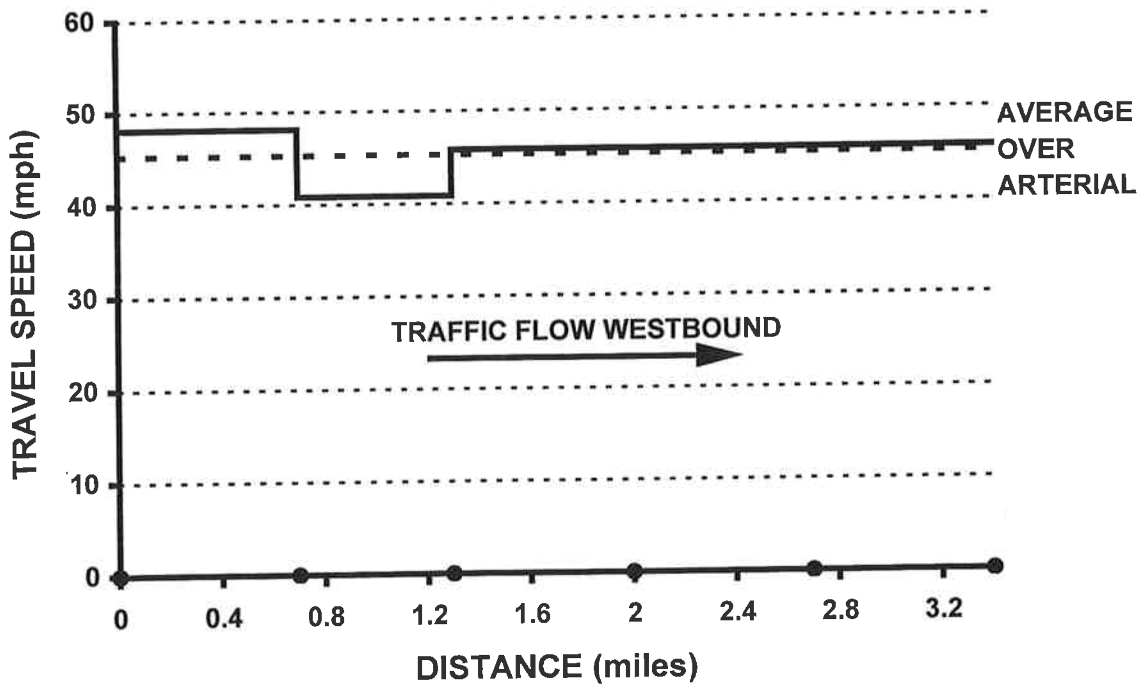

Figure 11-24. Speed profile for Calculation 7.

trolled, while Wright Street is all-way-stop controlled (AWSC). The signals are pretimed and coordinated with a 90-sec cycle length. The coordinated through movements have an arrival type of 4. Table 11-11 provides further information.

Determine the southbound arterial level of service by segment and for the entire facility. Do not aggregate the segments.

2. *Solution*: This is a slightly different problem because one of the intersections on the arterial is stop controlled. Neverthless, the methodology in Chapter 11 can be used, provided that the following conditions are met:

- The control delay at the stop-controlled intersection is calculated with the procedures presented in Chapter 10.
- Arrival Type 3 is used for the intersection directly downstream from the stop-controlled intersection. This should be done because stop control breaks up platoons and results in random arrivals downstream from the stop-controlled intersection.
- A stop-controlled intersection has a filtering effect similar to that of a traffic signal, and therefore the value of the filtering and metering factor (I) downstream from a stop-controlled intersection should be calculated as for a signal-controlled intersection.

TABLE 11-11. INPUT DATA FOR SAMPLE CALCULATION 10

INTERSECTION	SEGMENT	GREEN RATIO g/C	DEGREE OF SATURATION X	CAPACITY (VPH) c
Locust Street	1	0.411	0.403	1,628
Center Street	2	0.706	0.284	2,302
Wright Street	3	—[a]	0.880	749
North Avenue	4	0.467	0.396	1,655

[a] All-way-stop control.

