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Case studies of three interchange improvement projects in San Antonio, Texas were conducted. One project was a new partial interchange. The other two projects consisted of several interchange improvement components, including:

1. Widening frontage road approaches,
2. Relocating exit ramps,
3. Signal upgrading, and
4. Constructing frontage road turnarounds.

These improvements were put into operation in stages, and delay and volume data were collected after each stage so that the operational effects attributable to each element of the project could be isolated. Signal upgrading was the most cost-effective improvement with benefit/cost ratios in the range of 40:1 to 50:1. Frontage road widening and ramp relocation had benefit/cost ratios from 5:1 to 10:1. The retrofit turnarounds had benefit/cost ratios from less than 1:1 to 2.6:1.

From the case studies, guidelines were developed for identifying and evaluating interchange improvements. The guidelines provide recommended methods for collecting necessary traffic data, matching candidate improvements to diagnosed operational problems, analyzing the operational impacts of proposed interchange modifications, and estimating the cost-effectiveness of such projects. The guidelines can be used to select the most effective set of improvements at a given interchange or to prioritize improvements among a group of interchanges.

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GUIDELINES FOR IDENTIFYING IMPROVEMENTS AT DIAMOND INTERCHANGES

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Research Study Number 2-18-84-423

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INTRODUCTION

Traffic growth and increased development along urban freeway corridors have increased traffic congestion and accident potential and have reduced operating speeds at frontage road/arterial intersections, at exit and entrance ramps, and on the main lanes of the freeway. Access problems have developed at certain locations as well. The Texas State Department of Highways and Public Transportation (SDHPT) has constructed a number of relatively low-cost improvements at freeway/arterial interchanges to relieve congestion at these critical points. The projects generally improve ramp locations or the capacity of the frontage road/arterial intersections. Typical modifications include:

1. Relocating exit ramps,
2. Grade-separated ("braided") ramps,
3. Adding downstream ramps,
4. Reversing ramps,
5. Constructing frontage road turnarounds,
6. Widening frontage roads, and
7. Improving traffic signal operations.

Previous evaluations have demonstrated that these relatively minor modifications to freeway/arterial interchanges can produce significant benefits to both interchanging and main lane traffic.

Each of these improvements address different operational problems at interchanges. The specific problems must be accurately identified and matched with the appropriate solution to maximize the efficiency and cost-effectiveness of minor interchange modifications. The various elements of diamond interchanges, particularly in urban areas, are operationally interrelated because of the close spacing between the frontage road/arterial intersections and because of the coordinated signal phasing at the intersections. Changes made to one frontage road or cross-street approach will impact operations not only on that approach but also on the other roadways. Therefore, proposed improvements must be carefully selected to avoid merely shifting the problem to another location.

Background

Previous studies in the Study 210 series have evaluated the primary and secondary user benefits resulting from specific cases of several of the above improvements. Report 210 - 11, Evaluation of Minor Freeway Modifications, described data collection techniques and before-and-after analysis methods for quantifying the road user benefits associated with these types of projects. It also included case studies of ramp additions, ramp relocations and grade - separated ramps, all in the San Antonio area (1). Report 210 - 12F, An Analysis of Urban Freeway Operations and Modifications, included evaluations of a ramp reversal project in Houston and grade-separated ramps in San Antonio, using basically the same analysis methods as the previous report (2).

The analyses yielded benefit/cost ratios ranging from slightly over 1:1 to 10:1, based on current traffic volumes. Future traffic growth may increase the effective ratios over the functional lives of the projects. Savings in running cost, travel time cost, and delay and idling costs constituted the vast majority of user benefits. Changes in accident costs had little or no impact on the economic analyses of the projects studied.

Study Purpose

This study completes the work begun under Study 210. It includes the following objectives:

1. To evaluate the benefits derived from constructing a new interchange within an urban freeway network.
2. To evaluate the benefits associated with the following components of an interchange improvement project:
    a. Widening frontage road approaches,
    b. Relocating ramps,
    c. Signal upgrading, and
    d. Constructing frontage road turnarounds.
3. To develop guidelines for selecting minor interchange improvements.
4. To develop preliminary capacity and level-of-service criteria for examining the operational efficiency of existing diamond interchanges.

Work on the first two objectives is documented in Appendices A through C of this report. Similar to Study 210, this work included evaluations of specific improvements based on actual data collected before and after the projects.

The major portion of this report is devoted to the third objective -- guidelines for selecting minor interchange improvements. The recommended guidelines discuss data collection techniques and analysis methods for examining existing diamond interchanges and selecting cost-effective improvement projects. The fourth objective -- preliminary capacity and level-of-service criteria -- is discussed as part of the recommended guidelines.

Case Study Projects

The Appendices of this report present detailed evaluations of three interchange improvement projects. All three of these SDHPT projects were located in San Antonio. The Eisenhauer Road at I-35 (Appendix A) and Rittiman Road at I-35 (Appendix B) projects were similar. These are adjacent interchanges along I-35 in northeast San Antonio, and the projects were built under a single construction contract. The work at both sites included frontage road widening, construction of a frontage road turnaround, and signal upgrading. The Rittiman Road project also included the relocation of both exit ramps. The primary purpose for studying both of these projects was to examine the operational effects of similar physical improvements under different traffic loadings. The Eisenhauer/I-35 interchange had moderately high traffic volumes and delay -- level-of-service D and E -- with relatively light U-turning volumes. The Rittiman interchange experienced heavy traffic volumes and lengthy delays -- level-of-service F -- with high U-turning volumes. At both sites the improvements were put into operation in stages so that the operational effects attributable to each element of the projects could be isolated.

The Terminal Drive at U.S. 281 project (Appendix C) was the construction of a new partial interchange and a frontage road extension. The purpose of the project was to provide more direct access to San Antonio International Airport. Thereby, drivers could avoid more circuitous routes to the airport and bypass the congested Airport Boulevard/I-410 interchange. A study area consisting of the project site and three interchanges likely to be affected by the project was selected, and traffic volume and operational changes were analyzed.

In all three case studies, vehicular delay was the basic measure of operational effectiveness. This is consistent with the relatively new "NCHRP Signalized Intersection Capacity Method," developed through the National Cooperative Highway Research Program for use in the revised Highway Capacity Manual (3). Along with volume counts and the percentage of vehicles stopped, delay provides a good basis for quantifying user costs and benefits. Except as noted otherwise, all references in this report to "vehicle delay" refer to "stopped delay."

The Eisenhauer/I-35 and Rittiman/I-35 projects were highly successful. At Eisenhauer, the peak hour levels-of-service were improved from the D/E range to level-of-service B, despite 10 to 15 percent growth in peak hour traffic. The incremental evaluation revealed that the signal upgrading cut delay more than any other single element of the project at this site. The cost-effectiveness of each component of the Eisenhauer project, based on current traffic volumes, is summarized in Table 1.

TABLE 1. SUMMARY OF ECONOMIC ANALYSIS
EISENHAUER ROAD AT I-35

Improvement	Benefit/Cost Ratio
Frontage Road Widening	4.7:1
Signal Upgrading	52.9:1
Turnaround	0.2:1
Total Project	5.2:1

At Rittiman, the peak hour levels-of-service were improved from F to D. There, the frontage road widening and ramp relocations produced the largest increment of operational improvement. The cost-effectiveness of each component of the Rittiman project, based on current traffic volumes, is summarized in Table 2.

TABLE 2. SUMMARY OF ECONOMIC ANALYSIS
RITTIMAN ROAD AT I-35

Improvement	Benefit/Cost Ratio
Ramp Relocation, Auxiliary Lane, and Frontage Road Widening	9.4:1
Signal Upgrading	38.7:1
Turnaround	2.6:1
Total Project	7.6:1

The Terminal Drive/U.S. 281 project did not show as positive results as did the other projects. Some traffic was diverted from the congested Airport/I-410 interchange as intended. However, a number of drivers began using the new Terminal/Airport connection as a routing from southbound U.S. 281 to eastbound I-410. This routing more than offset the intended diversion of traffic from the Airport/I-410 interchange. Based on current traffic volumes, the calculated benefit/cost ratio is about 0.7:1. However, the new facility is operating well under capacity, and as traffic volumes grow, the delay savings will continue to increase as more traffic uses the interchange.

From Case Studies To Guidelines

The case studies in this report are similar to the Study 210 evaluations. The before-and-after evaluations are based on actual operations data collected before and after the improvement. On a broad level, these evaluations have confirmed that minor interchange modifications are a viable class of projects. They have been found to be generally cost-effective measures and, at least in the interim, to be solutions to capacity bottlenecks.

However, because of funding and personnel constraints, the ability to develop, select, and prioritize specific interchange improvement alternatives is crucial. In practice, of course, data representing actual conditions after construction are not available when considering alternative improvements. The purpose of the Recommended Guidelines, which follow, is to serve as an outline for identifying and evaluating potential minor interchange improvements.

The Recommended Guidelines are basically an organized collection of the techniques used and the lessons learned by the researchers during the case studies. It should be noted that the case studies were all projects in San Antonio. Differences in typical traffic characteristics or geometric conditions in other metropolitan areas may require different evaluation techniques or produce different results. However, the overall framework of evaluation should be applicable in any setting.

RECOMMENDED GUIDELINES

Summary of Approach

Figure 1 provides an overview of the recommended approach for identifying and evaluating minor interchange improvements. The procedure includes two major phases: a preliminary screening and a detailed analysis.

The first step of the preliminary screening is to identify those interchanges with operational problems, based on general observation of various operating characteristics. Preliminary data, consisting of approach volumes and average vehicular delay, are then collected at locations with observed problems. The delay data provide a good measure of the level of service for the interchange and help identify problems on specific approaches to the interchange. The screening analysis continues by diagnosing the detailed operational deficiencies which are causing the visible symptoms. Improvement opportunities are then systematically examined to determine which ones are potential solutions to the observed problems.

The detailed analysis evaluates the operational impacts and cost-effectiveness of the most-promising improvement alternatives. Depending on the specific case, additional detailed data may be required including turning movement counts, lane volume counts, or signal timing data. This information is used to run PASSER III, a computer model which can be used to analyze not only signal timing plans but also physical improvement projects. The operational improvements calculated by PASSER III then are converted to user benefits which, compared to estimated construction cost, provide a measure of cost-effectiveness for alternative improvements.

This evaluation procedure can be used to select the most cost-effective set of improvements at a given interchange. In addition it can serve as a guide for prioritizing improvements among a group of interchanges. It may not

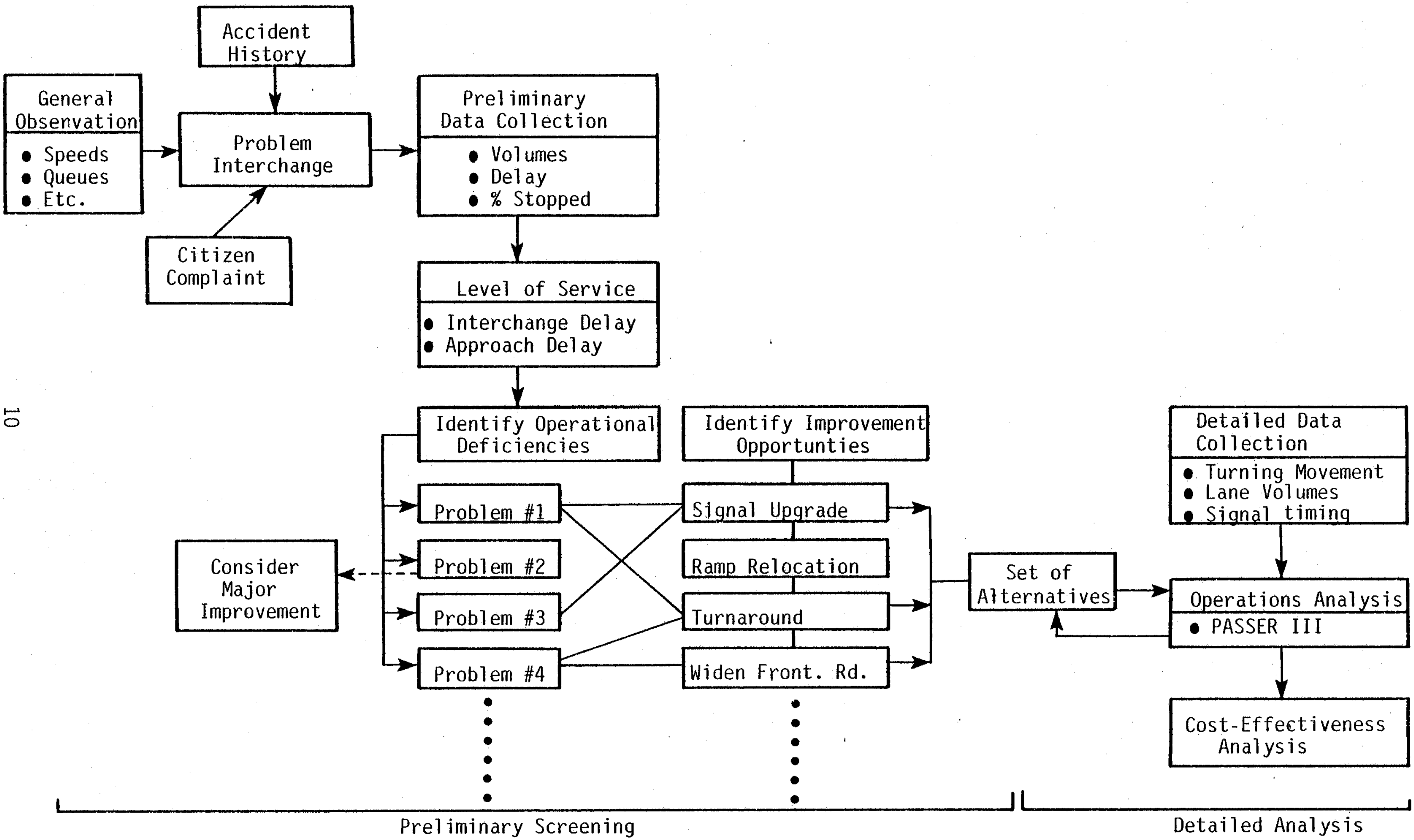


Figure 1. Overview of Recommend Approach - Identifying and Evaluating Minor Interchange Improvements

be necessary to apply the procedure at the same level of detail in all cases. However, three critical elements should be included:

1. Collecting current volume and delay data,
2. Identifying the specific operational deficiencies, and
3. Systematically matching candidate improvements to the identified problems.

Preliminary Screening

The first step in all evaluations should be a preliminary screening of problems and candidate improvements. This screening will help focus on the most promising alternatives. Once this step is completed, a more detailed analysis of the magnitude of the problem and the cost-effectiveness of the improvement can be undertaken.

Data Collection

At this point data collection should be limited to that necessary to obtain a general indication of the nature and magnitude of the problem.

a. General Observation -- This aspect of the evaluation may be the most important. At this stage the analyst should obtain a good on-site "feel" for operations and the nature of the problem. Many problems that are obvious to the informed observer may be completely masked in traffic volume or delay data. With some professional insight into the actual operation, more specific data will be highly useful. The kinds of operating characteristics that could indicate a problem include the following:

- Reduced freeway speeds -- Significant changes in freeway speed may be the result of friction between the main lanes and a ramp or auxiliary lane. Changes on the order of 10 miles per hour or more should be investigated. A slowing upstream of an exit, after which speeds increase, may indicate inadequate ramp/frontage road capacity or poor ramp terminal design. Slowing between an entrance and an exit may be the result of a weaving problem.

- Frontage road/crossroad queues -- Long queues at the intersection may be inevitable. However, they may be indicators of areas of potential improvement. Vehicles queued for a heavy through movement on the frontage road indicate inadequate downstream access. Very long queues may indicate that there is simply insufficient storage space on the frontage road.

- Lane Distribution -- This characteristic is a good indicator of problems in lane assignment. Long queues in the left or right lanes may mean that the turn capacity is insufficient. In some extreme cases, poor lane distribution can be a result of inadequate storage space. As was observed in a previous study, a left turn queue blocking a ramp prevented through and right-turning traffic from reaching the intersection.

Having surveyed the general operational characteristics, the analyst should have a good sense of which interchanges have symptoms of operating deficiencies. Other sources of information which can aid in or confirm this determination include accident histories and complaints or comments received from the public or from police agencies.

b. Preliminary Data Collection-- Once an interchange is identified as having operational problems a small set of data can be collected, at a relatively low level of effort, which are helpful in diagnosing the specific deficiencies. The data to be collected at this stage include:

- Approach volumes on the four exterior approaches,
- Vehicle delay on the exterior approaches, and
- Percent of vehicles stopping.

Figure 2 presents an example data form used in the collection of this data. The form was developed for use with a delay measurement technique known as the point sample method. Reilly's report, A Technique of Measurement of Delay at Intersections (4), describes the method in detail. At 15-second intervals, the number of stopped vehicles on an approach is counted and recorded. If the signal cycle length is an integer multiple of 15 seconds (e.g. 60 or 75 seconds), then an alternative fixed interval in the range of 13 to 15 seconds

Intersection Delay Study

Location: RITTIMAN @ I-35 — Date: 2 / 22 / 84

Approach: EB RITTIMAN — Day of Week: WEDNESDAY

Time Interval: 15 SEC — Study Period: 4:45 - 5:45 PM

57	15	3	5	20	21	22	30	40	52	265
39	0	0	7	11	24	23	29	35	40	213
24	0	0	6	17	21	25	27	31	15	166
0	1	6	10	16	22	29	14	5	0	103
8	10	13	19	24	15	4	0	6	10	109
11	15	16	18	2	0	2	3	10	11	90
12	21	2	0	0	7	3	12	15	0	77
0	0	9	11	12	6	0	0	4	5	47
6	12	22	22	34	41	25	10	4	9	185
10	27	34	34	32	30	10	2	8	12	199
14	27	30	33	45	24	8	6	11	19	222
23	27	33	39	39	21	0	4	10	27	223
30	35	51	51	55	25	10	8	15	29	308
33	39	38	25	10	0	9	10	21	24	209
24	29	30	33	21	18	8	1	16	23	205
29	32	39	42	42	31	3	0	9	12	244
22	28	32	34	34	15	0	0	5	12	182
15	17	22	8	0	0	1	5	7	10	85
12	0	1	2	8	9	5	8	0	0	45
2	3	5	6	0	0	1	4	13	6	40
0	0	0	3	0	0	1	6	0	0	10
2	5	8	10	13	1	0	3	9	13	64
13	10	0	0	4	6	6	7	0	2	53
6	10	11	12	3	0	1	2	3	11	59
										3403

Total Stopped Vehicles: 820 — % Delayed Vehicles = 81.8%

Total Vehicles: 1003 — Vehicle-Seconds of Delay = 51,045

Average Delay = 51 SEC.

Figure 2. Point Sample Method of Stopped Delay Measurement - Example Data Collection Form

is used so that the queue counts are distributed throughout the cycle length. The stopped vehicle counts are summed for the data collection period, and this total is multiplied by the interval length (15 seconds or the alternative) to estimate the total vehicle-seconds of stopped delay on that approach during the analysis period. Dividing the total vehicle-seconds by the approach volume yields the average stopped delay per approach vehicle.

Two or three observers are required to conduct this study. The total approach volume and the total stopped volume are recorded in addition to the 15-second counts. The point sample method is more efficient than other, more detailed, delay measurement techniques such as the input-output method. The data reduction time for the point sample method is approximately one-eighth that of the time required to reduce input-output data. When the two techniques were compared for the same sample, in previous studies, the point sample method produced delay estimates within six percent of the input-output method.

Screening Analysis

Combining the measured vehicular delay on the approaches with the general observation of operating symptoms provides a good basis for diagnosing the specific operational deficiencies of the diamond interchange.

a. Level of Service -- The concept of level of service at signalized intersections is undergoing a change. The critical movement analysis included in Transportation Research Circular No. 212, "Interim Materials on Highway Capacity," (5) has been widely used and accepted over the past several years. It is particularly useful in design applications for determining the number of lanes required. Appendix D-100 of the SDHPT Operations and Procedures Manual (6) includes an adaptation of the critical movement concept for diamond interchanges. It defines levels of service as follows:

Level of Service	Critical Lane Volumes
A	≤ 1250 vph
B	≤ 1300 vph
C	≤ 1350 vph
D	≤ 1480 vph
E	≤ 1650 vph

The emerging concept for signalized intersections defines level of service in terms of the average delay per vehicle. This is the approach taken in the relatively new "NCHRP Signalized Intersection Capacity Method," developed through the National Cooperative Highway Research Program for use in the revised Highway Capacity Manual. This approach is similar to the driver's viewpoint in characterizing intersection operation in terms of the delay experienced -- regardless of the vehicular volume present. It is more useful for analyzing operations at existing intersections. Levels of service based on average delay per vehicle have been defined (7).

Level of Service	Average Total Delay	Average Stopped Delay
A	≤ 16 seconds	≤ 12 seconds
B	≤ 22 seconds	≤ 17 seconds
C	≤ 28 seconds	≤ 22 seconds
D	≤ 35 seconds	≤ 27 seconds
E	≤ 40 seconds	≤ 31 seconds
F	> 40 seconds	> 31 seconds

• Interchange Efficiency -- The preliminary data collected enables the analyst to calculate the existing interchange level of service using both methods. The levels of service calculated by the two methods cannot be directly correlated. However, comparing the critical movement volume with the measured average delay can provide a conceptual notion of the operational efficiency of the interchange.

Figure 3 illustrates this concept. It is only a tool for first-cut screening to identify interchanges which warrant additional analysis. The line dividing "relatively efficient" operation from "relatively inefficient" operation is merely a plot of the alphabetic level of service limits by average delay and by critical lane volume. For example, the line passes through 1350 vehicles per hour (the upper limit of level of service C by the critical movement criteria) at 22 seconds delay (the upper limit of level of service C based on stopped delay). An interchange which falls above this line is actually operating at a lower level of service based on delay than one would expect based on the critical lane volume. That is, the actual delay is higher than expected, given the traffic volume and number of lanes available. Interchanges plotting below the line are doing a relatively good job from a delay viewpoint, given the existing volumes and laneage.