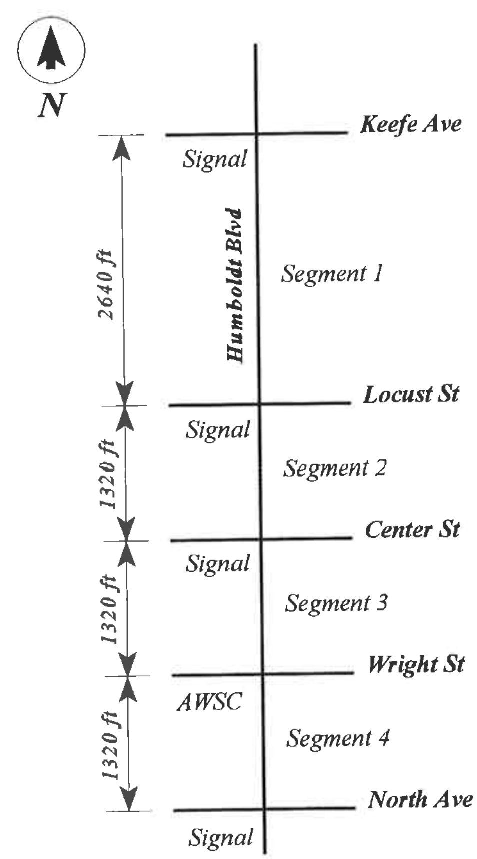

Figure 11-25. Arterial geometry for Calculation 10.

A summary of the arterial delay estimates is given in Figure 11-26. The value of I is selected from Table 11-8. The intersection control delay is calculated using Equation 11-2 for the signal-controlled intersection and Equation 10-1 for the stop-controlled intersection.

Based on the functional and design categories, free-flow speed, and intersection spacing, the arterial can be identified as belonging to Classification III. The segment running times are calculated as before from Table 11-4. Because the 0.50-mi length of Segment 1 exceeds the values given in Table 11-4, the free-flow speed is used to calculate the running time on the segment. The results of the LOS computations are shown in Figure 11-27.

3. *Discussion*: The intersection levels of service for Segments 1 to 4 are C, A, D, and B, respectively. Note that the same delay at signalized and stop-contolled intersections may represent different levels of service, because of the different LOS thresholds for signalized and stop-contolled intersections. From Figure 11-27 it can be seen that the stop-controlled intersection LOS is slightly worse than the arterial LOS. This difference is not surprising because all vehicles should stop at the stop-controlled intersection, whereas only a portion of all vehicles stop at signal-controlled intersections.

CALCULATION 11—TWO-LANE ARTERIAL

1. *Description*: Park Avenue is an undivided two-lane arterial with heavy traffic in both directions during the peak hour and relatively heavy turning volumes. The intersection spacing is given in Figure 11-28. Most of the arterial intersection approaches have exclusive left-turn lanes with permitted left-turn phasing. Several movements fail during the afternoon peak hour, when the signals operate on a 90-sec cycle. Table 11-12 provides information obtained from a capacity analysis of the individual intersections. Because of heavy side volumes, the through movements have an arrival type of 3. The free-flow speed on the facility is 30 mph.

Determine the arterial level of service in the eastbound direction by segment and for the entire facility. Do not aggregate the segments.

2. *Solution*: The facility can be classified as a minor arterial because it has a substantial access function. In terms of design, the arterial falls in the intermediate category because no parking is allowed along the facility, separate left-turn lanes are provided at most intersections, and four signals are found in approximately a mile. Minor arterials in the intermediate design category can be

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: *Humboldt Boulevard* — *South* - bound

File or Case # *Sample Calculation 10* — Date: *4/8/97*

Prepared by: ____

Segment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1^a \cdot PF^b$	Filtering/Metering Factor I^c	Incremental Delay d_2^d	Control Delay d^e (sec)	Through Movement LOS[f]
1	90	0.411	0.403	1,628	4	18.7	0.989	1.0	19.7	C
2	90	0.706	0.284	2,302	4	1.1	0.920	0.3	1.4	A
3	AWSC	-	0.880	749	-	-	-	-	28.0	D
4	90	0.467	0.396	1,655	3	12.8	1.000	0.7	13.5	B
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[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal place).
[f] Table 9-1.