Toward both ends of the plot, practical considerations dictate. A horizontal line is drawn at 12 seconds delay in the low-volume area. This indicates that interchanges with average delay less than 12 seconds probably are not improvement priorities, regardless of their "relative efficiencies". Likewise, a high-volume line is drawn at 31 seconds delay. Even though an interchange with more than 31 seconds average stopped delay may be "relatively efficient" based on the critical lane volume, the absolute delay may be

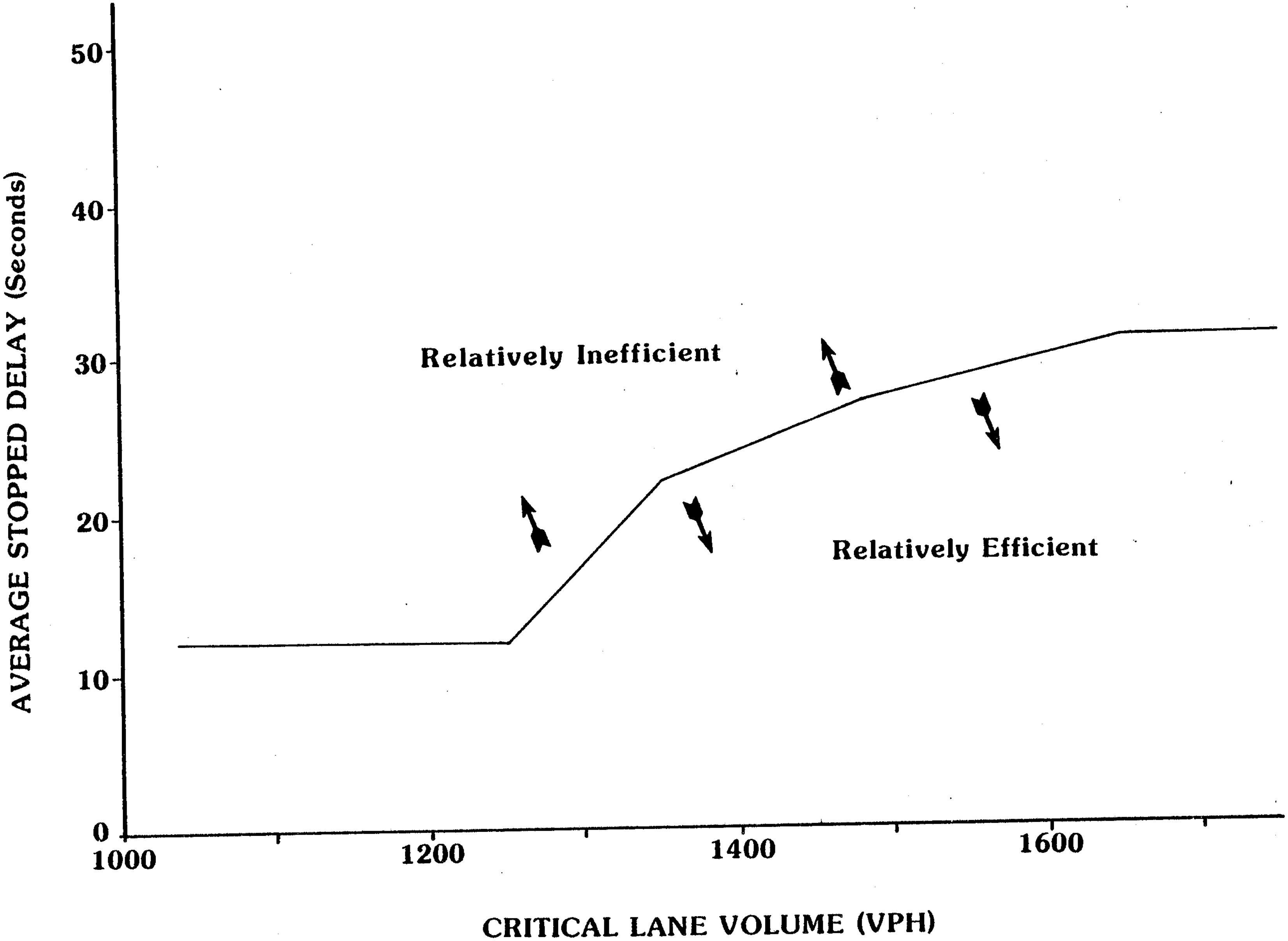


Figure 3. Delay vs. Volume - Relative Interchange Efficiency

considered unacceptable and the interchange considered a high priority for improvement.

Figure 3 should be considered only a rule-of-thumb. An exact line dividing the efficient and inefficient zones does not exist. Furthermore, specific site conditions may preclude achieving "relatively efficient" operations at a given location. However example applications of this concept are shown in Figures 4 and 5 for the case study sites discussed in Appendices A and B. The "After 1," "After 2," and "After 3" points refer to interchange operation after each increment of the improvement projects. The figures show that the projects were successful in significantly improving the relative efficiency of both interchanges.

- Approach Efficiency -- Another use of the delay data in the screening analysis is to focus attention on specific portions of the interchange. An example of this analysis is discussed in Appendix A -- the case study of improvements to the Eisenhauer/I-35 interchange. At that site, total traffic volumes on the exterior approaches to the west intersection (eastbound Eisenhauer and southbound frontage road) were less than eastside volumes during both peak periods. Although volumes were lower, the average delay was higher and a higher percentage of vehicles was stopped. This suggested that improvements to the west intersection would be the most effective.

The presence of similar characteristics should be investigated at all interchanges to focus attention on the least efficient approaches. Of course, the impacts throughout the interchange of improvements to one approach should not be overlooked.

b. Improvement Opportunities

The process of identifying a potentially successful improvement alternative is controlled largely by the successful identification of the probable

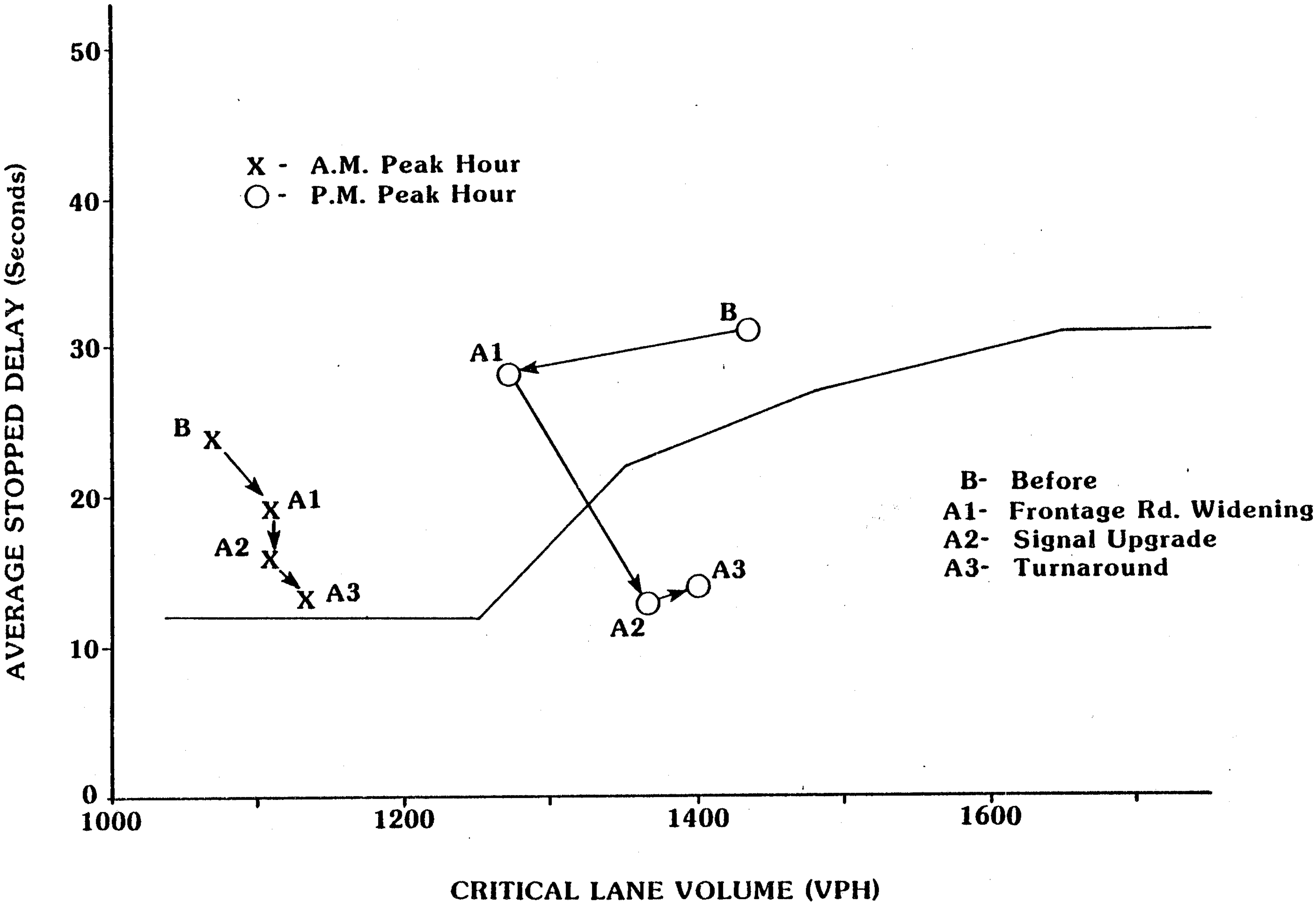


Figure 4. Delay vs. Volume - Eisenhauer Rd. at I-35

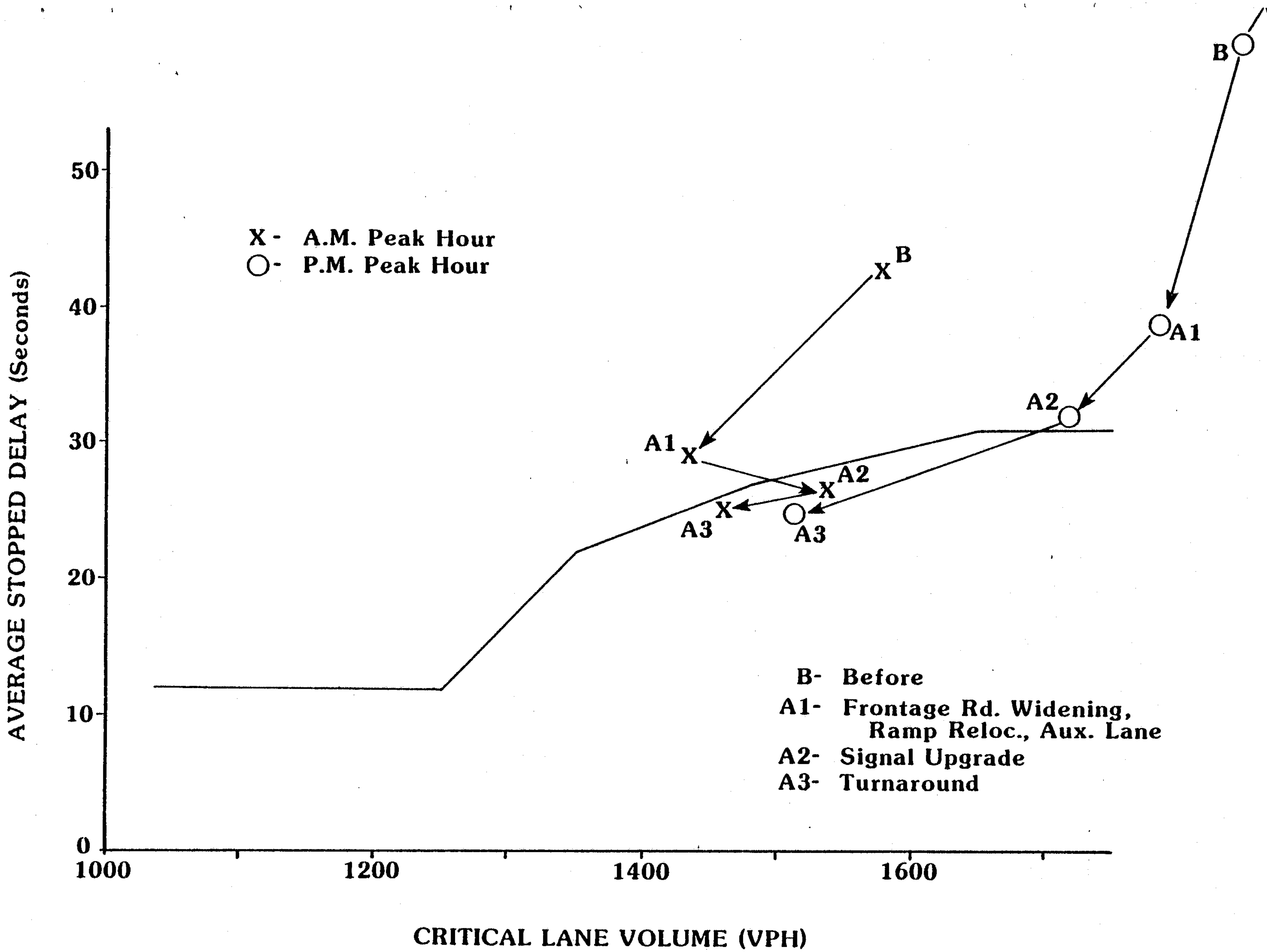


Figure 5. Delay vs. Volume - Rittiman Rd. at I-35

cause. Several typical "symptoms" and their possible causes are discussed in the following paragraphs. Along with each "cause" are listed data for verifying that "cause" and potential improvements. If the user identifies a different symptom from those listed, the same process may be followed by rationalizing which probable causes may apply.

All of these guidelines presume that the signal timing is near optimum. Improvements that primarily reduce delay can be seriously affected by signal operation. Careful attention should be directed toward this aspect of operation before more expensive alternatives are considered. For example, a signal upgrade (new controller and detectors) at an interchange in San Antonio produced enough benefits to pay for the cost of the improvements in a single year. <u>Therefore, considering the relative costs of signal improvements and geometric modifications, it is highly recommended that all possible signal alternatives be considered first</u>.

<u>Symptom: Frontage Road Queuing onto Main Lanes</u>

One of the most obvious and potentially hazardous conditions is a queue extending from the frontage road/arterial intersection, along the frontage road and exit ramp, to the main lanes. As indicated in Table 3, major queues may be the result of any one of several causes. There is no particular significance to the order listed.

<u>Left Turn/Right Turn Capacity</u> -- This type deficiency will most often be evident in long queues on the same side of the frontage road as the turn lane. A turning movement count and a count (or estimate) of lane distribution (number of vehicles using each approach lane) will indicate if the volumes are fairly balanced. If not, dual turning movements (particularly to the left) or a separate turning roadway (to the right) could be considered. Widening the

TABLE 3. IMPROVEMENT ALTERNATIVES FOR TYPICAL OPERATIONAL PROBLEMS

SYMPTOM	POSSIBLE CAUSES	DIAGNOSTIC DATA	IMPROVEMENT ALTERNATIVES
Frontage Road Queuing onto Main Lanes	• Left turn or right turn capacity	• Turning movement count • Excess capacity for some movements	• Reassign lane use
		• No excess capacity	• Widen frontage road
	• Frontage road storage	• Lane distribution • Balanced	• Widen frontage road
		• Unbalanced	• Relocate or reverse ramp
	• U-turn capacity	• Turning movement count	• Add turnaround
	• Turnaround operation	• Observation • Turning movement count	• Improve approach lane to turnaround • Improve merge at turnaround exit
	• Downstream demand	• Turning movement count	• Reverse ramp(s) • Add downstream ramp
Excessive Intersection Delay	• Approach capacity	• Turning movement count	• Widen frontage road
	• Lane use	• Turning movement count	• Widen frontage road • Dual left turns • Exclusive right turn
	• Access patterns	• License plate survey	• Relocate or reverse ramp
	• U-turn capacity	• Turning movement count	• Add turnaround
	• U-turn operation	• Observation • Turning movement count	• Improve approach lane to turnaround • Improve merge at turnaround exit
	• Downstream demand	• Turning movement count	• Reverse ramp(s) • Add downstream ramp
Reduced Main Lane Speeds	• Ramp capacity	• Ramp speed • Ramp volume • High volume/low speed	• Two-lane ramp
		• Low volume/low speed	• Improve ramp geometry
	• Weaving capacity	• Speed profile • Ramp/mainlane volumes	• Reverse ramp(s) • Add auxiliary lane

approach could be considered, primarily to improve the capacity of the above mentioned turning movements.

Frontage Road Storage -- This deficiency may be difficult to distinguish from others. Single-lane queues (usually left turn) can reach the exit and prevent full utilization of the frontage road storage capacity. In this case, left turn capacity improvements should be investigated before it is concluded that the frontage road storage is deficient. However, storage may be fairly uniform across the lanes. If queuing is uniform, then either widening or ramp relocation (or reversal) may be appropriate. Approaches with three or more lanes (excluding turnaround approach) may be difficult to widen. Two lane approaches are good candidates for widening. Where approach lane usage is fairly uniform, widening increases the flow rate over the stop bar, and thus is likely to reduce overall delay.

If storage is not uniform, then widening may not be helpful. If queues onto the exit are preventing full use of frontage road approach capacity, then relocation of the exit ramp is suggested. For this situation, the decision to relocate or reverse ramps should be based on ramp spacing and/or main lane conditions.

U-Turn Capacity -- The construction of a turnaround will significantly reduce the delay experienced by U-turning traffic. If the volume of U-turning traffic is high enough, their removal from the intersection may provide significant delay reductions for other traffic as well. A turnaround can also be justified based on direct savings to U-turning traffic alone.

Turnaround Usage -- Queues may develop partially as a result of the inability of U-turning traffic to access an existing turnaround. On-site observation will establish the need for improvement. If, during peak periods, U-turning vehicles are blocked from the entrance by other vehicles, then the approach lane should be extended.

Downstream Demand -- Crossing arterials that have no direct access from a ramp are the locations where this problem is most likely to occur. A turning movement count at the upstream intersection will provide a strong indication of the value of the improvement. If there is a heavy through movement on the upstream frontage road approach, and if a sizeable portion of that traffic is destined for the downstream intersection, then the addition of a ramp should be considered.

If there is a heavy through movement that is bound for some major generator (e.g., residential area, office complex, etc.) between arterials, then a ramp reversal should be considered.

Symptom: Excessive Intersection Delay

Although there is some overlap between this symptom and the excessive queuing discussed in the previous section, this symptom refers primarily to approaches where measured average delay is unacceptable, but the resulting queues do not interfere with operations on the ramp or the main lanes. Table 3 includes causes and potential improvements for delay problems.

Insufficient Approach Capacity -- This deficiency produces excessive delay across most or all movements on the approach. A turning movement count will establish per lane volumes and capacity deficiencies for specific movements. Signal timing improvements should be investigated first to ensure that phase splits are responsive to current approach volumes. The volume per lane can be compared on all the exterior approaches to focus the improvement alternatives on critical approaches. However, the impacts throughout the interchange due to improvements on one approach must be considered.

Widening the frontage road will provide additional capacity, with lanes assigned to best accommodate turn movement proportions. Dual left turn lanes or an exclusive right turn lane may be required to meet turning volume demand.

Unbalanced Lane Use -- This problem is probably due to heavy turning volumes, which will be detected through a turning movement count. Depending on the existing laneage, the suggested improvement may be widening the frontage road to provide dual left turn lanes or adding an exclusive, free-flowing right turn lane.

Access Patterns -- Some traffic may be routed unnecessarily through the frontage road/arterial intersection to gain access to adjacent land uses or the freeway. This concept is shown in Figure 6. In the "before" condition, drivers desiring access to the residential area adjacent to the frontage road are required to go through the downstream arterial intersection because of a lack of access opportunities between the exit ramp and the intersection. This route not only increases user costs for these motorists, but also indirectly impacts all of the other users at the intersection. A similar situation is shown for the movement from the residential area to the freeway.

A license plate survey may be required to verify these traffic patterns. If significant volumes are making these maneuvers, the ramp relocation should be considered to improve the access function of the frontage road. In the "after" condition shown in Figure 6, drivers are afforded considerably improved access and are able to avoid the arterial intersection.

Changes in access also have potentially negative impacts. Unwanted traffic volumes on local streets may be increased. If, after ramp relocation, the residential streets would be an attractive short cut to avoid a signalized intersection, then the overall impact of the ramp relocation should be reconsidered.

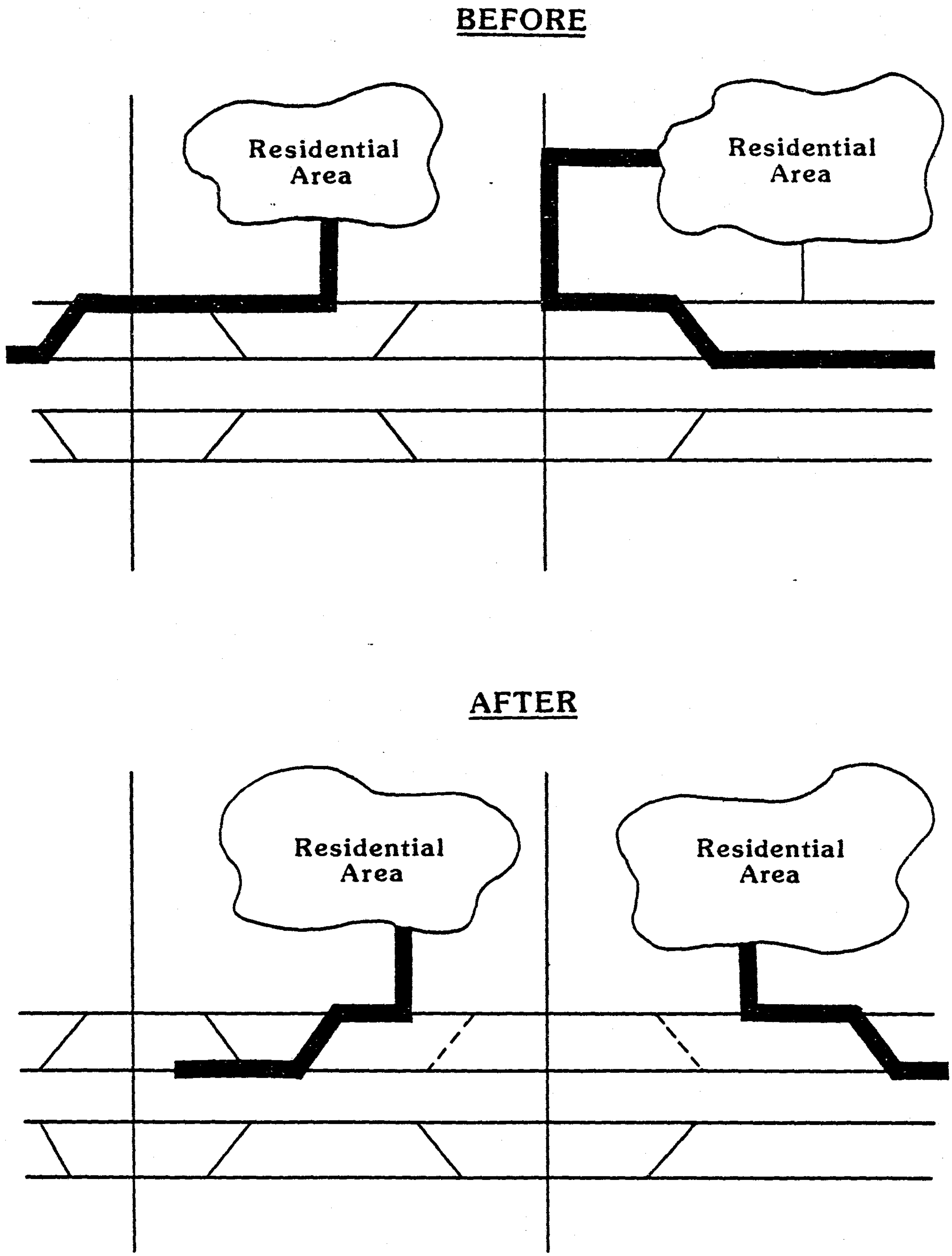


Figure 6. Schematic Diagram of Interchange Access Problems

Inadequate U-Turn Capacity -- This the same problem discussed in the "Queues onto Main Lanes" section. See that section for diagnosing the problem and for recommended improvements.

Ineffective Turnaround Operations -- This is the same problem discussed in the "Queues onto Main Lanes" section. See that section for diagnosing the problem and for recommended improvements.

Excessive Downstream Demand -- This is the same problem discussed in the "Queues onto Main Lanes" section. See that section for diagnosing the problem and for recommended improvements.

Symptom: Reduced Main Lane Speeds

There is some overlap between the influence area of this symptom and the excessive queuing discussed in an earlier section. However, this symptom refers to impacts on mainlane operation which emanate from problems created by the ramp's operation or location per se. The earlier section dealt with problems created by queues originating at the frontage road/arterial intersection.

Ramp Capacity -- This type of inadequacy may be characterized by very high ramp volume (approaching 2000 vehicles per hour), low ramp speed or both. When these two conditions are present together, it may be appropriate to consider a two-lane ramp, provided that the approach capacity of the downstream intersection is at least 2000 vehicles per hour.

Low speed operations on the ramp without high volumes may suggest that ramp geometry needs to be improved. Significant curves at either end of the ramp can produce reduced-speed operations.

Caution should be exercised to insure that reduced ramp capacity is not confused with other conditions. Deficient ramp capacity will result in queues forming on or upstream of the ramp -- not on the frontage road.

Weaving Capacity -- This problem is the result of inadequate weaving length given the exiting and through volumes. This length and the weaving volume are functions of the location and configuration of upstream ramps and of travel patterns. This condition can be detected by measuring main lane speeds in advance of and through the interchange to develop speed profiles for each lane. Ramp and mainlane volumes are needed for a detailed weaving analysis.

If this problem is present alone -- without a ramp capacity problem (discussed previously) -- then the speed reduction may occur only on the main lanes, and exiting vehicles will experience smoother operation once they have reached the ramp.

The improvement alternatives recommended for a weaving problem are reversing the ramp(s) or adding an auxiliary lane. The advisability of reversing a ramp or a pair of ramps should be examined carefully, considering access patterns in the area and impacts on the upstream and downstream frontage road/arterial intersections. If the analysis of access and traffic patterns indicates that ramp reversal would not be effective, then an auxiliary lane upstream of the exit ramp should be considered. An auxiliary lane does not affect the overall weaving volumes, but it reduces the weaving friction effects on main lane operations.

Detailed Analysis

At the completion of the screening analysis, a preliminary set of alternative interchange improvements has been identified. The detailed analysis examines the operational effectiveness of each alternative and provides information for selecting the most cost-effective improvement.

Data Collection

Additional data beyond the preliminary approach volume and delay data are needed to accurately diagnose the probable cause of operational symptoms (as discussed in the previous section) and as input to the detailed operational analysis (as discussed in the following section). Depending on the specific problems present, some or all of the following data items may be needed.

- Turning movement count -- The turning movement proportions at urban interchanges can change fairly rapidly due to land development changes in the vicinity. Therefore, a current count is essential. The PASSER III model, discussed later, requires volumes for each of the 14 vehicle movements possible from the four exterior approaches. These movements are shown on the PASSER III coding form included as Figure 8. Counts should be taken during both the morning and evening peak periods.

- Lane distribution -- Traffic volumes in each approach lane may be needed to select the proper remedy for frontage road storage problems. Counts of each lane may be used to more accurately model the interchange with PASSER III. Again, morning and evening peak period counts should be taken.

- Traffic signal characteristics -- The average cycle length and phase splits resulting from traffic-actuated signal operation should be measured in the field. These values may be used as input to PASSER III to replicate existing conditions. This serves as a basis for evaluating the operational effects of alternative improvements. Cycle length and splits should be timed during both peak periods. Wide variations in cycle length and splits during the peak periods also should be noted. This variability may influence the accuracy of PASSER III results.

The above items include those needed in most circumstances to accurately diagnose the operational problems and to evaluate improvements. In specific cases other items may be required including ramp or mainlane speed profiles and detailed travel pattern data. Use of these items as diagnostic indicators was discussed in the previous section. Standard study techniques for obtaining these data have been developed and documented (8).

Operations Analysis

The most appropriate tool for conducting detailed analyses of signalized diamond interchange modifications is PASSER III. PASSER is an acronym for Progression Analysis and Signal System Evaluation Routine. PASSER III was developed to assist traffic engineers in determining optimal traffic signal timings for signalized diamond interchanges. It can be used to analyze isolated interchanges as well as a series of interchanges through which frontage road signal progression is desired. The user's manual for PASSER III contains a detailed discussion of the program and its operation (9).

The isolated interchange analysis is the portion of PASSER III discussed in these guidelines. The program can evaluate any signal timing plan at a signalized diamond interchange. All the basic signal phasing sequences, known to PASSER III as phasing codes, can be analyzed, including all combinations of leading and lagging left turn phases. A special case of the lead-lead sequence -- the popular four-phase, two-overlap phasing -- is given a separate phasing code with a different procedure for calculating green splits. Given a cycle length, PASSER III calculates green splits for each movement based on the ratios of approach movement volume to signal capacity. The internal offset -- the time relationship between the two signalized intersections in an interchange -- is analyzed by a deterministic delay-offset technique to calculate delay on the interior approaches. Along with estimates of delay on the exterior approaches calculated using Webster's equation, this provides delay estimates for the entire interchange.

PASSER III provides several measures of effectiveness (MOE's) for each movement at the interchange's two intersections.

1. Ratio of movement volume to signal capacity -- the X-ratio -- for the critical lane of an approach.

2. Average vehicular delay in seconds per vehicle.

3. Probability of clearing the queue for the critical lane.

4. Ratio of queue length to available storage length for interior movements.

Three separate alphabetic levels-of-service (A through F) are assigned to each movement based on the first three of these MOE's. The estimate of vehicle delay is emphasized in these guidelines and provides a basis for the economic analysis. However, examining all the MOE's gives a more complete overview of interchange operations.

The following discussion highlights items for special attention when using PASSER III to analyze the minor interchange modifications included in this report. This summary is useful only as a supplement to the detailed instructions found in the PASSER III user's manual.

a. Passer III Input -- The input data requirements and format for using PASSER III to analyze an isolated interchange are relatively straightforward. Input coding sheets, which are shown as Figures 7 and 8, are available to simplify data input. The data are organized on three basic types of cards:

1. Freeway Header Card (one per freeway),

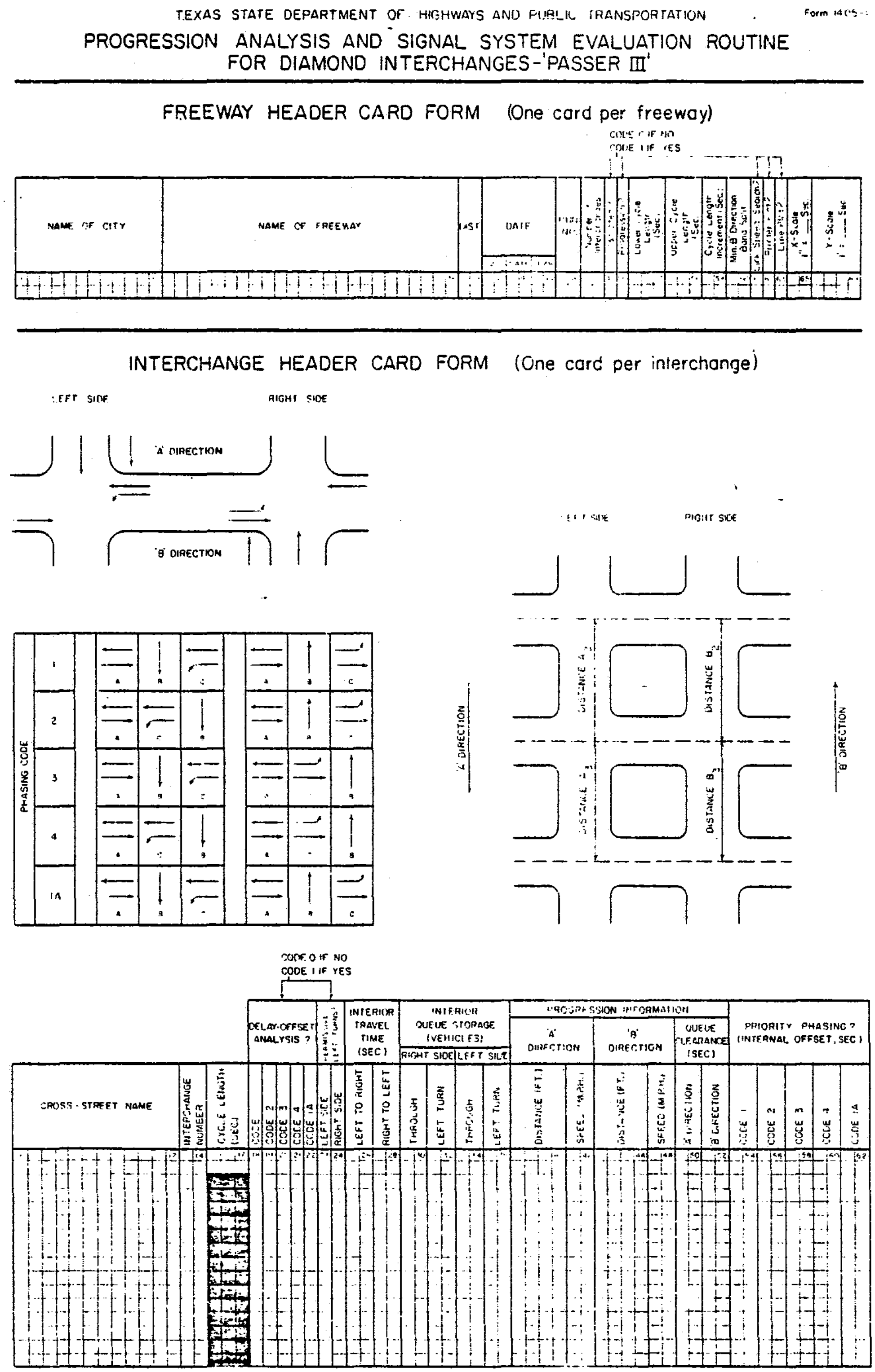
2. Interchange Header Card (one per interchange), and

3. Interchange Detail Card (three per interchange).

Therefore, five cards (i.e., lines) of data are required to analyze an isolated interchange.

For a single interchange analysis, the Freeway Header Card contains only information for the titles used in the output. Ones (1's) are entered in Columns 48 and 49, and the remainder of the card, which deals with frontage road progression, is left blank.

The Interchange Header Card contains most of the signal phasing and timing information. The cycle length (Columns 15-17) must be supplied and is



TEXAS STATE DEPARTMENT OF HIGHWAYS AND PUBLIC TRANSPORTATION

PROGRESSION ANALYSIS AND SIGNAL SYSTEM EVALUATION ROUTINE FOR DIAMOND INTERCHANGES-'PASSER III'

FREEWAY HEADER CARD FORM (One card per freeway)

NAME OF CITY	NAME OF FREEWAY	LAST	DATE	...	Lower Cycle Length (Sec)	Upper Cycle Length (Sec)	Cycle Length Increment (Sec)	Min. B' Direction Band Split	...	X-Scale	Y-Scale

INTERCHANGE HEADER CARD FORM (One card per interchange)

CROSS-STREET NAME	INTERCHANGE NUMBER	CYCLE LENGTH (SEC)	DELAY-OFFSET ANALYSIS ? CODE 1	CODE 2	CODE 3	CODE 4	CODE 1A	LEFT SIDE	RIGHT SIDE	INTERIOR TRAVEL TIME (SEC) LEFT TO RIGHT	RIGHT TO LEFT	INTERIOR QUEUE STORAGE (VEHICLES) RIGHT SIDE THROUGH	RIGHT SIDE LEFT TURN	LEFT SIDE THROUGH	LEFT SIDE LEFT TURN	PROGRESSION INFORMATION 'A' DIRECTION DISTANCE (FT.)	SPEED (MPH)	'B' DIRECTION DISTANCE (FT.)	SPEED (MPH)	QUEUE CLEARANCE (SEC) 'A' DIRECTION	'B' DIRECTION	PRIORITY PHASING ? (INTERNAL OFFSET, SEC) CODE 1	CODE 2	CODE 3	CODE 4	CODE 1A

Figure 7. PASSER III Input

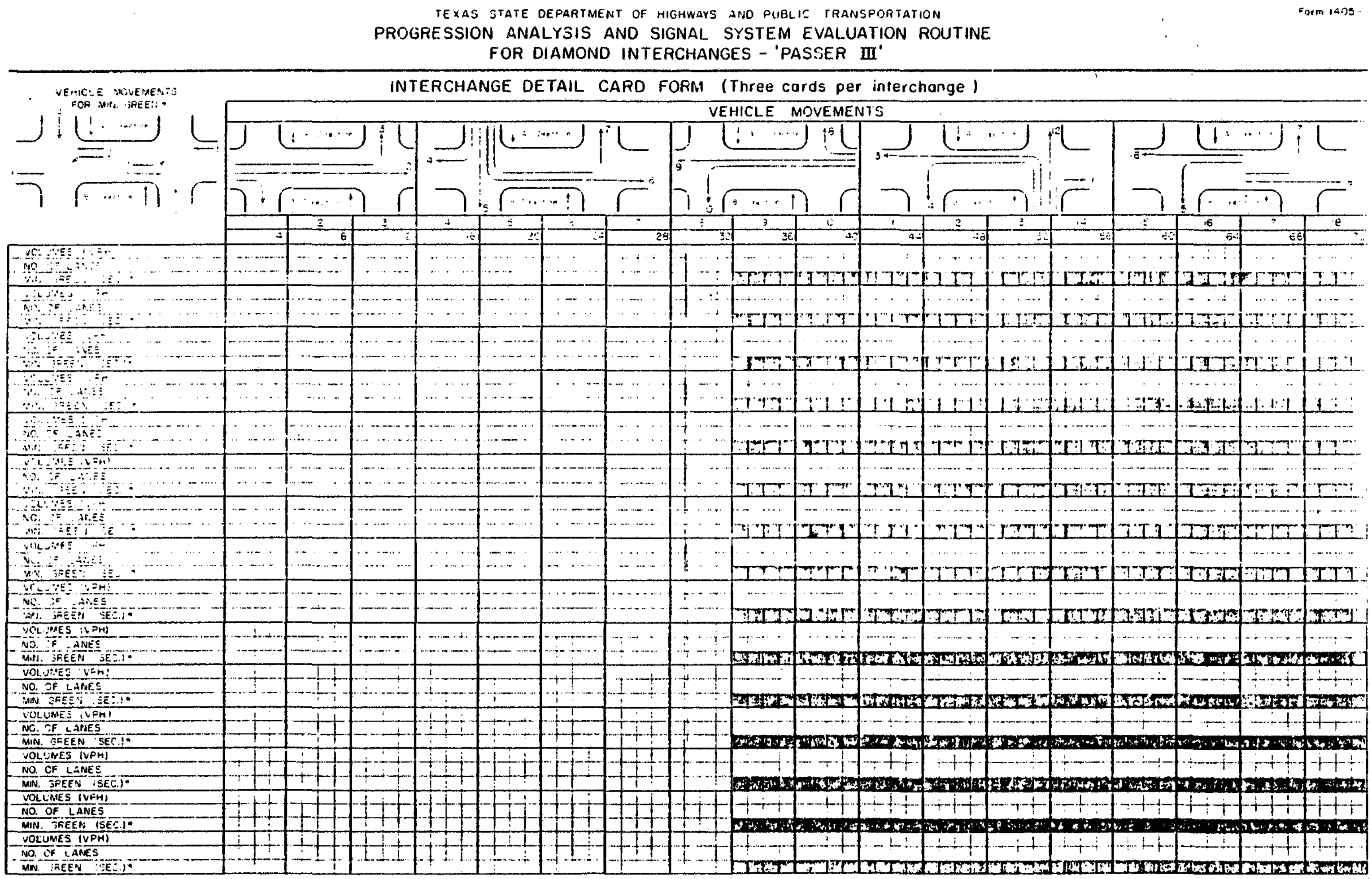


Figure 8. PASSER III Input

critical in analyzing interchange design modifications. When analyzing an existing interchange with fixed-time signal control, the actual cycle length should be used. An average of cycle lengths measured during the analysis period should be used to represent existing actuated signal operation. When analyzing interchange improvement alternatives, an appropriate cycle length must be calculated beforehand for use in PASSER III. As discussed in more detail later this is a critical step. For many of the physical improvement alternatives (e.g., frontage road widening, turnarounds), overall delay is reduced primarily because the cycle length can be shortened. The Webster method, given in Appendix A of the PASSER III user's manual, can be used to calculate cycle length.

If signal upgrading or retiming alternatives are being considered, then optional offsets should be examined by entering the appropriate codes in Columns 18 through 22. If delay-offset analyses are requested, then minus ones (-1's) should be entered in Columns 53 through 62, and the optimal offset found will be used. If a delay-offset analysis is not requested, then the desired internal offset should be supplied in the appropriate phasing code columns in Columns 53 through 62.

There are three Interchange Detail Cards. The first -- Line 1 -- contains traffic volumes. These are the detailed turning counts obtained in the "second level" of data collection discussed previously. This line of data would be adjusted to analyze potential improvements which would remove traffic from the interchange. Turnarounds, ramp reversals, and ramp additions can be considered in this manner.

Line 2 contains the effective number of lanes serving each movement volume. The program assumes a saturation flow rate of 1,800 vehicles per hour green per lane. This assumption can be adjusted by factoring the effective number of lanes as shown in the PASSER III user's manual. However, in the analysis of alternative improvements, the relative change in MOE's is the desired output. Because the types of improvements being analyzed generally would not alter saturation flow rates differentially, using the assumed 1800 vph rate simplifies input and produces reasonable answers.

The effective number of lanes is critical when analyzing certain types of interchange problems. Lengthy queues may be developing in certain lanes because of inadequate weaving distance between the exit ramp and the cross-street or because of inappropriate lane use designations. These conditions should be observed during the turning movement counts, and the representative effective number of lanes assigned to each movement. Line 2 is adjusted when analyzing frontage road widening and improvements, such as turnarounds, which affect the turning movement mix on an approach.

<u>Line 3</u> contains the minimum green times for each signal phase. The eight movement codes used on this line are shown in the upper left corner of the input form and are different from the 18 movement codes used on Lines 1 and 2. The minimum green times for movements 1, 2, 3, 4, 5, and 7 must provide adequate pedestrian crossing times. The sum of conflicting green times at each intersection must not be greater than the cycle length entered on the Interchange Header Card:

$$1 + 2 + 5 \leq \text{Cycle length}$$

and

$$3 + 4 + 7 \leq \text{Cycle length}$$

When the sum of the conflicting greens is less than the cycle length, the program uses the Webster method to calculate the splits as described in Appendix A of the PASSER III user's manual.

To replicate existing conditions the sum of the conflicting minimum greens must equal the cycle length:

1 + 2 + 5 = Cycle length
and
3 + 4 + 7 = Cycle length

If not, some optimization will occur. An additional condition must be met to replicate an existing four-phase, two-overlap signal operation:

1 + 2 + 3 + 4 = Cycle length + (2 x Overlap)

Generally, the interior through movements -- Movements 6 and 8 -- are satisfied by the sum of the exterior through movement and the interior left turn (i.e., 1 + 5 and 3 + 7). When this is true, the minimum green times for Movements 6 and 8 can be entered as nominal minimums, say 10 or 15 seconds, with no effect on the more critical green splits.

b. PASSER III Outputs -- The General Signalization Information table produced by PASSER III contains the most important information for evaluating minor interchange modifications. An example is shown as Figure 9. The green times and measures of effectiveness are shown for each of the eight signal phases. Again, the delay estimate is emphasized in these guidelines, although all of the evaluation criteria are useful. For both the left side and right side intersections, column A refers to the exterior cross-street approach, column B is the frontage road approach, column C is the interior left turn, and column D is the interior through movement. The total vehicle delay for each movement is obtained by multiplying the average delay per vehicle by the movement volume. These products can then be totaled to estimate total interchange delay.

c. PASSER III Applications -- This section discusses methods for using PASSER III to analyze minor interchange modifications. Because PASSER III was developed primarily to analyze signal timing, the delay estimates and other calculations are quite sensitive to signal cycle lengths and green splits. Therefore, it is important to consider the type of traffic signal and the

GENERAL SIGNALIZATION INFORMATION

I-35 AT EISENHAUER B RUN NO. 1 5/ 2/83

MEASURES OF EFFECTIVENESS	LEFT SIDE				RIGHT SIDE			
	A	B	C	D	A	B	C	D
GREEN TIME (SEC.)	50.9	20.0	39.1	90.0	23.9	21.7	64.4	88.3
VOLUME/CAPACITY RATIO, X	0.45	0.38	0.46	0.19	0.56	0.56	0.56	0.26
LEVEL OF SERVICE	A	A	A	A	A	A	A	A
DELAY (SEC./VEH.)	23.02	42.41	31.72	3.31	41.83	42.91	17.86	4.18
LEVEL OF SERVICE	B	C	C	A	C	C	B	A
PROBABILITY OF CLEARING QUEUE	1.00	1.00			0.98	0.97		
LEVEL OF SERVICE	A	A			A	A		
STORAGE RATIO			N.A.	N.A.			N.A.	N.A.

PHASE ORDER - ABC/ABC
INTERNAL OFFSET - 0 SECONDS

Figure 9. PASSER III Output Dashes General Signalization Information

signal operation strategy for the existing condition and for the improved interchange.

PASSER III uses the Webster equation to calculate delay on the exterior approaches. This equation is formulated to estimate vehicle delay on approaches to intersections with fixed time signal control. The validity and accuracy of using Webster's equation to estimate delay under traffic-actuated control varies. Actuated controller settings are often such that the maximum green times on each approach generally control the splits and cycle length during peak periods (i.e., the phases usually "max out"). In that case, the delay characteristics are similar to fixed-time control, and Webster's equation should produce accurate results.

Another actuated-control strategy uses relatively short vehicle intervals to extend the green time on an approach, with long maximum green times available. This strategy is intended to clear long queues consistently on each approach. With this strategy, signal phases often "gap out," even during peak traffic periods. Therefore, the effective splits and cycle lengths are much more variable. Under this condition, Webster's equation is less accurate in estimating vehicle delay. The case studies performed in this study were at locations with traffic-actuated signals using long maximum greens and short vehicle intervals. Although the absolute PASSER III delay estimates are not as accurate in this situation, the relative improvement due to interchange modifications can be approximated.

From PASSER III'S viewpoint, minor interchange modifications can be grouped into three basic categories:

- Improvements which change the <u>signal characteristics</u>,
- Improvements which change the <u>approach volumes</u>, and
- Improvements which change the <u>effective number of lanes</u> for each movement.

Each category will be discussed separately.

<u>Improvements Which Change Signal Characteristics</u> - This category includes:

- Signal retiming and
- Signal upgrading.

As discussed earlier retiming the existing traffic signal control equipment should be the first improvement alternative analyzed. PASSER III is perfectly suited to optimizing phasing patterns, internal offsets, and green splits. To quantify the operational improvement, PASSER III first should be run to replicate the existing operation. The actual signal timing, volume, and geometrics are input to PASSER III. In this mode, the model generates estimates of delay for comparison purposes but does not optimize any signal timing parameters.

The first step in generating the new signal timing is to calculate the optimal cycle length using Webster's method. Given this cycle length, PASSER III can then analyze alternative phasing patterns and offsets and optimize green splits. Using the delay calculated for the optimal solution, the potential operational improvement can be estimated as follows:

$$\frac{\text{Measured Delay}_{(\text{Before})}}{\text{PASSER III Delay}_{(\text{Before})}} \times \text{PASSER III Delay}_{(\text{After})} = \text{Modified Delay}_{(\text{After})}$$

$$\text{Delay Reduction} = \text{Measured Delay}_{(\text{Before})} - \text{Modified Delay}_{(\text{After})}$$

This method of factoring PASSER III delay estimates based on actual delay as measured in the field yields reasonable results within the accuracy required for cost-effectiveness analyses. A more rigorous approach to this modification involves adjusting the saturation flow per lane (i.e., adjusting the effective number of lanes per movement) until the PASSER III estimate for the before condition matches the measured delay. The ultimate success of this approach for traffic-actuated signals may depend on the actuated control strategy.