Figure 11-26. Calculation 10, solution: using worksheet for summary of arterial intersection delay estimates.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE

Arterial: *Humboldt Boulevard* *South* - bound

File or Case # *Calculation 11* Date: *4/8/97*

Prepared by: *RJE*

$$ART\ SPD = \frac{3{,}600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Intersection Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	0.50	III	30	1	60.0	19.7	-	79.7	0.50	22.6	C
2	0.25	III	30	2	31.8	1.4	-	33.2	0.25	27.2	B
3	0.25	III	30	3	31.8	28.0	-	59.8	0.25	15.1	D
4	0.25	III	30	4	31.8	13.5	-	45.3	0.25	19.9	C
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[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = *216.2*

Grand Sum of Length (y) = *1.25*

$3{,}600 \times (y)/(x)$ = *20.8*

Arterial Level of Service = *C*

Figure 11-27. Calculation 10, solution: using worksheet for computation of arterial level of service.

identified as Classification III or IV. In this case Classification III was selected because of the relatively high density of access points.

A summary of the arterial delay estimates is given in Figure 11-29. The value of I is selected from Table 11-8. Park Avenue is a westbound one-way facility west of Atherton Street; therefore inflow to Segment 1 takes place only from the cross streets. Consequently, an I of 1.0 is assumed for Segment 1. The intersection delay is calculated using Equation 11-2. The segment running times are calculated from Table 11-4. For segment lengths outside the values given in Table 11-4, running times per mile are extrapolated. The results of the LOS computations are shown in Figure 11-30.

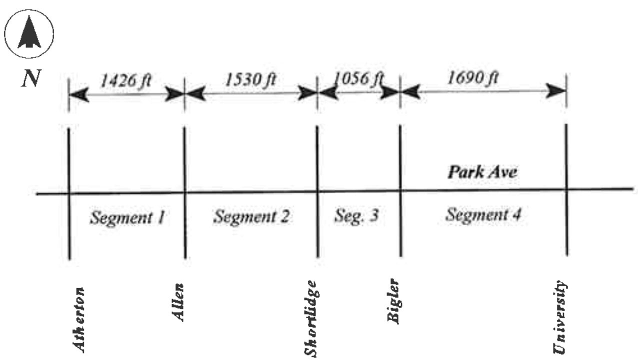

Figure 11-28. Arterial geometry for Calculation 11.

3. *Discussion*: As shown in Figure 11-30 and in the speed profile in Figure 11-31, the average speed during the peak hour along the arterial is 14 mph in the eastbound direction. The corresponding level of service is C, which is somewhat better than the level field observations indicate. A possible reason for this discrepancy may be that queue interaction occurs because of the high degrees of saturation. Queue interaction occurs whenever a downstream queue reduces saturation flow at an upstream intersection, thereby reducing capacity and increasing delay. In this case, the queue from the oversaturated eastbound approach at University Drive may extend back far enough to affect capacity at Bigler Street and intersections even further upstream. Queue interaction may also occur between left-turn movements and the adjacent through movements. Because high left-turn volumes are serviced by permitted phasing only, left-turn queues may spill out of the left-turn bays and block the through movements, reducing saturation and capacity and increasing delay.

This example illustrates the care required when analysing oversaturated conditions, because the methodology presented herein does not take into account queue interaction.

TABLE 11-12. INPUT DATA FOR SAMPLE CALCULATION 11

INTERSECTION	SEGMENT		EFFECTIVE GREEN TIME RATIO g/C		DEGREE OF SATURATION X		CAPACITY (VPH) c	
	EB	WB	EB	WB	EB	WB	EB	WB
Atherton	—	1	—	0.289	—	0.022	—	538
Allen	1	2	0.556	0.556	0.667	0.951	1,004	1,007
Shortlidge	2	3	0.633	0.467	0.906	0.977	1,037	869
Bigler	3	4	0.556	0.556	0.589	1.105	1,033	1,018
University	4	—	0.489	0.600	1.075	0.456	911	1,115

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: *Park Avenue* *East* - bound

File or Case # *Calculation 11* Date: *5/5/97*

Prepared by:

Seg-ment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1{}^a * PF^b$	Filtering/ Metering Factor I^c	Incre-mental Delay $d_2{}^d$	Control Delay d^e (sec)	Through Move-ment LOS[f]
1	*90*	*0.556*	*0.667*	*1,004*	*3*	*14.1*	*1.000*	*3.5*	*17.6*	*B*
2	*90*	*0.633*	*0.906*	*1,037*	*3*	*14.2*	*0.693*	*12.8*	*27.0*	*C*
3	*90*	*0.556*	*0.589*	*1,033*	*3*	*13.2*	*0.302*	*2.5*	*15.7*	*B*
4	*90*	*0.489*	*1.075*	*911*	*3*	*23.0*	*0.780*	*52.1*	*75.1*	*E*
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[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal place).
[f] Table 9-1.

Figure 11-29. Calculation 11, solution: using worksheet for summary of arterial intersection delay estimates.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE

Arterial: *Park Avenue* *East* - bound

File or Case # *Calculation 11* Date: *5/5/97*

Prepared by: *RJE*

$$ART\ SPD = \frac{3{,}600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Intersection Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1	0.27	III	30	1	35.1	19.7	-	54.8	0.27	17.7	C
2	0.29	III	30	2	37.1	27.0	-	64.1	0.29	16.3	C
3	0.20	III	30	3	28.0	15.7	-	43.7	0.20	16.5	C
4	0.32	III	30	4	40.3	75.1	-	115.4	0.32	10.0	D
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9											
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = 278.0

Grand Sum of Length (y) = 1.08

3,600 * (y)/(x) = 14.0

Arterial Level of Service = C

Figure 11-30. Calculation 11, solution: using worksheet for computation of arterial level of service.

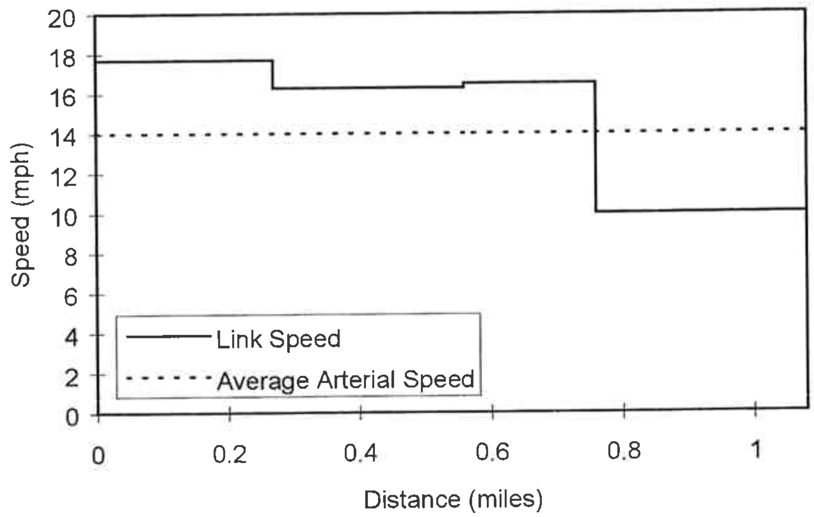

Figure 11-31. Speed profile for Calculation 11, eastbound traffic.

APPENDIX I

TEST-CAR METHOD FOR EXISTING ARTERIALS

The following steps are used when applying the test-car method for determining levels of service on existing urban and suburban arterials.