The effects of upgrading traffic signal equipment may be difficult or impossible to analyze using PASSER III. Upgrading from a fixed-time controller to traffic-actuated control can not be modelled directly by PASSER III. One approach to modelling such an improvement when traffic flow rates are highly variable is to assume that the actuated controller will result in cycle lengths which are near the optimal cycle lengths for short-period flow rates during the analysis period. For example, four separate optimal cycle lengths would be calculated using the 15-minute flow rates during a peak hour. Each cycle length would be input to PASSER III along with the corresponding 15-minute flow rates, converted to hourly volumes. The delay per vehicle would be multiplied by the number of vehicles during the 15-minute period, and the hourly delay calculated by adding the four periods delays. The comparative fixed-time delay would be estimated by making a run for each 15-minute flow rate but with the same "hourly optimal" cycle length during each period.

Another type of signal upgrading which cannot be directly modeled by PASSER III is the use of advanced detector placement and logic as in the case study projects at Eisenhauer/I-35 and Rittiman/I-35. Such improvements permit detector switching and phase skipping to improve signal efficiency. This clearly improved operations at the case study sites, however a true simulation model would be required to estimate these benefits beforehand.

<u>Improvements Which Change Approach Volumes</u> - This category of minor interchange modifications includes:

- Turnarounds,
- Ramp reversals, and
- Ramp additions.

These projects divert traffic from the signalized intersections. This diversion has two effects on interchange delay. First, the number of vehicles experiencing signal delay on the affected approach is reduced. Therefore, if the average delay per vehicle were to remain constant on that approach then the total vehicular delay would be decreased. Second, the total number of vehicles entering the interchange is reduced. Thus, the signal cycle length can be shortened, reducing signal delay throughout the interchange.

The volume changes caused by these improvements can be entered in PASSER III in a straightforward manner. The volumes entered on Line 1 of the Interchange Detail Cards are reduced for the appropriate movements. Line 2 -- the effective number of lanes for each movement -- will probably need to be revised as well to account for the different proportions of turning movements.

The other important aspect of PASSER III input for this type of improvement is signal timing. Again, optimal signal timing should be assumed to isolate the effects of the physical improvement. The existing approach volumes should be used in Webster's equation to calculate the optimal cycle length for the existing condition. This value is used in the Interchange Header Card. The analyst should then permit PASSER III to optimize the green splits by entering nominal minimum greens, say 10 or 15 seconds, for all movements on Line 3 of the Interchange Detail Cards. The average delay calculated for each approach is multiplied by the approach volume and then summed to determined the total interchange delay. The delay calculated in this run is the basis for comparing various improvement alternatives.

To model the proposed improvement, the same steps are followed. The analyst calculates a new, shorter cycle length using the reduced approach volumes. Again, nominal minimum greens should be entered on Line 3, and PASSER III will calculate the optimal splits for the new conditions. The delay calculated in this run is totaled for the interchange and subtracted from the base case delay to determine the estimated operational improvement.

As discussed earlier, the accuracy of PASSER III in estimating the delay on each approach may vary depending on actuated-controller settings and variations in traffic flow rates throughout the peak hour. However, the percentage change in total interchange delay should be a reasonable estimate of the expected operational improvement.

Improvements Which Change the Effective Number of Lanes - This category includes:

- Frontage road widening,
- Ramp relocations, and
- Lane use reassignments.

Frontage road widening increases the total number of approach lanes and, therefore, increases the effective number of lanes carrying one or more of the movements on that approach. Relocating a ramp farther back from the cross-street can change the proportions of turning movements in each lane by providing more weaving distance and thus more balanced lane distribution. This change in turning movement proportions, as in reassigning lane use, changes the effective number of lanes serving each movement.

Using PASSER III to model this type of project is nearly identical to the application discussed in the previous section. The only difference is that the volumes entered on Line 1 of the Interchange Detail Card remain the same before and after the improvement.

The data which are changed to represent the before and after conditions include:

- Signal cycle length (calculated using Webster's equation) -- Interchange Header Card
- Effective number of lanes -- Line 2 of the Interchange Detail Card

The green splits will also change but this is performed by PASSER III, with nominal minimum greens entered for both the before and after cases.

Cost-Effectiveness Analysis

The detailed analysis of improvement alternatives will provide an indication of whether an alternative will provide an operational improvement. However, these analyses will not indicate whether an improvement is economically attractive or how efficiently public funds will be expended. The following paragraphs discuss the computation of various road user costs and illustrate how they may be used to assess attractiveness and to prioritize projects.

a. Safety vs. Operations -- One of the first considerations in a cost-effectiveness analysis is what type(s) of benefits may be expected. As mentioned previously, road user benefits may take the form of reduced accident costs and/or reduced user time and vehicle costs. An ideal evaluation scheme would permit the use of a common base (such as dollars) to evaluate and prioritize all projects. However, simplifying safety benefits to dollar savings is a very presumptive process that includes various assumptions regarding predicted effectiveness and dollar values for forestalled property damage, injury and fatal accidents. Many projects would be highly cost-effective and would receive high priorities if they were assumed to forestall one fatal accident per year.

There are numerous methods for rating safety improvements. Since most of the projects examined in this research were operations-oriented, no attempt is made at integrating the economic aspects of safety. However, the potential safety implications should be recognized in each analysis, and at least subjectively considered along with the economic analyses of operations. Generally, if one accepts the tenet that "efficiency breeds safety," then improvement that is operationally efficient will likely produce neutral or positive safety benefits.

b. Types of Road-User Costs -- Excluding safety benefits, there are four basic categories of road-user costs:

Running costs:	vehicle operating costs including costs of constant speed operation, as well as stopping and accelerating from a stop;
Travel time costs:	value of the occupants' time while moving, including driver and passengers. Typical current values are \$10.20/hour/occupant, and 1.3 occupants per vehicle;
Delay costs:	value of occupants' time while stopped, typically at an intersection;
Idling costs:	vehicle operating costs while stopped. Typically these costs are negligible except for long-duration level-of-service "F" conditions.

It should be recognized the road user costs in urban areas are likely to be extremely high, even under the most efficient alternatives. Computation of "before" and "after" costs, as well as the net savings is frequently useful in gaining a perspective of the relative magnitude of improvements. All of the following tables and graphs are based on composite vehicle costs, representing 95% passenger cars, 3% SU trucks and buses, and 2% semi tractor-trailers.

c. Computation of Road-User Costs --

- Running Costs

Running costs require the input of speed, distance and number of stops. Speed and distance are used to estimate costs of constant speed operation. Speed and number of stops are combined to estimate "speed change cycle" or "stopping" costs. Table 4 shows running cost by speed and roadway type, adapted from Reference (10). Friction from vehicle-vehicle interactions and roadside distractions accounts for the difference in costs.

Stopping costs apply to intersections and therefore to city streets (including frontage roads). Although more detailed estimates of speed change cycle costs can be developed, the assumption that all vehicles stop and the use of the values in Table 5 should suffice for most analyses.

- -

Example

An example employing all of these techniques is a ramp addition project, as shown in Figure 10.

"Before" Running Costs =

100 vehicles X 0.75 miles X $.1254/mile

+ 100 vehicles X 1 intersection X .0761 $/stop

= $9.405 + $7.61 = $17.02

"After" Running Costs =

100 vehicles X 0.75 miles X $ .1166/mile

+ 100 vehicles X 0 intersections

= $8.75

Therefore the next benefit, considering only running costs, is $17.02 - $8.75 = $8.27 per day.

- -

TABLE 4. RUNNING COSTS ON FREEWAYS AND CITY STREETS BY UNIFORM SPEED

			Freeways			
Speed (mph)	30	35	40	45	50	55
Cost ($/mile)	.1201	.1054	.1073	.1110	.1130	.1166
			City Streets			
Speed (mph)	30	35	40	45	50	
Cost ($/mile)	.1214	.1215	.1228	.1254	.1294	

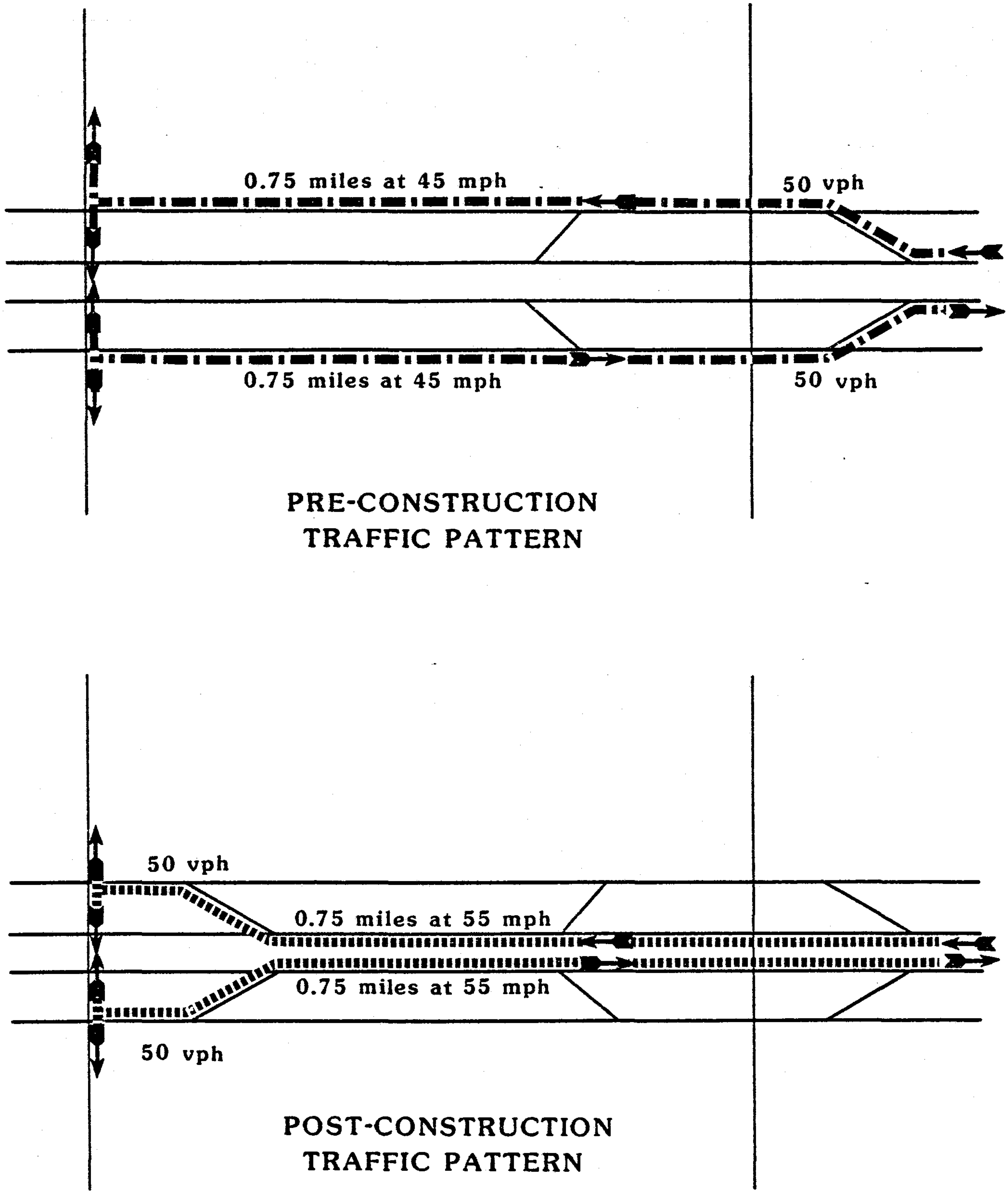


Figure 10. Operational Change Due to Ramp Addition

TABLE 5. EXCESS COSTS OF SPEED CHANGE CYCLES[1]

Initial Speed (mph)	30	35	40	45	50
Cost ($/veh)	.0352	.0462	.0611	.0761	.0966

[1]Assumes all vehicles stop.

- Travel Time Costs

The value of the road user's time can make up a significant portion of the overall user cost. Travel time will be important when there is a change in operating speed. The actual dollar value ascribed to this element will be dependent upon the accepted value of time and the assumed number of occupants. Recent research has indicated that the current value of road user time is $10.20 per hour. Average occupancy of 1.3 persons per vehicle is a typical rate for urban areas. The cost of travel time is given by the equation:

$$\text{Travel Time Costs} = \frac{\text{Distance}}{5280} \cdot \frac{1}{\text{Speed}} \cdot \frac{\text{Time}}{\text{Cost}} \cdot \frac{\text{Number of}}{\text{Occupants}}$$

Where: Distance = feet

Speed = miles per hour

Unit Time Cost = user defined
(current research indicates $10.20/hour)

Number of Occupants = user defined
(default = 1.3 per vehicle)

Table 6 presents unit travel time factors for an assumed time value of $1.00/hour and a 1.3 occupancy rate. The user may multiply an acceptable time value by the appropriate factor and distance to obtain total travel time cost.

TABLE 6. UNIT TRAVEL TIME FACTORS BY OPERATING SPEED

Operating Speed (mph)	30	35	40	45	50	55
Travel Time Factor $\frac{\text{occupant-hours}}{\text{mile}}$	.043	.037	.033	.029	.026	.024

Note: To convert to travel time cost, multiply by distance in miles and time value.

- -

Example

The previous ramp addition can be used as an example of travel time cost analysis.

Before Travel Time Cost = 100 vehicles X .75 miles X .029 X $10.20

= $22.19

After Travel Time Cost = 100 vehicles X .75 miles X .024 X $10.20

= $18.36

Therefore, the net travel time benefit of the new route is

$22.19 - $18.36 = $3.83 per day.

o <u>Delay Costs</u>

Estimating the expected savings in delay is typically more difficult than estimating running costs or travel time costs. The most conclusive and predictable types of delay reduction are those circumstances where a group of vehicles is actually removed from the intersection. The savings that will accrue to this group will be a function of aggregate time saved.

$$\text{Delay Cost} = \text{Number of Vehicles} \times \frac{\text{Delay (sec/veh)}}{3600} \times \text{Time Cost} \times \text{Occupancy}$$

- -

- -

Example

As mentioned previously, the 100 vehicles using the new ramp no longer are delayed at the upstream intersection. The delay savings are computed as follows:

$$\text{Before Delay Cost} = 100 \text{ vehicles} \times \frac{30 \text{ sec}}{3600} \times 10.20 \times 1.3$$
$$= \$11.05 \text{ per day}$$

$$\text{After Delay Cost} = 100 \text{ vehicles} \times \frac{0 \text{ sec}}{3600} \times 10.20 \times 1.3$$
$$= 0$$

Therefore, the daily savings were at least $11.05

- -

It is generally recognized that the reduction in intersection volume will reduce the overall delay to all other vehicles in the intersection. Benefits of this type have been referred to as "indirect" benefits since they accrued to a group that was not rerouted or otherwise directly affected. In the actual ramp addition projected evaluated in Research Report 210-11, the vehicles that continued using the upstream intersection experienced a reduction in average delay from 29 seconds to 21 seconds.

Delay is highly sensitive to signal timing, geometry, lane use and other factors. Because of this sensitivity, estimation of indirect benefits from expected delay reductions should be undertaken very carefully.

- Idling Costs

Idling costs are generally negligible compared to other costs. The amount of idling time is equivalent the vehicle-seconds of delay. A composite estimate of fuel consumed at idle is 0.379 gallons per vehicle-hour. Since this rate would produce user costs of less than 10 percent of delay costs, it is recommended that very little attention be devoted to this aspect.

d. Road-User Benefits from Interchange Improvement Projects -- Not all of the above types of user costs are affected by the various minor interchange improvements. Table 7 shows the categories of user costs to be considered for each type of improvement. Idling costs are not shown because they are generally negligible compared to other costs. If they are included in an economic analysis, they should be considered whenever delay savings are calculated.

TABLE 7. TYPES OF USER COST SAVINGS BY IMPROVEMENT

Improvement	Running Costs	Travel Time Costs	Delay Costs
Improve Signal			X
Widen Frontage Road			X
Add Turn Lanes			X
Reassign Lane Use			X
Add Turnaround			X
Improve Existing Turnaround			X
Relocate Ramp	X	X	X
Reverse Ramp(s)	X	X	X
Add Downstream Ramp	X	X	X
Add Ramp Capacity	X	X	
Improve Ramp Geometry	X	X	
Add Auxiliary Lane	X	X	

e. Benefit/Cost Analysis -- Using the preceding methods along with the results of the operational analysis yields an estimate of road-user cost savings expected for the project. If the user benefits were calculated based on peak-hour savings, then the total daily benefits should be estimated. The analyst should consider carefully the user benefits assigned to off-peak periods. Some types of improvements (e.g., turnarounds) may produce significant delay reduction during both peak and off-peak periods. Other improvements (e.g., ramp relocation) may have little or no impact on delay or running costs under low volume conditions.

A conservative approach was taken in the case studies. It was assumed that essentially all of the daily user benefits were received during the 12 hours with the highest traffic volumes. Further, it was assumed that the benefits are proportional to the total traffic volume entering the interchange. For example, if the combined morning and evening peak hour volumes represent 30 percent of the 12-highest-hour volume, then it was assumed that the combined peak user savings are roughly 30 percent of the 12-highest-hour user savings. Annual benefits are estimated based on the 250 working days per year.

Once annual user benefits are calculated, they can be used with the estimated construction cost to determine a benefit/cost ratio for the improvement being evaluated. The annual user benefit is multiplied by the uniform series present worth factor for the assumed functional life and discount rate. The product is the present worth of the user benefits accruing over the life of the improvement. Dividing this value by the estimated construction cost yields a benefit/cost (B/C) ratio for the proposed interchange improvement.

The B/C ratio is useful in determining which improvements are the most cost-effective for a given interchange. In addition, it can be used to prioritize improvement among a group of interchanges. The overall accuracy of the calculations used in estimating the B/C ratio should always be considered. If competing projects have B/C ratios within, say, 20 percent of each other, then the cost-effectiveness of the projects is similar enough that other considerations should determine which project is built.

SUMMARY

Overall Effectiveness of Minor Interchange Improvements

The case studies conducted as part of this study along with previous evaluations in the Study 210 series establish that relatively minor improvements to urban diamond interchanges are a viable class of projects. Well-selected packages of improvements at appropriate locations can reduce interchange delay by as much as 50 to 60 percent, with conservatively-calculated benefit/cost ratios in the range of 5:1 to 10:1.

Generally, retiming traffic signals or upgrading signal equipment is by far the most cost-effective interchange improvement. The signal upgrading portions of the case study projects had benefit/cost ratios on the order of 40:1 to 50:1. Signal projects should be considered before any other physical improvement. PASSER III can be used to develop optimal timing patterns for existing equipment, and advanced detector placement and logic will improve operations further.

The conservatively-calculated benefit/cost ratios for the turnarounds built in the case study projects were lower than for the other portions of the improvements -- 2.6:1 and 0.2:1. This indicates that the site-specific potential benefits of retrofit turnarounds should be very carefully analyzed. The turnarounds do provide flexibility and capacity to accommodate changing traffic patterns. They also provide off-peak as well as peak period delay savings. Of course, the case studies represent retrofit turnarounds. On new interchanges or major reconstructions, the additional cost of building a turnaround is much lower, and the benefit/cost ratios would be higher.

Data Collection

New land developments in urban freeway corridors result in rapidly-changing traffic patterns at interchanges. Therefore, the periodic collection of volume and delay data is essential to monitor interchange performance and improvement priorities. The point sample method of measuring intersection delay provides valuable data on interchange performance at a reasonable level of effort.

Analysis of Improvement Alternatives

The Recommended Guidelines contained herein for identifying and selecting interchange improvements are not cookbook procedures. The number of variables and site-specific conditions that affect interchange performance preclude such an approach. Rather, they are intended to provide general guidance in systematically identifying and selecting improvements. It may not be necessary to apply the procedures at the same level of detail in all situations. However, three critical elements should be included.

1. Collecting current volume and delay data,
2. Identifying specific operational deficiencies, and
3. Systematically matching candidate improvements to the identified problems.

PASSER III is a valuable tool for detailed analysis of improvement alternatives. It is perfectly-suited to optimizing signal timing and is relatively easy to use for that purpose. It cannot analyze directly signal upgrading from fixed-time to actuated control or advanced detection schemes.

Another application of PASSER III is the operational evaluation of physical improvement projects. An accurate representation of signal operations associated with these improvements must be entered to obtain meaningful results. If actuated-control strategies are used which cannot be closely approximated by a fixed-time representation, then the PASSER III estimates of delay may vary in accuracy from approach to approach. However, comparisons with case study data show that PASSER III can provide a good estimate of the impact on overall interchange delay resulting from improvement projects.

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APPENDIX A

Case Study Number 1

I-35 at Eisenhauer Road
San Antonio, Texas

Improvements:

- Frontage road widening
- Traffic signal upgrading
- Frontage road turnaround

Conditions Before Construction

Eisenhauer Road intersects I-35 in northeast San Antonio. The interchange geometry and traffic conditions prior to the improvements are summarized in Figure A-1. The volume and delay data were collected on December 15 and 16, 1981. Statistics shown for the total interchange are the totals for the exterior approaches. The interchange was a typical urban diamond interchange with one-way frontage roads. A traffic-actuated signal controller with four-phase, two-overlap phasing was used.

Based on the average stopped delay per vehicle for the entire interchange, traffic operations were approximately level-of-service D during the morning peak hour and level-of-service E during the evening peak hour. Total traffic volumes on the exterior approaches to the west intersection (eastbound Eisenhauer and southbound frontage road) were less than east side volumes during both peak periods. However, the average delay per vehicle was higher and a higher percentage of vehicles was stopped on the west side. This suggested that improvements to the west intersection would be the most effective.

The potential volumes for frontage road turnarounds were fairly light. Approximately 40 vehicles made the northbound to southbound movement during both the morning and evening peak hours. The southbound to northbound u-turn was 35 vehicles per hour during the morning and 60 vehicles per hour during the evening peak.

Improvements

The improvement project at I-35 and Eisenhauer consisted of three elements.

1. The southbound frontage road was widened to three lanes from the southbound exit ramp to the intersection. The new lane is for left-turning vehicles only, with the former inside lane now used for either left turns or through movements.

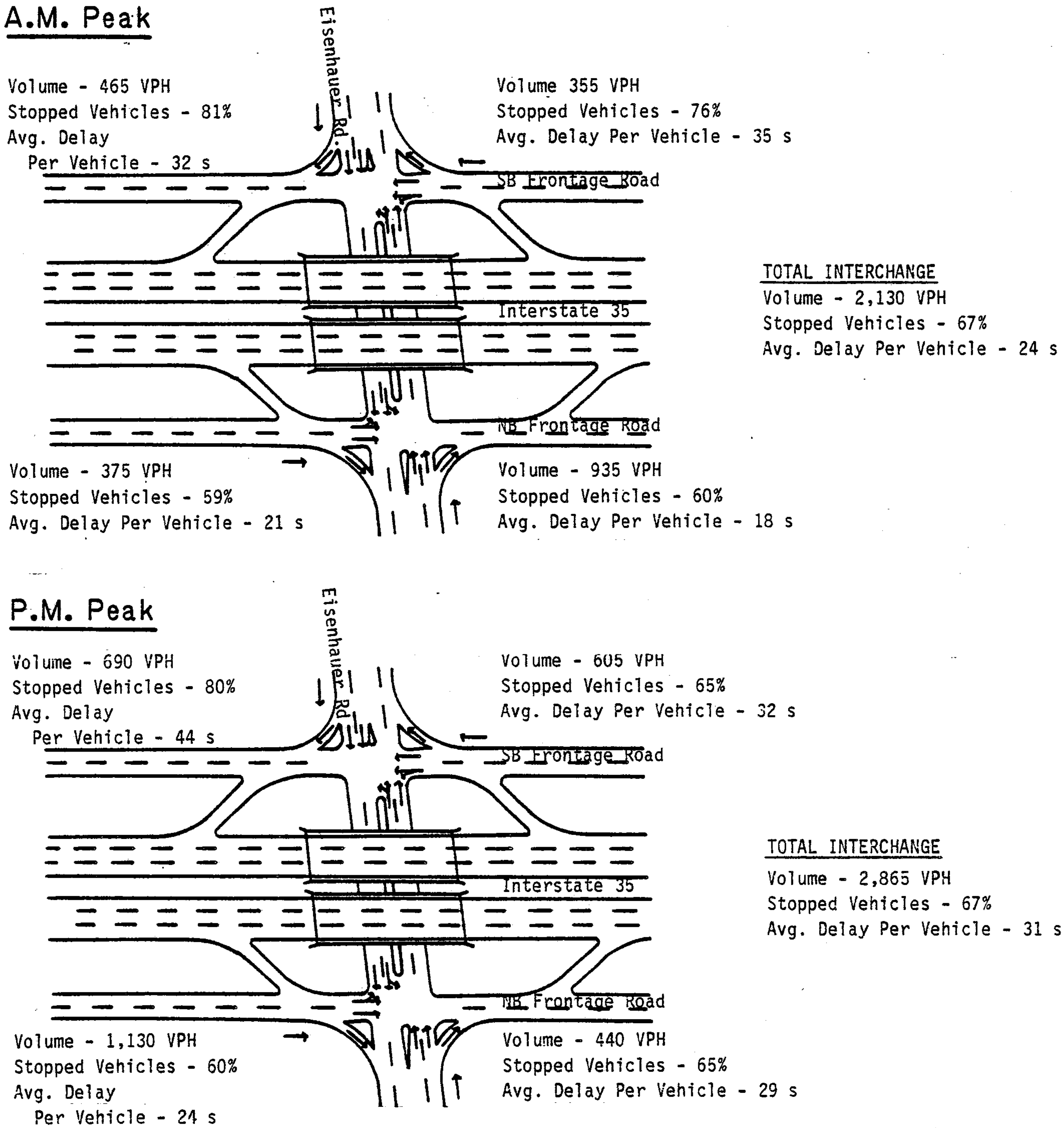


Figure A-1. Interchange Geometry and Operations Eisenhauer Road at I-35
"Before" Condition

2. The traffic signal was upgraded by the installation of a new controller and new vehicle detectors. This permits a more sophisticated variation of the four-phase, two-overlap phasing plan.

3. A frontage road turnaround was constructed on the north side of Eisenhauer, accommodating southbound to northbound u-turns.

The contract cost for the construction project is summarized below:

Frontage road widening	$133,000
Signal upgrading	29,000
Turnaround	265,000
TOTAL	$427,000

Conditions After Construction

The three elements of the project were put into service one at a time to permit incremental evaluations of the operations improvements attributable to each element. The added frontage road lane was opened first. Volume and delay data for this condition (After 1) were collected between November 17 and December 6, 1983. Next, the new traffic signal was put into operation (After 2), and data were collected between February 23 and February 28, 1984. Finally, the turnaround was opened (After 3), and data were collected from April 3 to April 18, 1984.

Each phase of the improvement had been in service approximately one week before data were collected. Although traffic operations appeared normal in each case, drivers may not have fully adjusted to the new geometrics or signal control. Another factor which may have influenced driver behavior was the use of temporary traffic control devices to open the improvements incrementally. For example, traffic cones were used to close the entrance to the turnaround until it was opened. Overall, however, the data obtained after construction should be fairly representative of the basic geometric conditions.

The interchange geometry and traffic conditions for the three cases after construction are shown in Figures A-2 to A-4.

Discussion of Findings

Traffic volumes and delay data are summarized for the Before condition and the three After cases in Tables A-1 and A-2. The summary tables show substantial reductions in vehicle delay due to the improvement projects. Between the Before and After 1 periods, morning peak hour traffic increased 14 percent. However, the average delay per vehicle dropped by 21 percent, representing a shift from level-of-service D to level-of-service C. Morning traffic volumes remained fairly constant through the After 2 and After 3 cases. With constant volumes, the signal upgrading and turnaround each reduced average delay. Taking all the improvements together morning peak hour average delay was reduced by 46 percent, and operations moved from level-of-service D to level-of-service B.

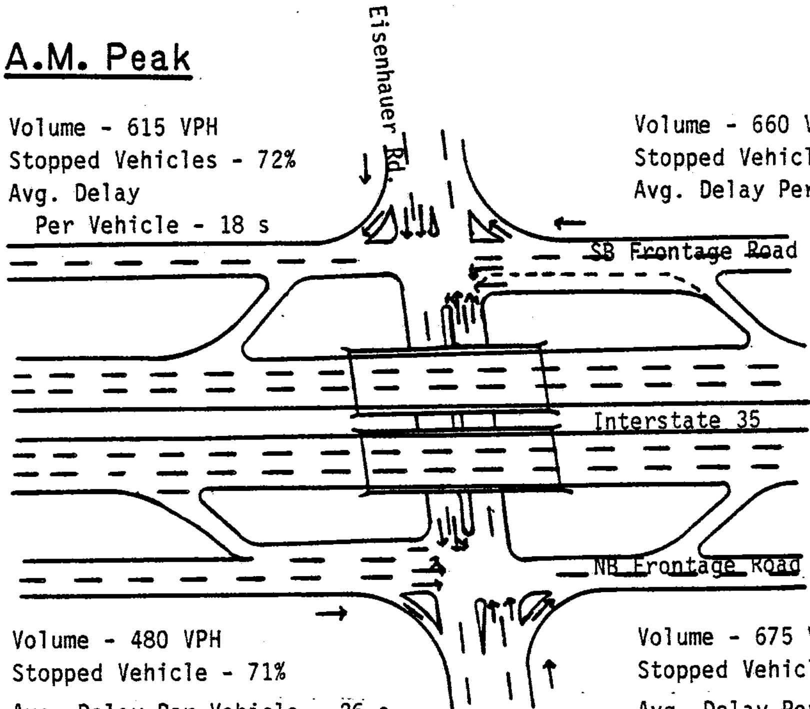
Evening peak hour traffic grew by about nine percent during the entire study period but average delay was cut by 55 percent. Interchange level-of-service improved from E to B. The signal upgrading produced the largest increment of improvement by reducing delay by more than half. However, the addition of the turnaround had negligible impacts on evening peak hour operations.

User costs based on delay and idling are summarized in Table A-3. The vehicle delay for conditions before each increment of improvement were adjusted to account for changes in traffic volumes between the before and after data collection. Thus, the change in delay represents change due to the improvement project only. The actual percentage of vehicles stopped and the average delay per stopped vehicle on each approach for the before condition were applied to the approach volumes for the after condition. Because the total

A.M. Peak

Volume - 615 VPH
Stopped Vehicles - 72%
Avg. Delay
Per Vehicle - 18 s

Volume - 660 VPH
Stopped Vehicles - 58%
Avg. Delay Per Vehicle - 18 s



Volume - 480 VPH
Stopped Vehicle - 71%
Avg. Delay Per Vehicle - 26 s

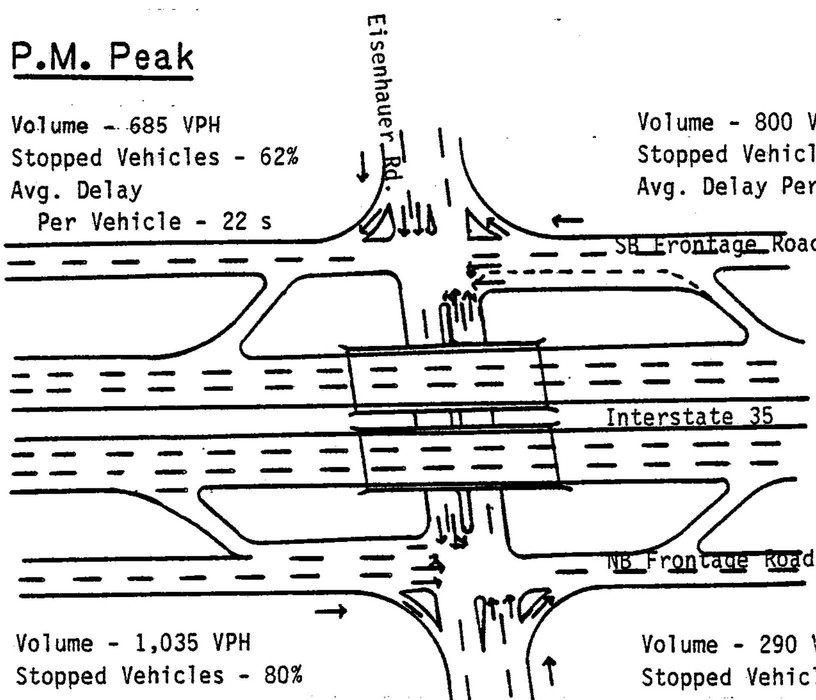
Volume - 675 VPH
Stopped Vehicles -55%
Avg. Delay Per Vehicle - 15 s

TOTAL INTERCHANGE
Volume - 2,430 VPH
Stopped Vehicles - 63%
Avg. Delay Per Vehicle - 19 s

P.M. Peak

Volume - 685 VPH
Stopped Vehicles - 62%
Avg. Delay
Per Vehicle - 22 s

Volume - 800 VPH
Stopped Vehicles - 57%
Avg. Delay Per Vehicle 18 s



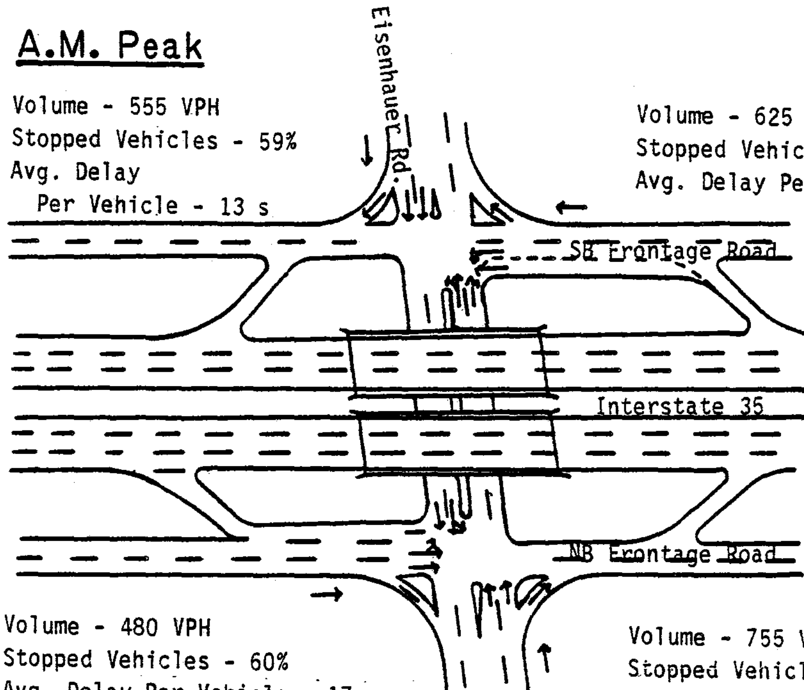
Volume - 1,035 VPH
Stopped Vehicles - 80%
Avg. Delay Per Vehicle - 41 s

Volume - 290 VPH
Stopped Vehicles - 81%
Avg. Delay Per Vehicle - 28 s

TOTAL INTERCHANGE
Volume - 2,810 VPH
Stopped Vehicles - 69%
Avg. Delay Per Vehicle - 28 s

Figure A-2. Interchange Geometry and Operations Eisenhauer Road at I-35
"After 1" Condition
• Frontage Road Widening

A.M. Peak



Volume - 555 VPH
Stopped Vehicles - 59%
Avg. Delay
Per Vehicle - 13 s

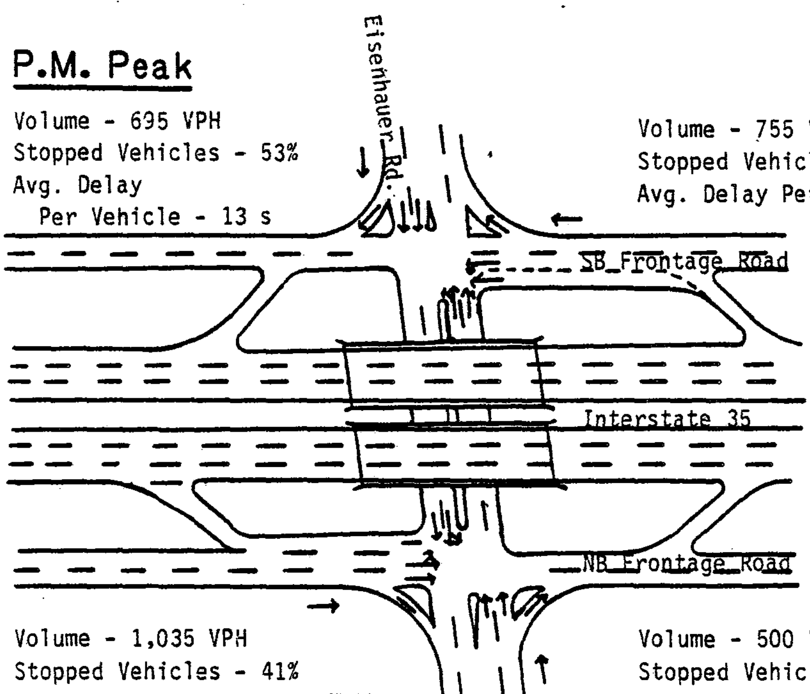
Volume - 625 VPH
Stopped Vehicles - 65%
Avg. Delay Per Vehicle - 14 s

TOTAL INTERCHANGE
Volume - 2,415 VPH
Stopped Vehicles - 60%
Avg. Delay Per Vehicle - 16 s

Volume - 480 VPH
Stopped Vehicles - 60%
Avg. Delay Per Vehicle - 17 s

Volume - 755 VPH
Stopped Vehicles - 58%
Avg. Delay per Vehicle - 18 s

P.M. Peak



Volume - 695 VPH
Stopped Vehicles - 53%
Avg. Delay
Per Vehicle - 13 s

Volume - 755 VPH
Stopped Vehicles - 58%
Avg. Delay Per Vehicle - 15 s

TOTAL INTERCHANGE
Volume - 2,985 VPH
Stopped Vehicles - 53%
Avg. Delay Per Vehicle - 13 s

Volume - 1,035 VPH
Stopped Vehicles - 41%
Avg. Delay Per Vehicle - 10 s

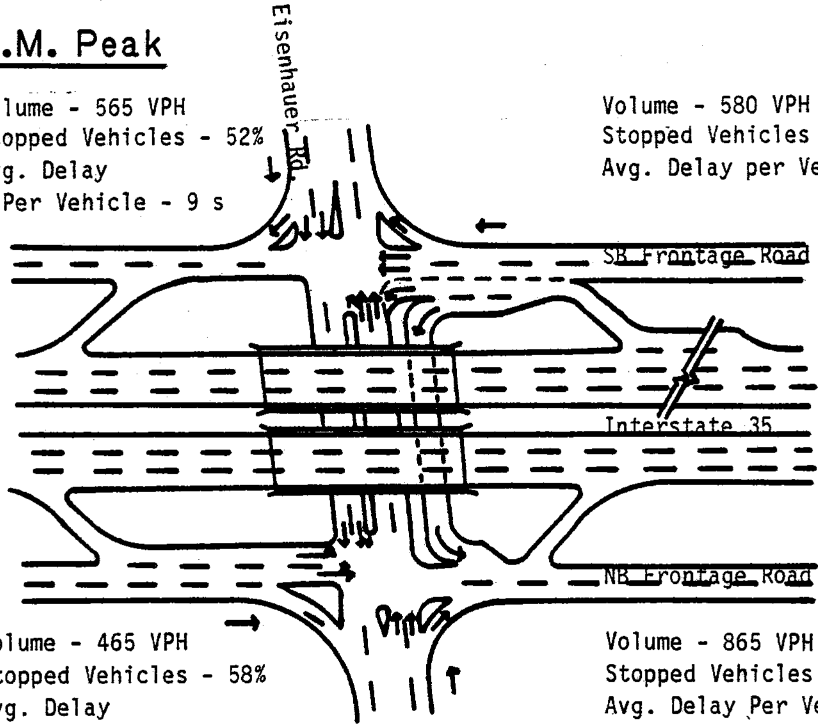
Volume - 500 VPH
Stopped Vehicles - 73%
Avg. Delay Per Vehicle - 17 s

Figure A-3. Interchange Geometry and Operations Eisenhauer Road at I-35
"After 2" Condition
- Frontage Road Widening
- Signal Upgrading

A.M. Peak

Volume - 565 VPH
Stopped Vehicles - 52%
Avg. Delay
Per Vehicle - 9 s

Volume - 580 VPH
Stopped Vehicles - 45%
Avg. Delay per Vehicle - 11 s



TOTAL INTERCHANGE
Volume - 2,475 VPH
Stopped Vehicles - 54%
Avg. Delay Per Vehicle - 13 s

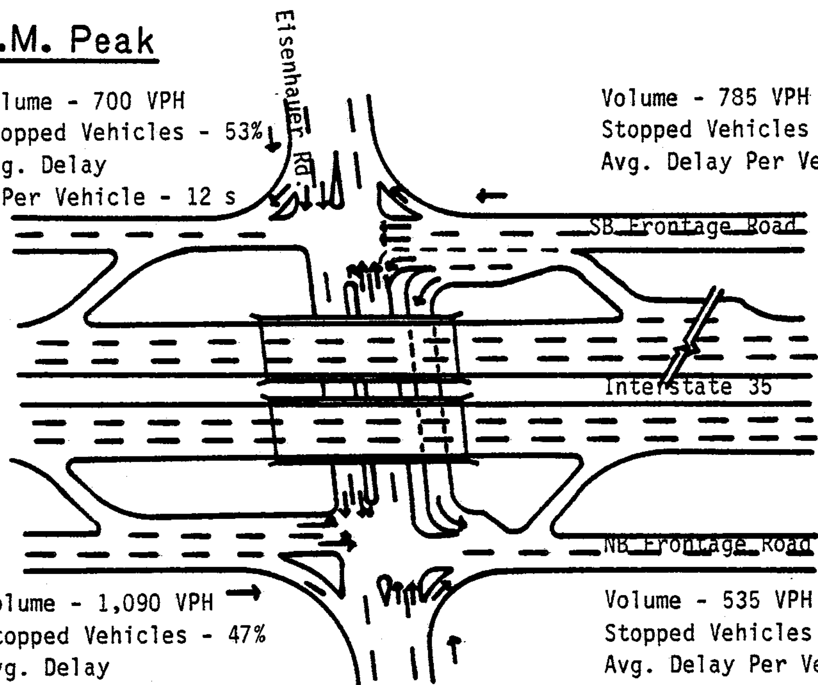
Volume - 465 VPH
Stopped Vehicles - 58%
Avg. Delay
Per Vehicle - 14 s

Volume - 865 VPH
Stopped Vehicles - 58%
Avg. Delay Per Vehicle - 17 s

P.M. Peak

Volume - 700 VPH
Stopped Vehicles - 53%
Avg. Delay
Per Vehicle - 12 s

Volume - 785 VPH
Stopped Vehicles - 44%
Avg. Delay Per Vehicle - 13 s



TOTAL INTERCHANGE
Volume - 3,110 VPH
Stopped Vehicles - 51%
Avg. Delay Per Vehicle - 14 s

Volume - 1,090 VPH
Stopped Vehicles - 47%
Avg. Delay
Per Vehicle - 13 s

Volume - 535 VPH
Stopped Vehicles - 71%
Avg. Delay Per Vehicle - 21 s

Figure A-4. Interchange Geometry and Operations Eisenhauer Road at I-35
"After 3" Condition

- Frontage Road Widening
- Signal Upgrading
- Turnaround

TABLE A-1. INTERSECTION DELAY SUMMARY

Location: Eisenhauer Road at I-35
Time: A.M. Peak Hour

	Before	Widen Frontage Road (After 1)	+ Signal Up-grade (After 2)	+ Turnaround (After 3)
Approach: NB Frontage Road				
Total Volume	375	480	480	465
% Delayed Vehicles	59%	71%	60%	58%
Vehicle-Seconds of Delay	8,020	12,360	8,160	6,525
Average Delay per Stopped Vehicle	36	36	28	24
Average Delay per Vehicle	21	26	17	14
Approach: SB Frontage Road				
Total Volume	355	660	625	580
% Delayed Vehicles	76%	58%	65%	45%
Vehicle-Seconds of Delay	12,365	11,655	8,535	6,195
Average Delay per Stopped Vehicle	46	30	21	18
Average Delay per Vehicle	35	18	14	11
Approach: EB Eisenhauer				
Total Volume	465	615	555	565
% Delayed Vehicles	81%	72%	59%	52%
Vehicle-Seconds of delay	14,855	11,235	7,035	5,280
Average Delay per Stopped Vehicle	39	25	21	18
Average Delay per Vehicle	32	18	13	9
Approach: WB Eisenhauer				
Total Volume	935	675	755	865
% Delayed Vehicles	60%	55%	58%	58%
Vehicle-Seconds of Delay	16,760	10,200	13,920	14,655
Average Delay per Stopped Vehicle	30	27	32	29
Average Delay per Vehicle	18	15	18	17
Total Interchange				
Total Volume	2,130	2,430	2,415	2,475
% Delayed Vehicles	67%	63%	60%	54%
Vehicle-Seconds of Delay	52,000	45,450	37,650	32,655
Average Delay per Stopped Vehicle	36	30	26	24
Average Delay per Vehicle	24	19	16	13

TABLE A-2. INTERSECTION DELAY SUMMARY

Location: Eisenhauer Road at I-35
Time: P.M. Peak Hour

	Before	Widen Frontage Road (After 1)	+ Signal Upgrade (After 2)	+ Turnaround (After 3)
Approach: NB Frontage Road				
Total Volume	1,130	1,035	1,035	1,090
% Delayed Vehicles	60%	80%	41%	47%
Vehicle-Seconds of Delay	27,120	42,780	10,455	14,355
Average Delay per Stopped Vehicle	40	51	25	28
Average Delay per Vehicle	24	41	10	13
Approach: SB Frontage Road				
Total Volume	605	800	755	785
% Delayed Vehicles	65%	57%	58%	44%
Vehicle-Seconds of Delay	19,650	14,355	11,490	10,560
Average Delay per Stopped Vehicle	50	31	26	31
Average Delay per Vehicle	32	18	15	13
Approach: EB Eisenhauer				
Total Volume	690	685	695	700
% Delayed Vehicles	80%	62%	53%	53%
Vehicle-Seconds of Delay	30,360	14,805	8,940	8,760
Average Delay per Stopped Vehicle	55	35	24	24
Average Delay per Vehicle	44	22	13	12
Approach: WB Eisenhauer				
Total Volume	440	290	500	535
% Delayed Vehicles	65%	81%	73%	71%
Vehicle-Seconds of Delay	12,870	8,145	8,745	11,265
Average Delay per Stopped Vehicle	45	35	24	30
Average Delay per Vehicle	29	28	17	21
Total Intersection				
Total Volume	2,865	2,810	2,985	3,110
% Delayed Vehicles	67%	69%	53%	51%
Vehicle-Seconds of Delay	90,000	80,085	39,630	44,940
Average Delay per Stopped Vehicle	47	41	25	28
Average Delay per Vehicle	31	28	13	14

TABLE A-3. ECONOMIC ANALYSIS - EISENHAUER ROAD AT I-35

	Delay and Idling Costs[1]				User Benefits			
	Before[2]		After					
	Daily	Annual	Daily	Annual	Annual	Present[3] Value	Construction Cost	B:C Ratio
Frontage Road Widening	$2,029	$507,200	$1,735	$433,700	$73,500	$625,700	$133,000	4.7:1
Signal Upgrading	1,795	448,700	1,074	268,600	180,100	1,533,300	29,000	52.9:1
Turnaround	1,100	274,900	1,074	268,600	6,300	53,600	265,000	0.2:1
Eisenhauer Total					$259,900	$2,212,600	$427,000	5.2:1

[1]Based on estimated delay during 12 highest-volume hours per day, 250 working days per year.
[2]Delay for before cases adjusted to represent traffic volumes after the improvement.
[3]20-year functional life, 10% discount rate.

interchange volumes generally increased between evaluation periods, the adjusted before delay is a conservative estimate of the actual delay which would have resulted had the after volumes been present with the before geometrics.