1. Identify and inventory the geometry and the access control of each arterial segment, the segment lengths, existing signal timing, and the 15-min flow rates for selected times of the day (such as the peak a.m. period, the peak p.m. period, and a representative off-peak period, by direction of flow).
2. Determine the appropriate free-flow speed for the arterial section being evaluated. For existing arterials, this speed may be determined by making runs with a test car equipped with a calibrated speedometer at times of low volumes. An observer should read the speedometer at midblock locations where the vehicle is not impeded by other vehicles, and readings should be recorded for each segment of an arterial. These observations may be supplemented by spot speed studies made at typical midblock locations during low-volume conditions. Other data, such as design type, access points, roadside development, and speed limit, may be considered also.
3. Use Tables 11-2 and 11-3 along with the physical information and free-flow speed already cited to determine the arterial classification.
4. Make test-car travel time runs over the arterial section during the selected times of the day.
   a. The observer should use appropriate measurement equipment to obtain the information specified in the travel time field worksheet in Appendix II. The equipment may be a computer-based collection system or a pair of stopwatches.
   b. Travel times between centers of signalized intersections should be recorded, along with the location, cause, and duration of each delay.
   c. Test-car runs should begin at different times in the signal cycle to avoid making all trips the first in a platoon.
   d. Some midblock speedometer readings should also be recorded to check on unimpeded travel speeds and to see how they relate to free-flow speed.
   e. Data should be summarized for each segment and time period to provide the average travel time and the average delay time for a signal and for other delays and events (four-way stops, parking disruptions, etc.).
   f. The minimum number of test-car runs depends on the variance in the data and the occurrence desired. Six to 12 runs for each traffic-volume condition may be adequate. (See *HRB Proc.*, 1952, pp. 864–866.)
   g. An instrumented test car should be used if available to reduce labor requirements and to facilitate recording and analysis. Computer-produced summaries of test-car runs, with all data recorded and analyzed by the computer, are now common.
5. For each segment and time period, the average travel speed should be determined by using travel times and segment lengths. Average travel speed for each arterial section should also be determined.
6. Table 11-1 should be used to obtain a LOS value for each arterial segment and for the overall arterial, for each time period and each direction of flow. This determination is made by comparing the average travel speed obtained in Step 5 with the speed values given in Table 11-1 for the appropriate arterial classification.
7. The test-car data can be modified to permit evaluation of different signal timing plans. As shown in Table 11-6, the adjustment factors can be applied to control delays to evaluate effects on control delay of changes in offsets. It is then possible to evaluate effects of these changes on average travel speeds and levels of service.

APPENDIX II

WORKSHEETS FOR USE IN ANALYSIS

WORKSHEETS **Page**

SUMMARY OF ARTERIAL INTERSECTION DELAY ESTIMATES

Arterial: ______________________ ______________________ - bound

File or Case #______________________ Date: ______________________

Prepared by: ______________________

Segment	Cycle Length C	Green Ratio g/C	v/c Ratio X	Lane Group Capacity c	Arrival Type AT	Uniform Delay $d_1^a \cdot PF^b$	Filtering/ Metering Factor I^c	Incremental Delay d_2^d	Control Delay d^e (sec)	Through Movement LOS[f]
1										
2										
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11										
12										
13										
14										
15										

[a] Equation 11-3.
[b] Table 11-6.
[c] Table 11-8.
[d] Equation 11-4.
[e] Equation 11-2 (round delay estimates to one decimal place).
[f] Table 9-1.

COMPUTATION OF ARTERIAL LEVEL OF SERVICE

Arterial: ____________ ____________ -bound

File or Case #____________ Date: ____________

Prepared by: ____________

$$ART\ SPD = \frac{3{,}600(sum\ of\ length)}{sum\ of\ time}$$

Segment	Length (mi)	Arterial Class	Free-Flow Speed (mph)	Section	Running Time[a] (sec)	Intersection Control Delay[b]	Other Delay (sec)	Sum of Time by Section	Sum of Length by Section	Arterial Speed[c] (mph)	Arterial LOS by Section
1											
2											
3											
4											
5											
6											
7											
8											
9											
10											
11											
12											
13											
14											
15											

[a] Use Table 11-4 and multiply by segment length.
[b] From worksheet for summary of arterial intersection delay estimates.
[c] See upper right corner of this worksheet for equation.
Note: Round delay estimates to one decimal place.

Grand Sum of Time (x) = ____________

Grand Sum of Length (y) = ____________

$3{,}600 \times (y)/(x)$ = ____________

Arterial Level of Service = ____________

TRAVEL TIME (TT) FIELD WORKSHEET

Arterial ______________________ Date ______________

Driver ______________ Recorder ______________ Direction ______________

Signal Location	*Distance (mi)*	*Run No. ___ Time* *Cumulative TT (sec)*		*Delay Time (sec)*	*Run No. ___ Time* *Cumulative TT (sec)*		*Delay Time (sec)*	*Run No. ___ Time* *Cumulative TT (sec)*		*Delay Time (sec)*

S — Signal (lower box)
LT — Left Turn (upper box)
P — Pedestrian
PK — Parking (upper box)
4W — 4-Way Stop (upper box)