The analysis shown represents delay savings for the 12 highest hourly volumes of the day, on weekdays only, and therefore understates total user benefits. It is assumed that the annual reduction in delay will remain constant over the entire functional life of the improvement. In fact, volumes will probably grow, and additional user benefits will accrue. Therefore, the estimated user benefits are considered to be conservative.

Implications for Other Sites

The signal upgrading at Eisenhauer Road and I-35 produced the largest single increment of improvement. Since signal upgrading is also usually less costly than any significant roadway improvement, signal retiming or upgrading should always be considered and evaluated as the first step in diamond interchange modifications. Signal improvements generally reduce delay on all approaches to the interchange. Occasionally, delay will be increased on an approach on which the percentage of vehicles stopped previously was well below the interchange average.

Construction of the turnaround produced only marginal immediate improvements at this location. This was to be expected given the relatively light southbound-to-northbound volumes. A recent count found only about 15 vehicles using the turnaround during the morning peak hour (fewer than expected) and about 70 vehicles using the turnaround during the evening peak (slightly more than expected). The opposing northbound-to-southbound movement has increased considerably since the Before counts were taken with 115 vehicles and 65 vehicles during the morning and evening peak hours, respectively. This shift in volumes demonstrates that the u-turn movements at diamond interchanges are variable, depending more on land uses in the immediate vicinity than on overall traffic growth. Adding turnarounds improves the flexibility of an interchange to meet changing access patterns and provides delay improvements during both peak hour and off peak periods. However, the potential benefits of retrofit turnarounds should be examined carefully before construction.

APPENDIX B

Case Study Number 2

I-35 at Rittiman Road
San Antonio, Texas

Improvements:

-Ramp relocation
-Frontage road widening
-Traffic signal upgrading
-Frontage road turnaround

Conditions Before Construction

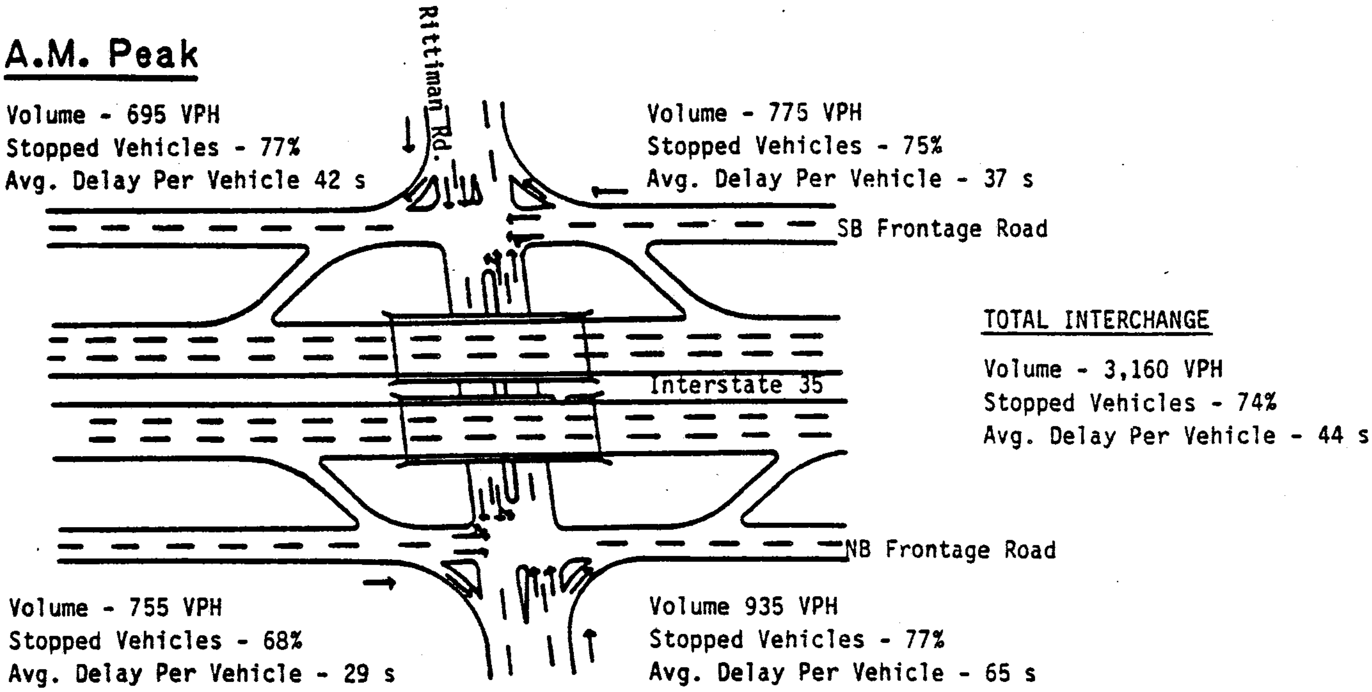
Rittiman Road intersects I-35 in northeast San Antonio. It is the next interchange south of the Eisenhauer interchange discussed in the previous case study. The interchange geometry and traffic conditions prior to the improvements are summarized in Figure B-1. The interchange was a typical urban diamond interchange with one-way frontage roads. A traffic-actuated signal controller with four-phase, two-overlap phasing was used.

Based on the average stopped delay per vehicle for the entire interchange, traffic operations were at level-of-service F during both the morning and evening peak hours. During the morning peak hour, vehicle delay was distributed fairly evenly among all approaches, with the heavy westbound Rittiman volume experiencing the highest average delay. Traffic into the east intersection was heavier during the evening peak, but average delays and the proportion stopping were higher on westside approaches. Overall, delay characteristics indicated that improvements to both sides of the interchange were required.

During the evening peak hour, northbound freeway speeds in the outside lane averaged 43 miles per hour approaching the Rittiman exit. This was about five miles per hour lower than speeds in the center lane and about ten miles per hour slower than the inside lane. The ramp operation influenced freeway speeds but did not create major freeway interference.

The turnaround volumes for the southbound to northbound movement were light with about 40 vehicles per hour during both the morning and evening peaks. However, the potential use of a south turnaround was relatively high with 185 vehicles and 230 vehicles during the morning and evening peak hours, respectively.

A.M. Peak



P.M. Peak

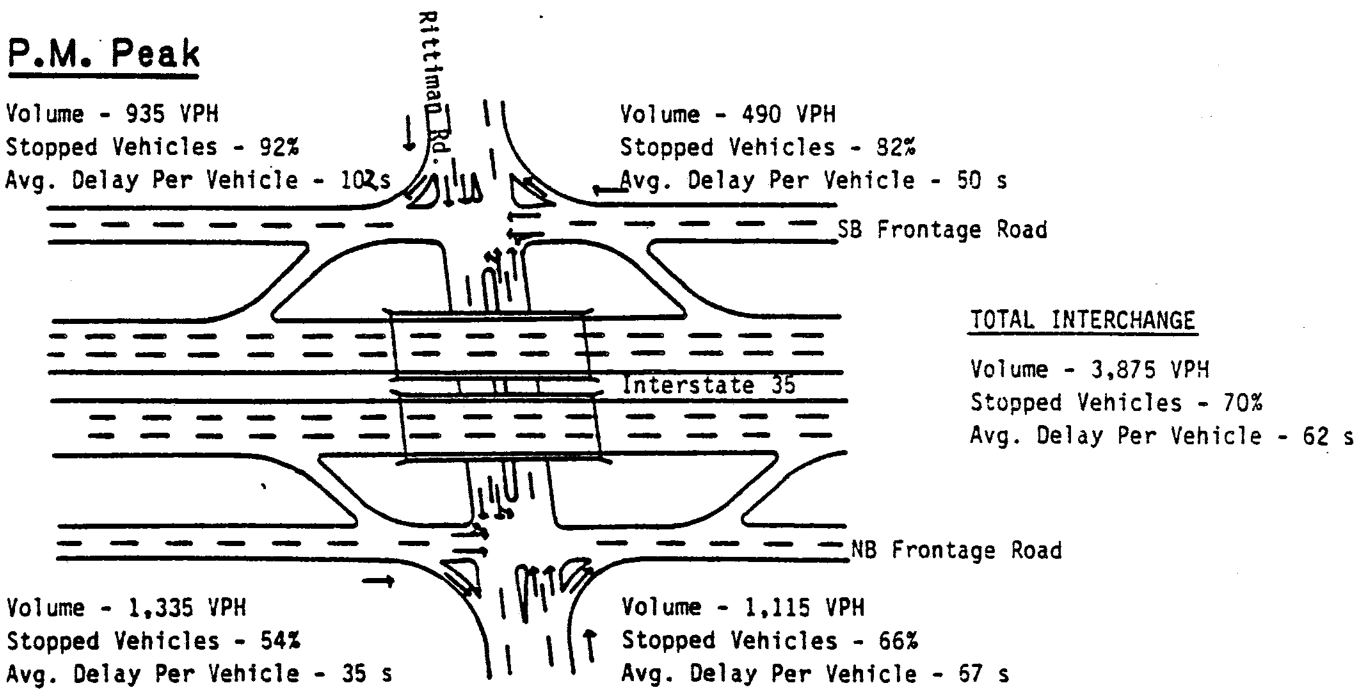


Figure B-1. Interchange Geometry and Operations Rittiman Road at I-35 "Before" Condition

Improvements

The improvements project at I-35 and Rittiman consisted of four elements.

1. The southbound exit ramp was relocated approximately 400 feet to the north and lengthened. The northbound exit ramp was located about 500 feet to the south and lengthened. An auxiliary lane was added between the I-410 entrance and the northbound exit at Rittiman.

2. A lane was added to both frontage roads approaching Rittiman. The new lanes are for left-turning vehicles only, with the former inside lanes now used for either left-turn or through movements.

3. The traffic signal was upgraded by the installation of a new controller and new vehicle detectors. This permits a more sophisticated variation of the four-phase, two-overlap phasing plan.

4. A frontage road turnaround was constructed on the south side of Rittiman, serving the northbound to southbound u-turn movement.

The contract cost for the construction project is summarized below:

Frontage road widening, ramp relocation, and auxiliary lane	$436,000
Signal Upgrading	29,000
Turnaround	338,000
TOTAL	$803,000

Conditions After Construction

The elements of the project were put into service in three phases to permit incremental evaluations. The relocated ramps, auxiliary lane, and widened frontage roads were opened first. Volume, delay, and speed data for this phase (After 1) were collected between December 7 and December 14, 1983. Next, the signal was upgraded (After 2). Data were collected on February 21 and 22, 1984. The turnaround was opened last (After 3), and data were collected from April 12 to April 18, 1984.

As discussed in the previous case study, traffic may not have fully adjusted to the new conditions during the incremental improvement phases. However, the data obtained during each phase should be fairly representative of the basic geometric conditions.

The interchange geometry and traffic conditions for the three improvement phases are shown in Figures B-2 to B-4.

During the After 1 phase, the average speed in the outside through lane approaching the northbound Rittiman exit had increased from 43 miles per hour to 50 miles per hour. During the After 2 phase, the average speed in this location had dropped to 45 miles per hour.

Discussion of Findings

Traffic volumes and delay data are summarized for the Before condition and the three After phases in Tables B-1 and B-2.

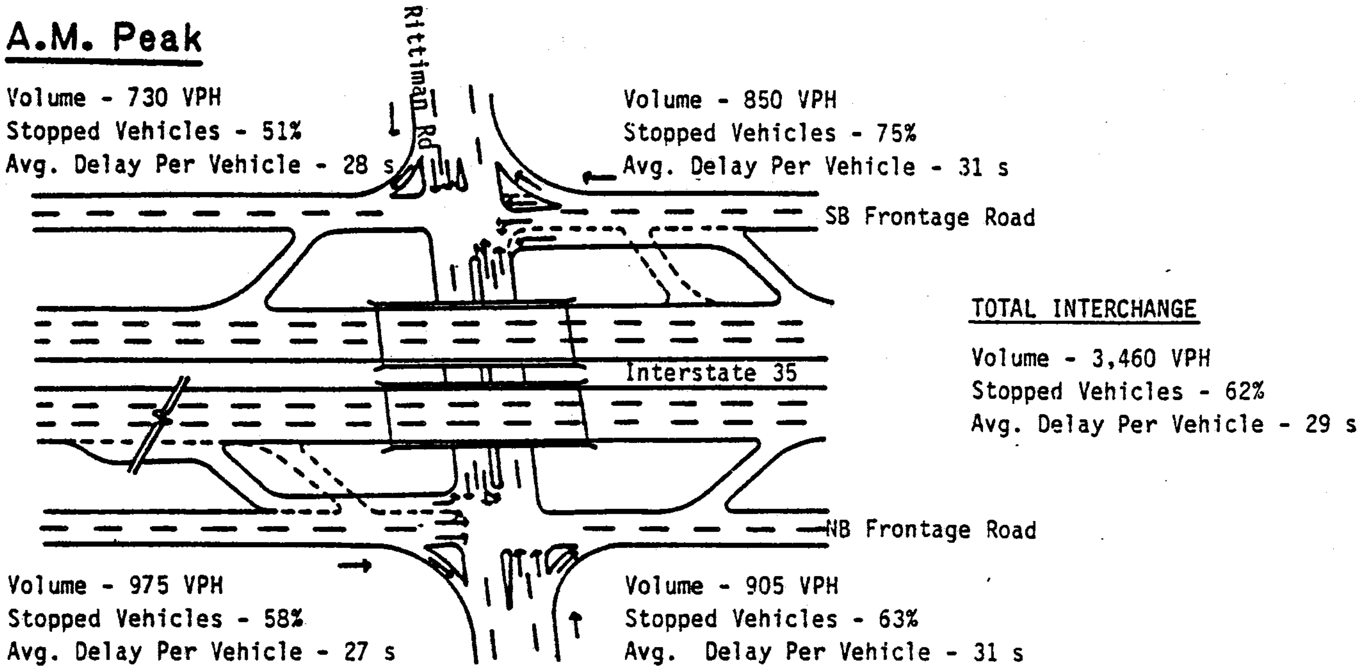
The first phase of improvements produced the greatest delay reduction at this interchange. Although total traffic volumes during both the morning and evening peaks grew about nine percent between the Before and After cases, average stopped delay per vehicle dropped by about 35 percent during both periods.

The signal upgrading and turnaround each reduced total interchange delay moderately. The effects were more substantial during the more highly congested evening peak hour.

Taken together, the improvement projects reduced the average delay during the morning peak hour by 43 percent, to level-of-service D, despite a 20 percent increase in total volume. A 60 percent reduction in evening peak hour delay improved operations to level-of-service D.

Effects on freeway speeds were mixed. The first phase improvements increased speeds significantly in the northbound outside lane during the

A.M. Peak



P.M. Peak

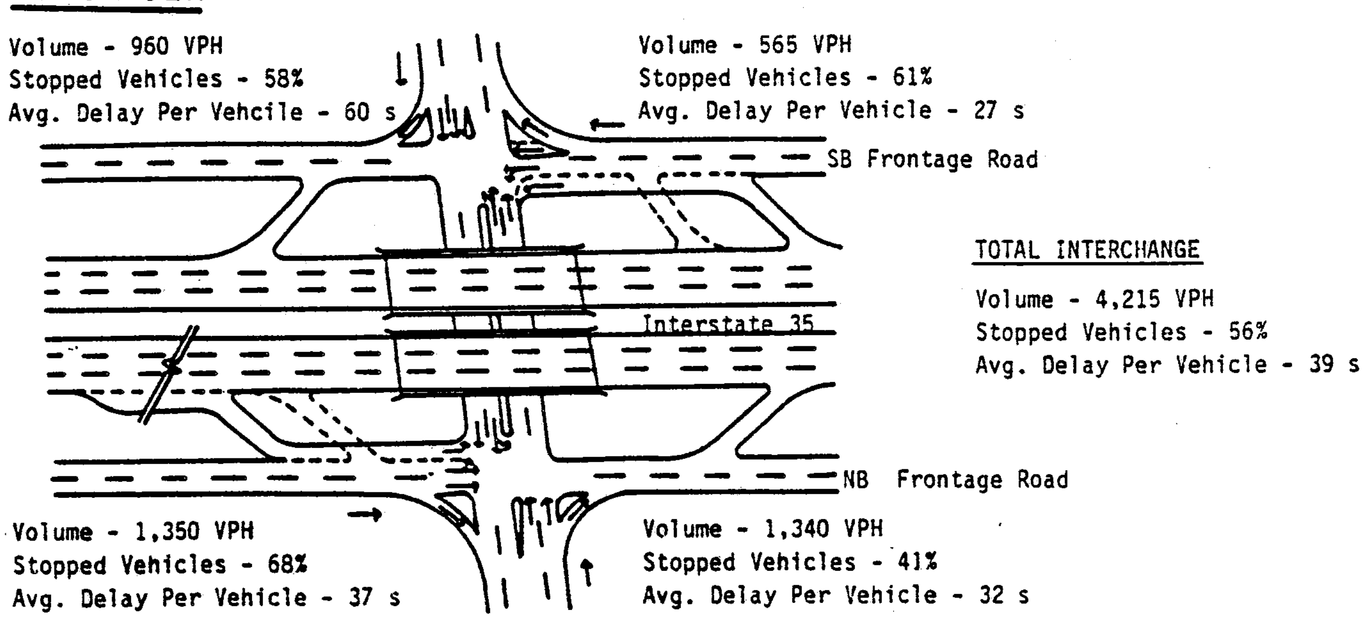
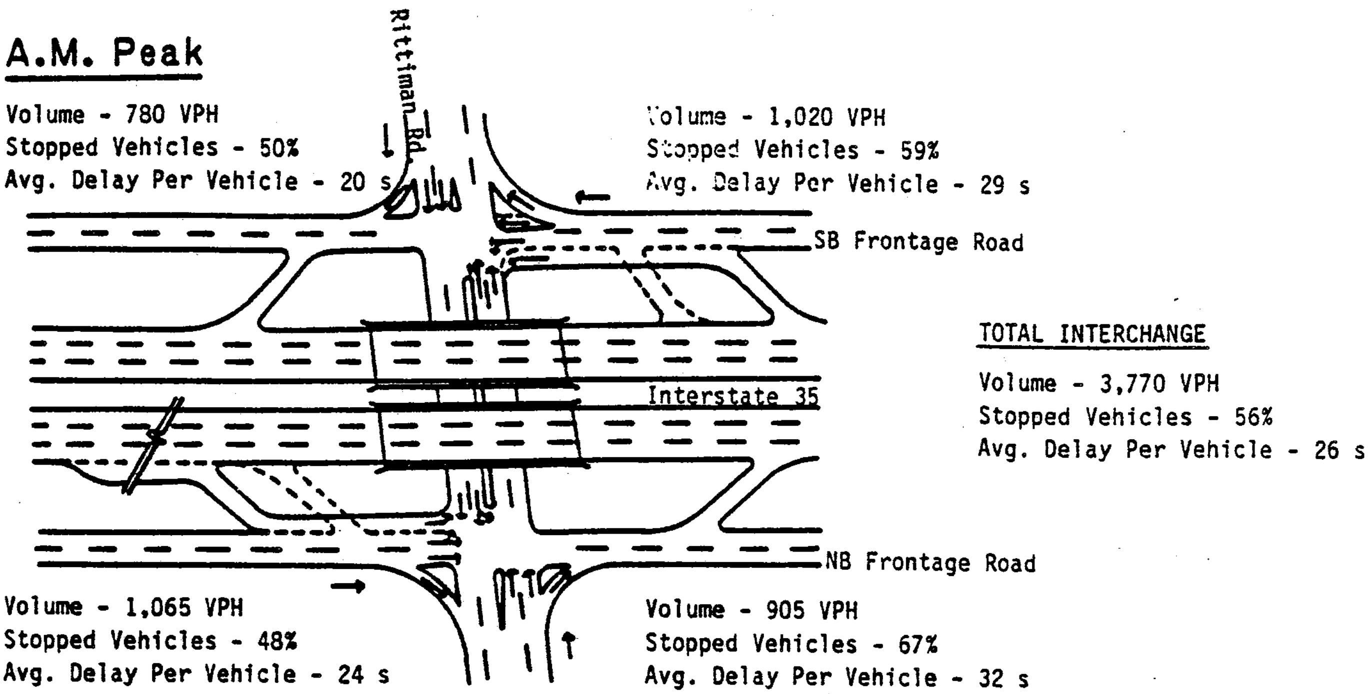


Figure B-2. Interchange Geometry and Operations Rittiman Road at I-35 "After 1" Condition

- Frontage Road Widening
- Ramp Relocation



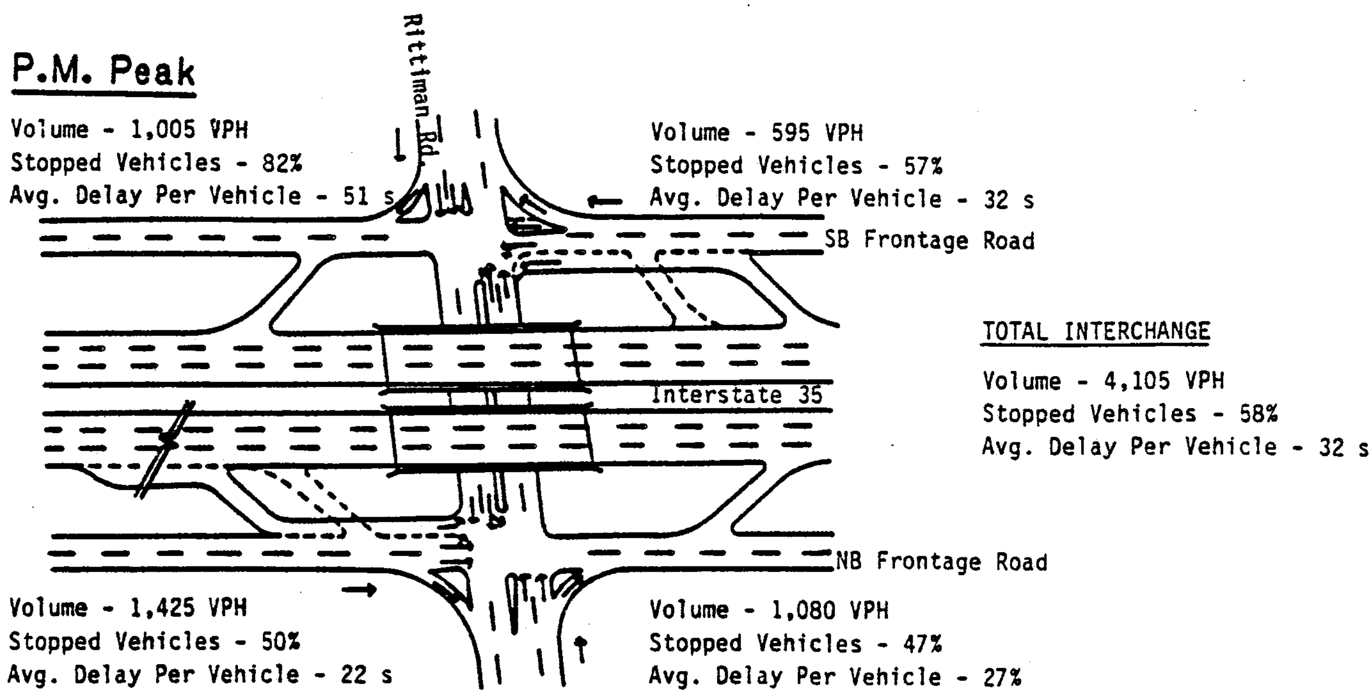


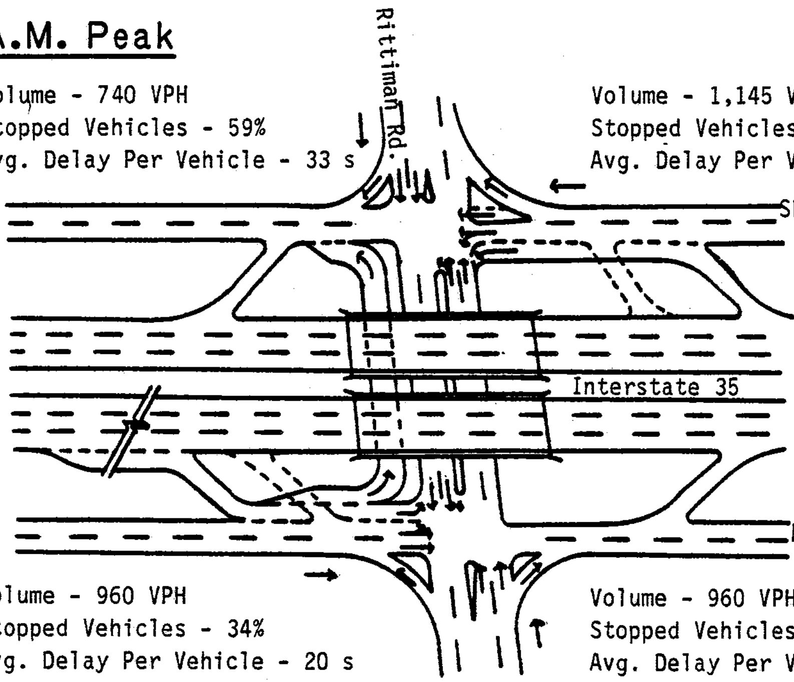
Figure B-3. Interchange Geometry and Operations Rittiman Road at I-35 "After 2" Condition

- Frontage Road Widening
- Ramp Relocation
- Signal Upgrading

A.M. Peak

Volume - 740 VPH
Stopped Vehicles - 59%
Avg. Delay Per Vehicle - 33 s

Volume - 1,145 VPH
Stopped Vehicles - 55%
Avg. Delay Per Vehicle - 35 s



TOTAL INTERCHANGE

Volume - 3,805 VPH
Stopped Vehicles - 50%
Avg. Delay Per Vehicle - 25 s

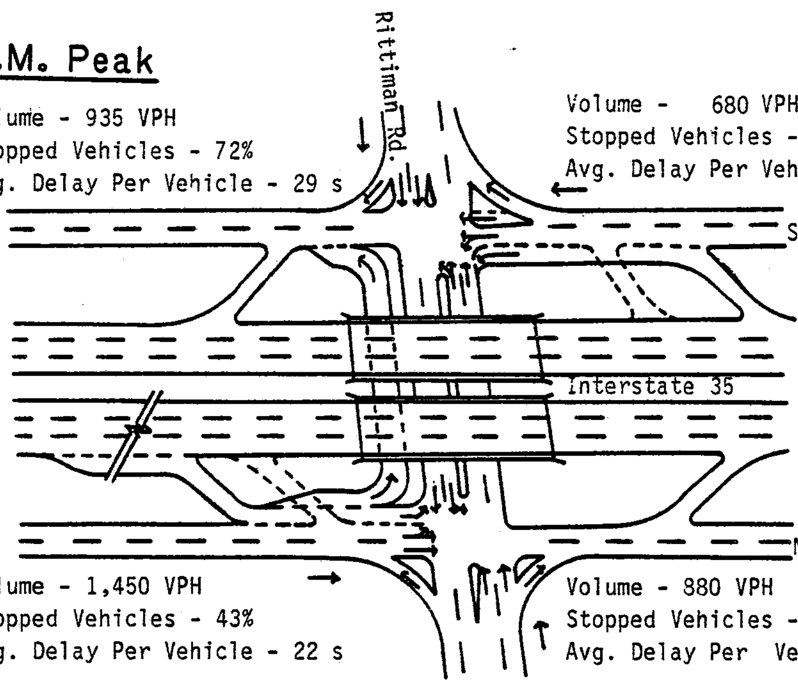
Volume - 960 VPH
Stopped Vehicles - 34%
Avg. Delay Per Vehicle - 20 s

Volume - 960 VPH
Stopped Vehicles - 51%
Avg. Delay Per Vehicle - 14 s

P.M. Peak

Volume - 935 VPH
Stopped Vehicles - 72%
Avg. Delay Per Vehicle - 29 s

Volume - 680 VPH
Stopped Vehicles - 54%
Avg. Delay Per Vehicle -



TOTAL INTERCHANGE

Volume - 3945 VPH
Stopped Vehicles - 58%
Avg. Delay Per Vehicle - 27 s

Volume - 1,450 VPH
Stopped Vehicles - 43%
Avg. Delay Per Vehicle - 22 s

Volume - 880 VPH
Stopped Vehicles - 63%
Avg. Delay Per Vehicle - 24 s

Figure B-4. Interchange Geometry and Operations Rittiman Road at I-35
"After 3" Condition

- Frontage Road Widening
- Ramp Relocation
- Signal Upgrading
- Turnaround

TABLE B-1. INTERSECTION DELAY SUMMARY

Location: Rittiman Road at I-35
Time: A.M. Peak Hour

	Before	Relocate Ramps, Widen Frontage Roads, Add Auxiliary Lane (After 1)	+ Signal Upgrade (After 2)	+ Turnaround (After 3)
Approach: NB Frontage Road				
Total Volume	755	975	1,065	960
% Delayed Vehicles	68%	58%	48%	34%
Vehicle-Seconds of Delay	21,840	26,310	25,065	19,020
Average Delay per Stopped Vehicle	43	47	49	58
Average Delay per Vehicle	29	27	24	20
Approach: SB Frontage Road				
Total Volume	775	850	1,020	1,145
% Delayed Vehicles	75%	75%	59%	55%
Vehicle-Seconds of Delay	28,585	26,160	29,145	39,960
Average Delay per Stopped Vehicle	49	41	48	63
Average Delay per vehicle	37	31	29	35
Approach: EB Rittiman				
Total Volume	697	730	780	740
% Delayed Vehicles	77%	51%	50%	59%
Vehicle-Seconds of Delay	29,940	20,790	15,630	24,405
Average Delay per Stopped Vehicle	54	56	40	56
Average Delay per Vehicle	42	28	20	33
Approach: WB Rittiman				
Total Volume	935	905	905	960
% Delayed Vehicles	77%	63%	67%	51%
Vehicle-Seconds of Delay	60,785	28,185	29,265	13,635
Average Delay per Stopped Vehicle	84	49	48	28
Average Delay per Vehicle	65	31	32	14
Total Intersection				
Total Volume	3,160	3,460	3,770	3,805
% Delayed Vehicles	74%	62%	56%	50%
Vehicle-Seconds of Delay	140,150	101,445	99,105	97,020
Average Delay per Stopped Vehicle	60	47	47	51
Average Delay per Vehicle	44	29	26	25

TABLE B-2. INTERSECTION DELAY SUMMARY

Location: Rittiman at I-35
Time: P.M. Peak Hour

	Before	Relocate Ramps, Widen Frontage Roads, Add Auxiliary Lane (After 1)	+ Signal Upgrade (After 2)	+ Turnaround (After 3)
Approach: NB Frontage Road				
Total Volume	1,335	1,350	1,425	1,450
% Delayed Vehicles	54%	68%	50%	43%
Vehicle-Seconds of Delay	46,225	49,785	31,065	31,425
Average Delay per Stopped Vehicle	64	54	44	50
Average Delay per Vehicle	35	37	22	22
Approach: SB Frontage Road				
Total Volume	490	565	595	680
% Delayed Vehicles	82%	61%	57%	54%
Vehicle-Seconds of Delay	24,460	15,420	19,080	21,195
Average Delay per Stopped Vehicle	61	45	56	58
Average Delay per Vehicle	50	27	32	31
Approach: EB Rittiman				
Total Volume	935	960	1,005	935
% Delayed Vehicles	92%	58%	82%	72%
Vehicle-Seconds of Delay	95,280	57,240	51,045	27,225
Average Delay per Stopped Vehicle	111	103	62	41
Average Delay per Vehicle	102	60	51	29
Approach: WB Rittiman				
Total Volume	1,115	1,340	1,080	880
% Delayed Vehicles	66%	41%	47%	63%
Vehicle-Seconds of Delay	74,355	42,570	29,325	20,685
Average Delay per Stopped Vehicle	101	77	58	37
Average Delay per Vehicle	67	32	27	24
Total Intersection				
Total Volume	3,875	4,215	4,105	3,945
% Delayed Vehicles	70%	56%	58%	58%
Vehicle-Seconds of Delay	240,320	165,015	130,515	100,530
Average Delay per Stopped Vehicle	89	70	55	45
Average Delay per Vehicle	62	39	32	25

evening peak. It was not possible to determine how much of this improvement was due to the auxiliary lane and how much was due to the ramp relocation. Because the before speeds did not indicate extensive ramp queueing onto the freeway (which a ramp relocation would solve), it is assumed that the auxiliary lane was primarily responsible for the speed improvement. Speeds decreased at this location after the signal upgrading. The signal project had substantial, positive effects on delay and stopping on the northbound frontage road and, therefore, could not have been the cause of this speed reduction. Other aspects of the freeway's operation must have been responsible.

User costs based on delay and idling are summarized in Table B-3. As in the previous case study, delay estimates for the before and after cases were adjusted to represent constant traffic volumes. The estimated user cost savings over the life of the project are conservative because they represent delay savings which remain constant over the life of the improvement.

Implications For Other Sites

The first increment of improvement -- the frontage road widening and ramp relocation -- produced significant delay reductions on the frontage roads but even greater improvements on the Rittiman Road approaches. This phase of the improvement reduced the volume per lane on the frontage roads which in turn, with traffic-actuated control, diminished the frontage road traffic's signal demand. Thus, the percentage of vehicles stopped and the average delay per vehicle dropped dramatically for the Rittiman approaches during both peak periods.

The actual use of the northbound-to-southbound turnaround has been 40 to 50 percent greater than expected. Recent counts showed 265 vehicles using the turnaround during the morning peak hour and 350 u-turning vehicles during the evening peak hour. The volume of this movement has increased by a much higher

TABLE B-3. ECONOMIC ANALYSIS - RITTIMAN ROAD AT I-35

	Delay and Idling Costs[1]				User Benefits			
	Before[2]		After					
	Daily	Annual	Daily	Annual	Annual	Present[3] Value	Construction Cost	B:C Ratio
Ramp Relocation, Auxiliary Lane, and Frontage Road Widening	$5,577	$1,394,300	$3,647	$911,700	$482,600	$4,108,700	$436,000	9.4:1
Signal Upgrading	3,704	925,900	3,176	794,000	131,900	1,122,900	29,000	38.7:1
Turnaround	3,116	778,900	2,705	676,200	102,700	874,300	338,000	2.6:1
Rittiman Total					$717,200	$6,105,900	$803,000	7.6:1

[1]Based on estimated delay during 12 highest-volume hours per day, 250 working days per year.
[2]Delay for before cases adjusted to represent traffic volumes after the improvement
[3]20-year functional life, 10% discount rate

proportion than the total interchange volume. This indicates that drivers have selected the turnaround over a previous route which has become less desirable. The relatively large volume of u-turn traffic before construction, despite the lengthy delays at the interchange, was an indicator of this latent demand. When a large number of drivers are willing to negotiate a highly-congested interchange for a u-turn movement, there is probably a substantial number which are selecting an alternate routing, assuming that a reasonable alternative exists. This traffic represents latent demand for a turnaround.

APPENDIX C

Case Study Number 3

Terminal Drive
San Antonio, Texas

Improvement:

-Addition of partial interchange

Conditions Before Construction

Terminal Drive connects the San Antonio International Airport to U.S. 281 on the west side of the airport complex. Before the construction project, Terminal Drive terminated, in a tee-intersection with the northbound frontage road of U.S. 281 (see Figure C-1).

The improvement project at Terminal Drive and U.S. 281 was foreseen to have operational impacts over the study area shown in Figure C-1. Traffic volumes and average delays per vehicle at the interchanges in this area are shown in Figures C-2 and C-3, for the Before condition. These data were collected between August 31 and September 22, 1982.

The interchange of Airport Boulevard and I-410 was the most-congested interchange in the study area. Based on the average stopped delay per vehicle for the exterior approaches to the interchange, Airport/I-410 operated at level of service E during the morning peak hour, and level of service F during the evening peak hour. The interchange of Jones-Maltsberger and I-410 operated at level of service C during the morning peak hour and level of service D during the evening peak hour. The Jones-Maltsberger/U.S. 281 interchange provided level of service B during the morning peak hour and level of service C during the evening peak hour.

The network did not provide convenient access from U.S. 281 to the airport. Southbound drivers on U.S. 281 would use either the congested Airport/I-410 interchange or a circuitous route through the airport property originating at Isom Road and U.S. 281. Southbound traffic leaving the airport had to pass through the Airport/I-410 interchange to enter southbound U.S. 281 via Airport.

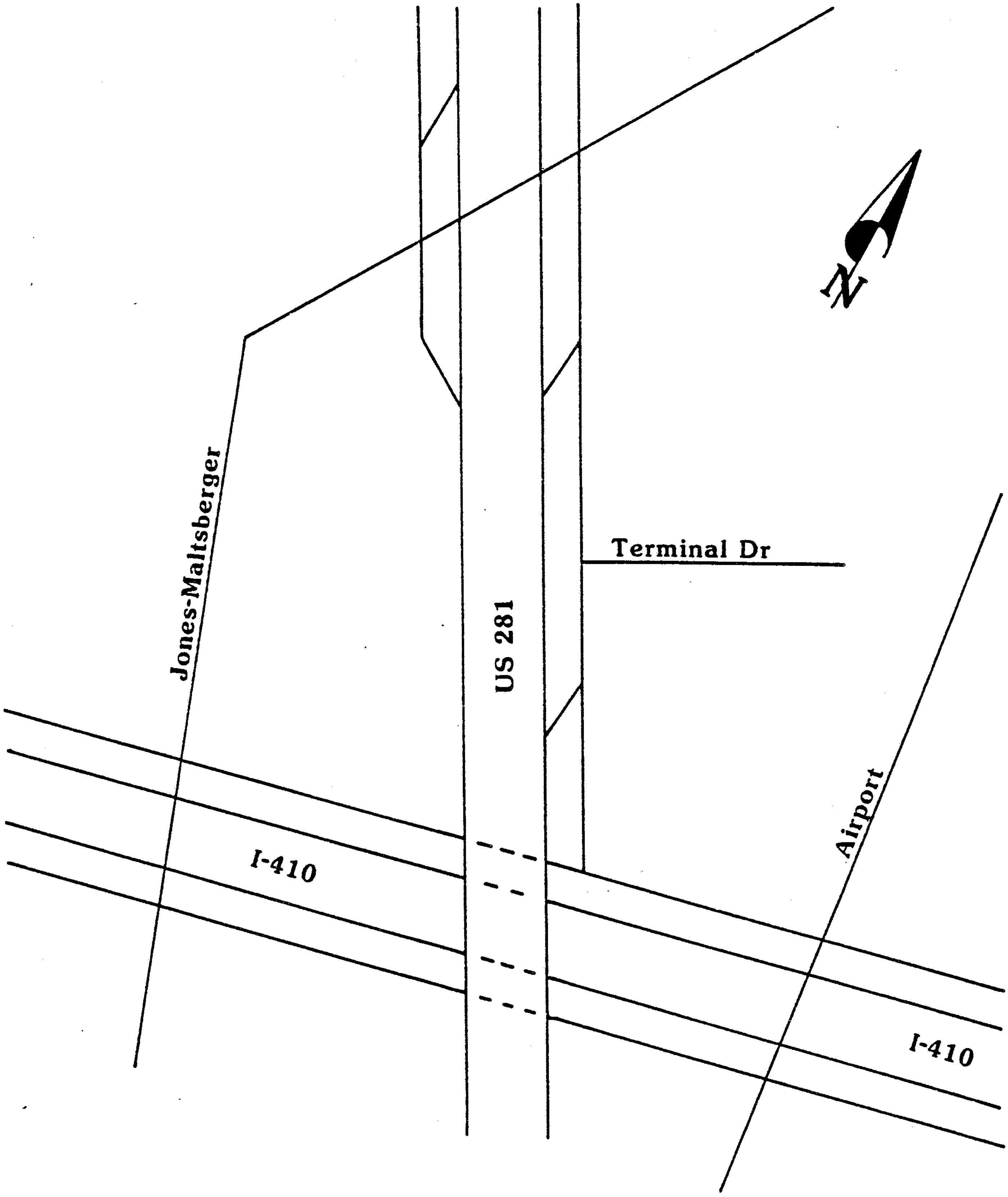


Figure C-1. Terminal Drive Study Area Before Construction

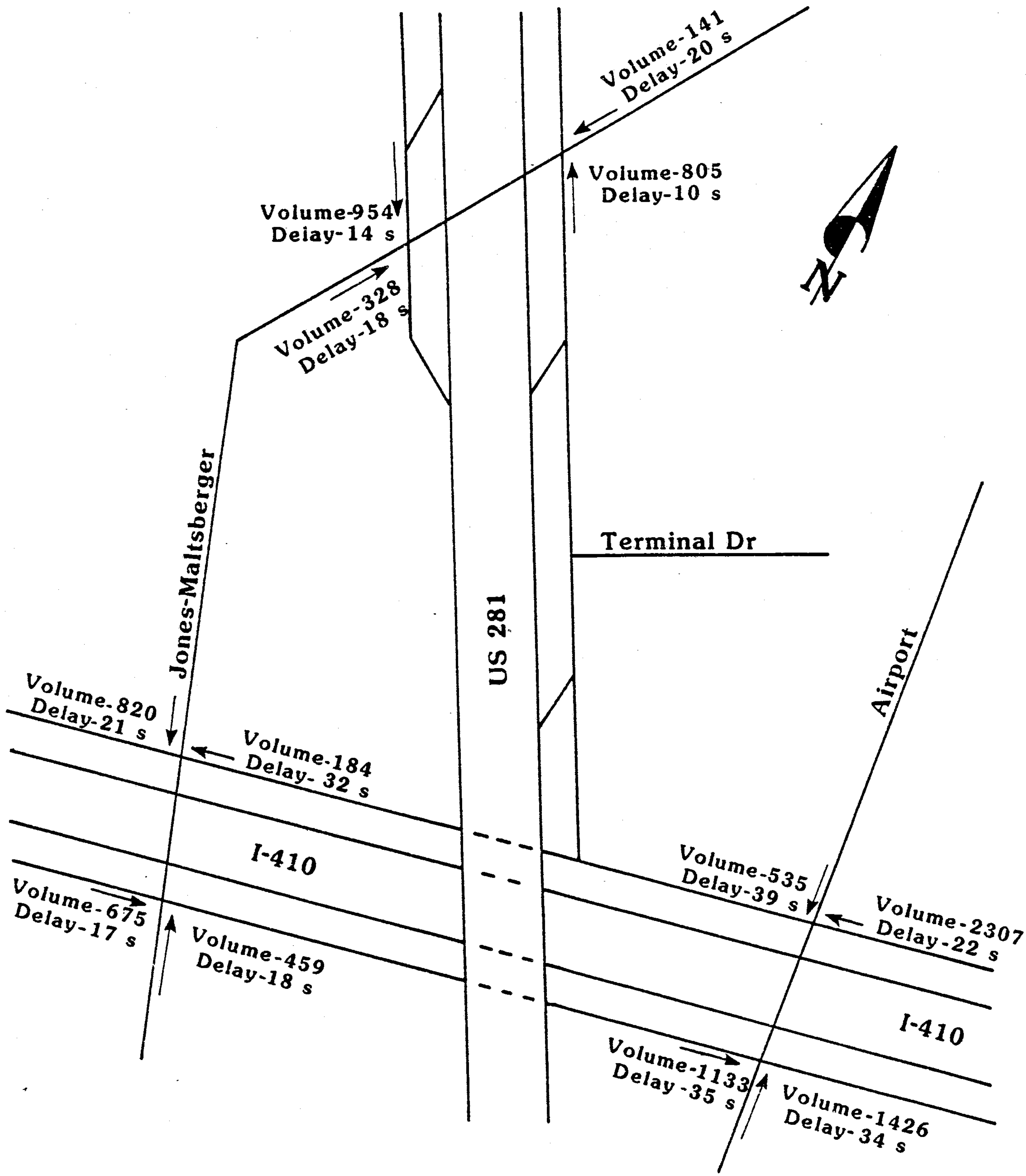


Figure C-2. A.M. Peak Hour Volume and Delay Terminal Drive Study Area Before Construction

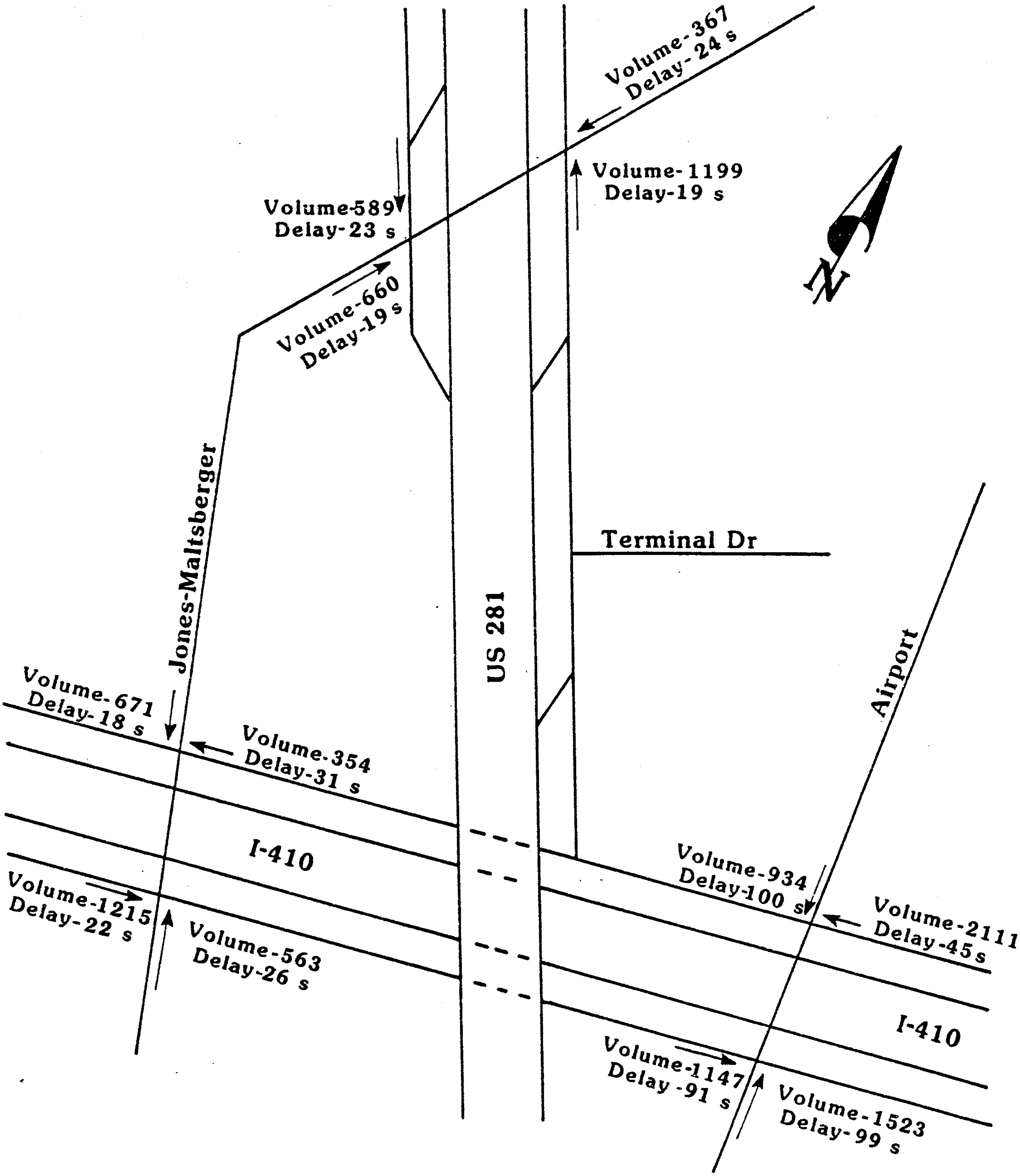


Figure C-3. P.M. Peak Hour Volumes and Delay Terminal Drive Study Area Before Construction

Improvements

The improvement project at Terminal Drive and U.S. 281 consisted of several elements.

1. Terminal Drive was extended over U.S. 281 to an intersection with the southbound frontage road.
2. The southbound U.S. 281 frontage road was extended southerly to intersect the westbound I-410 frontage road.
3. A southbound exit ramp from U.S. 281 was built south of Jones-Maltsberger Road, replacing the southbound entrance ramp at that location.
4. A southbound entrance ramp to U.S. 281 was added south of the new Terminal Drive connection.

The total project is shown schematically in Figure C-4.

The primary purpose of the project was to provide direct access to the airport from southbound U.S. 281, replacing the two less desirable routes described earlier. The secondary purpose was to divert some traffic from the Airport/I-410 interchange. Besides being the primary access to the airport, this interchange is a major connection between I-410 and U.S. 281. The Terminal Drive connection would divert that portion of the southbound Airport Boulevard traffic that is bound for downtown San Antonio. Finally, the extension of the southbound U.S. 281 frontage road would have secondary impacts on the two Jones-Maltsberger interchanges. Previously, southbound U.S. 281 traffic bound for I-410 would use Jones-Maltsberger as the connection, traveling through both interchanges. The frontage road connection was expected to decrease volume at the Jones-Maltsberger/U.S. 281 interchange and redistribute traffic at the Jones-Maltsberger/I-410 interchange (see Figure C-5).

Construction began in September, 1982 and was completed in September, 1983 at a contract cost of $2,011,000.

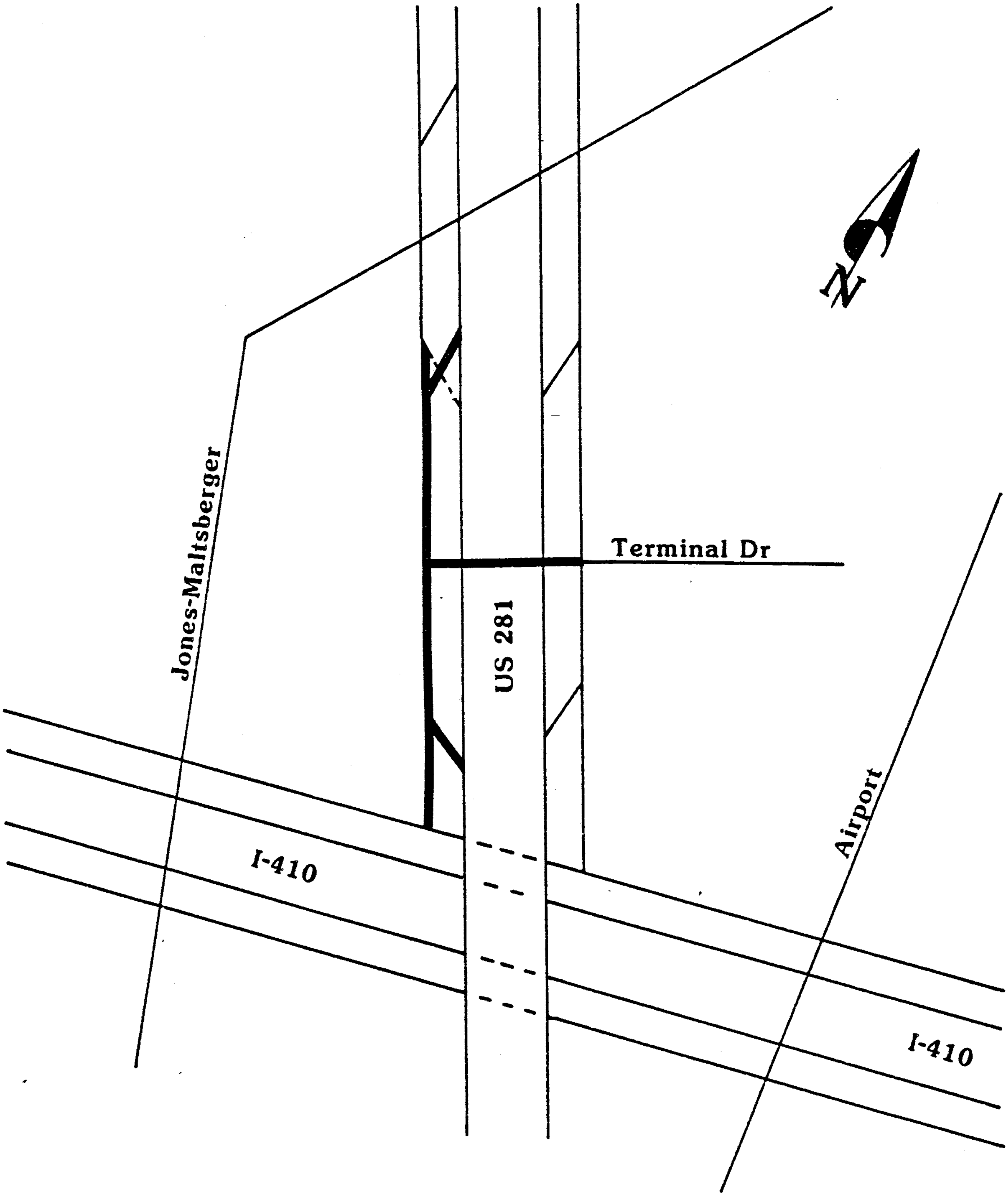
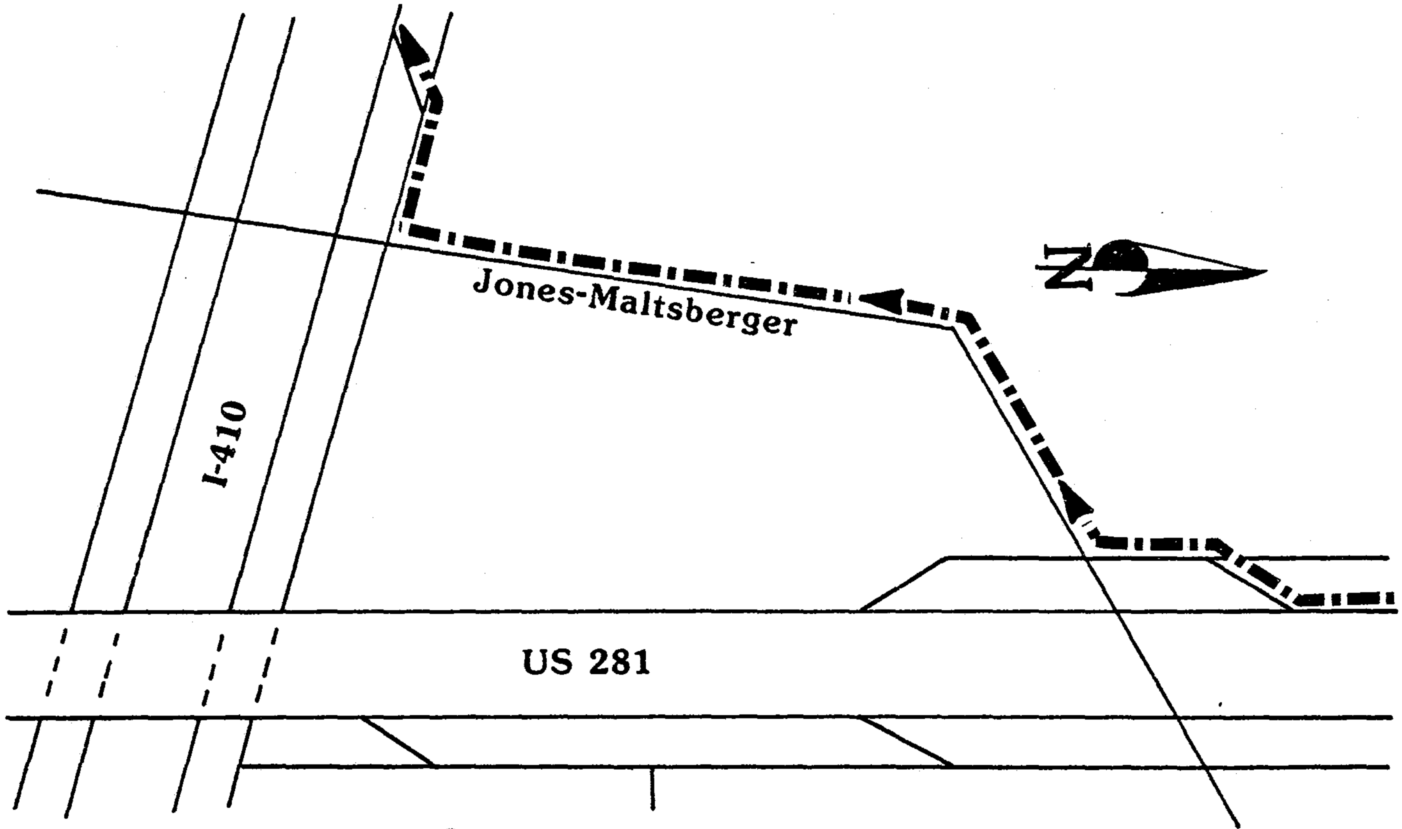
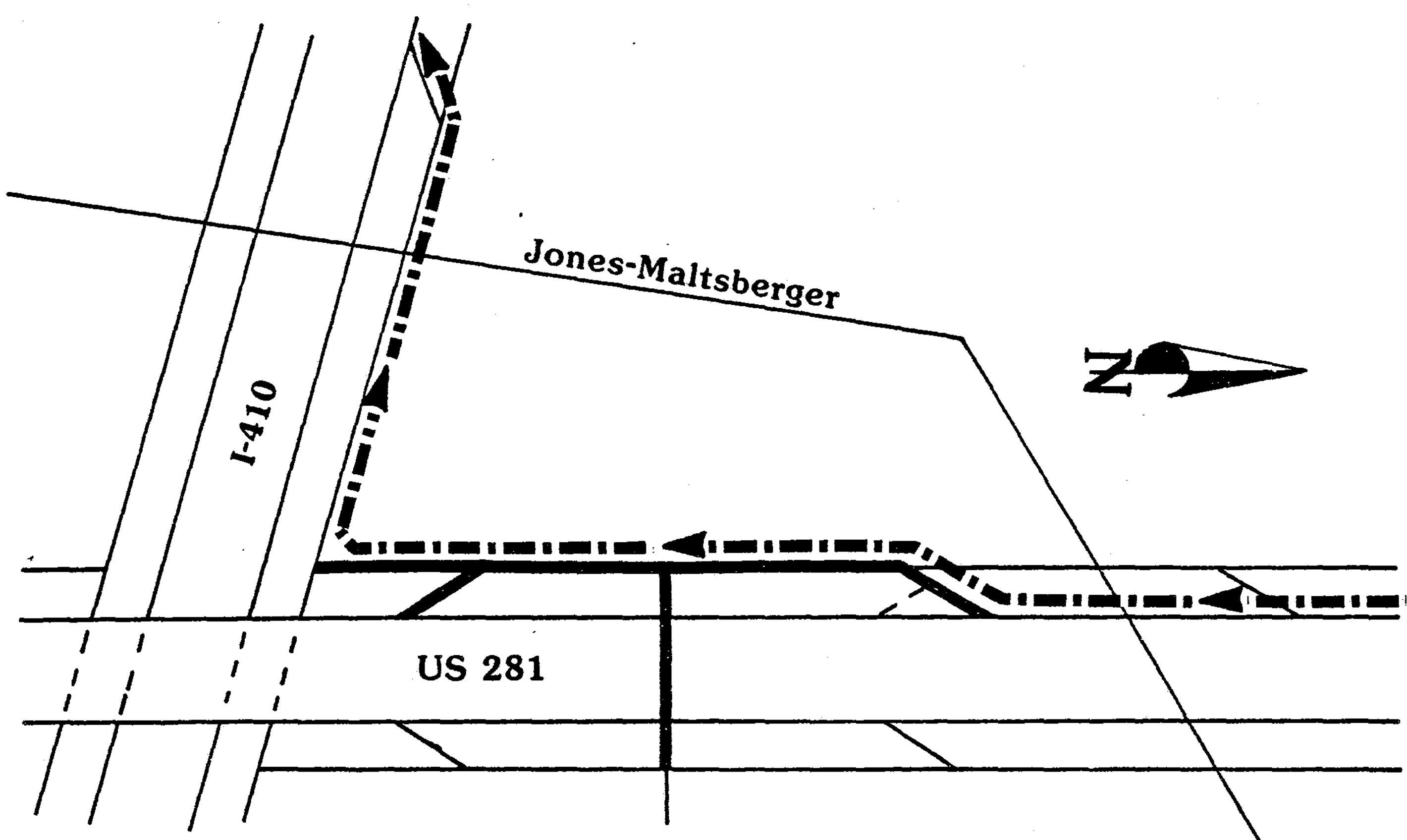


Figure C-4. Terminal Drive Study Area After Construction



Pre-construction Traffic Pattern



Post-construction Traffic Pattern

Figure C-5. Diversion from Jones-Maltsberger to Frontage Roads

Conditions After Construction

Peak hour volumes and average delay per vehicle for the study area interchanges after the project was constructed are shown in Figures C-6 and C-7. The data shown were collected between September 27 and November 16, 1983.

Discussion of Findings

Table C-1 summarizes volume and operational data before and after the Terminal Drive project. For the total study area, morning peak hour traffic increased 12 percent, but the average delay per vehicle increased only one second (4%). Impacts during the evening peak were more positive. Average delay per vehicle decreased by seven seconds (14%) despite a 15 percent increase in traffic.

Some of the intended rerouting of traffic occurred; other diversions resulted which were not anticipated. During the morning peak hour, 422 vehicles turned east from the southbound U.S., 281 frontage road and used the new Terminal Drive extension to enter the airport. A license plate survey was conducted to determine the routing of this traffic. The survey revealed that about 150 of these vehicles were bound for destinations in and about the airport. In addition, 45 vehicles and 155 vehicles in the morning and evening peak hours, respectively, traveled west on the extension to the southbound frontage road and avoided the Airport/I-410 interchange. These two groups realized the benefits of the direct access provided by the project.

However, as shown in Figure C-8, about 270 of the morning eastbound vehicles on the Terminal Drive extension turned south onto Airport Boulevard, entered the Airport/I-410 interchange, and turned left onto the eastbound I-410 frontage road. This unintended diversion produced the growth in southbound traffic on Airport during both peak periods.

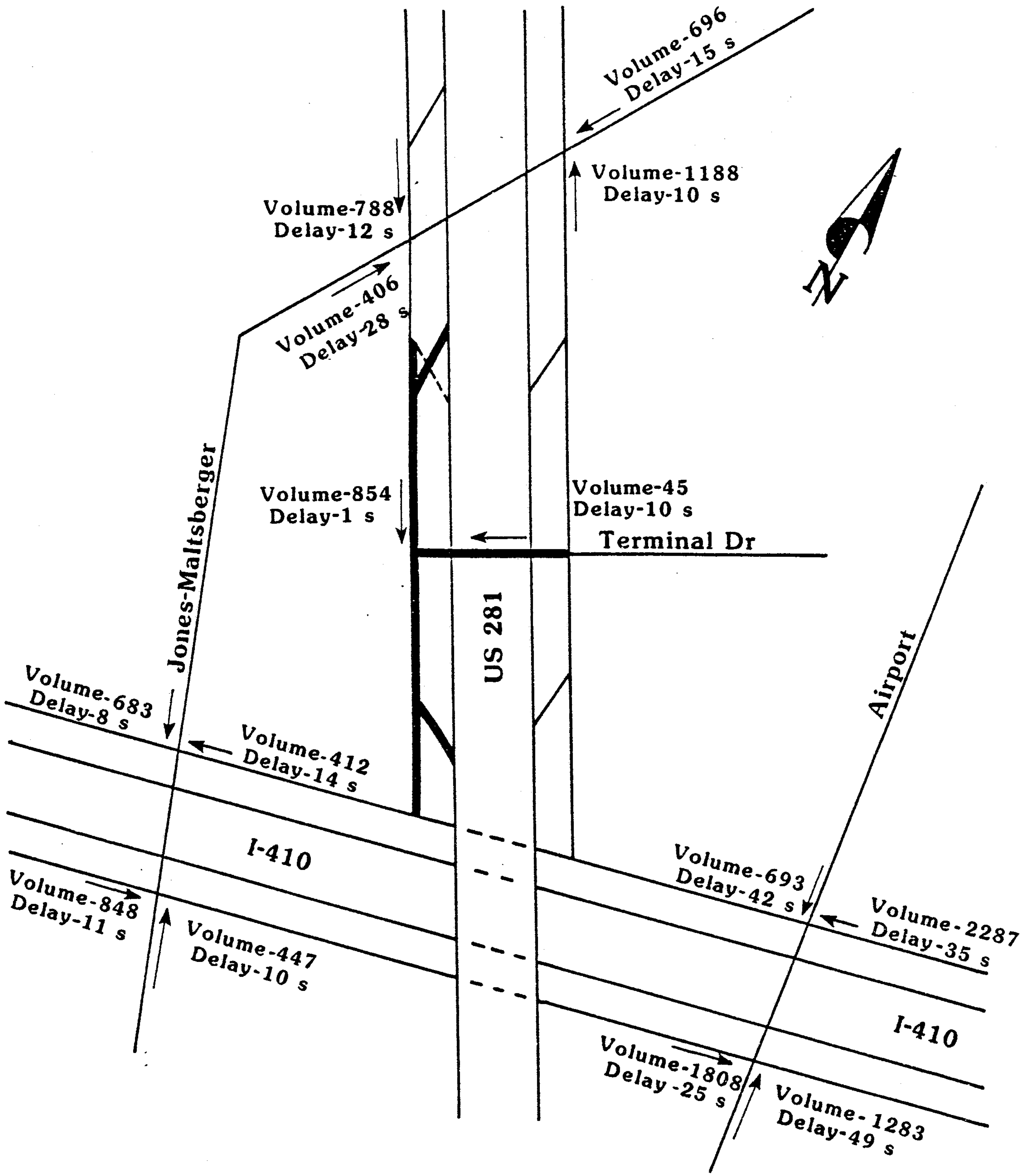


Figure C-6. A.M. Peak Hour Volume and Delay Terminal Drive Study Area After Construction

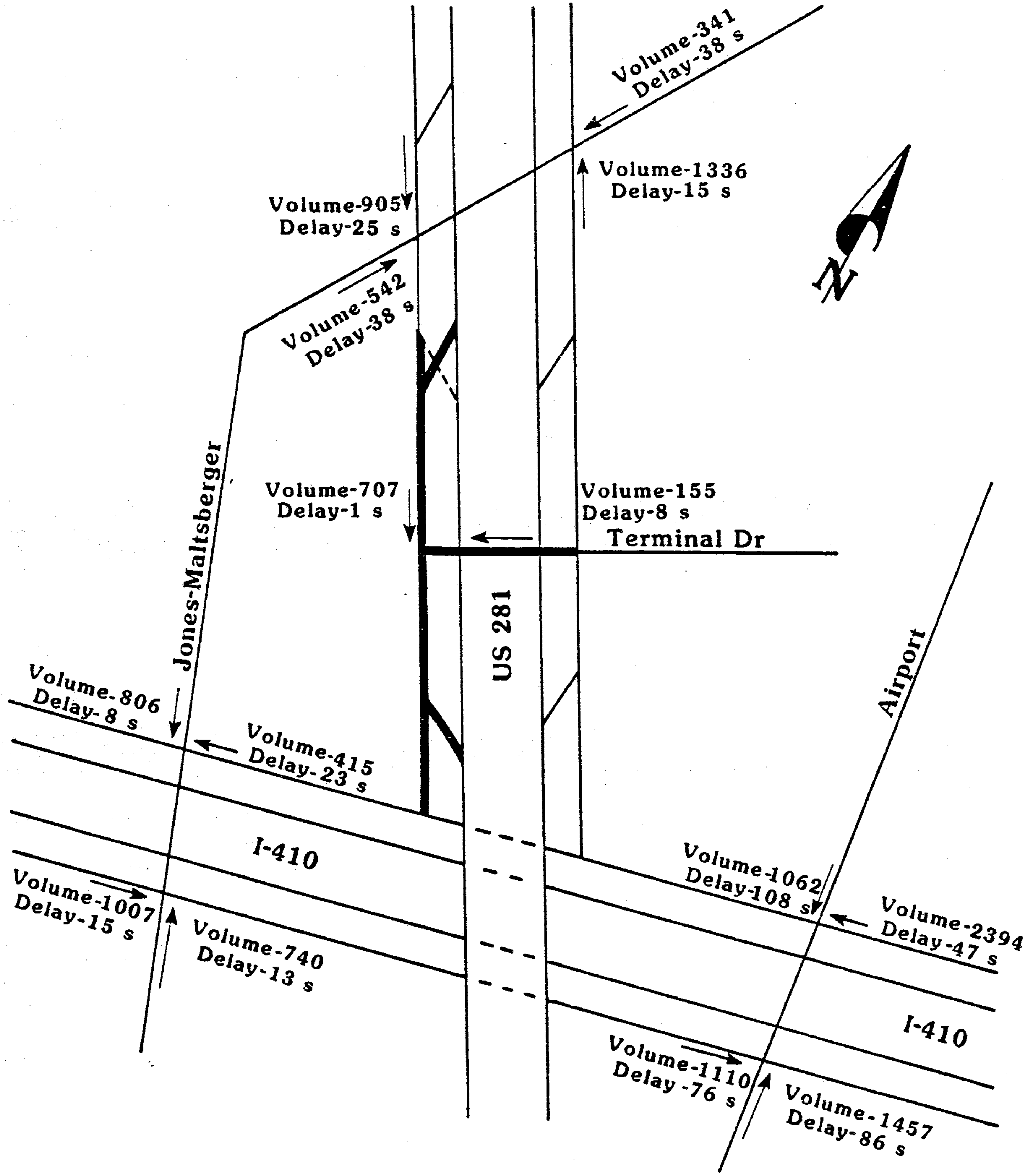


Figure C-7. P.M. Peak Hour Volume and Peak Terminal Drive Study Area After Construction

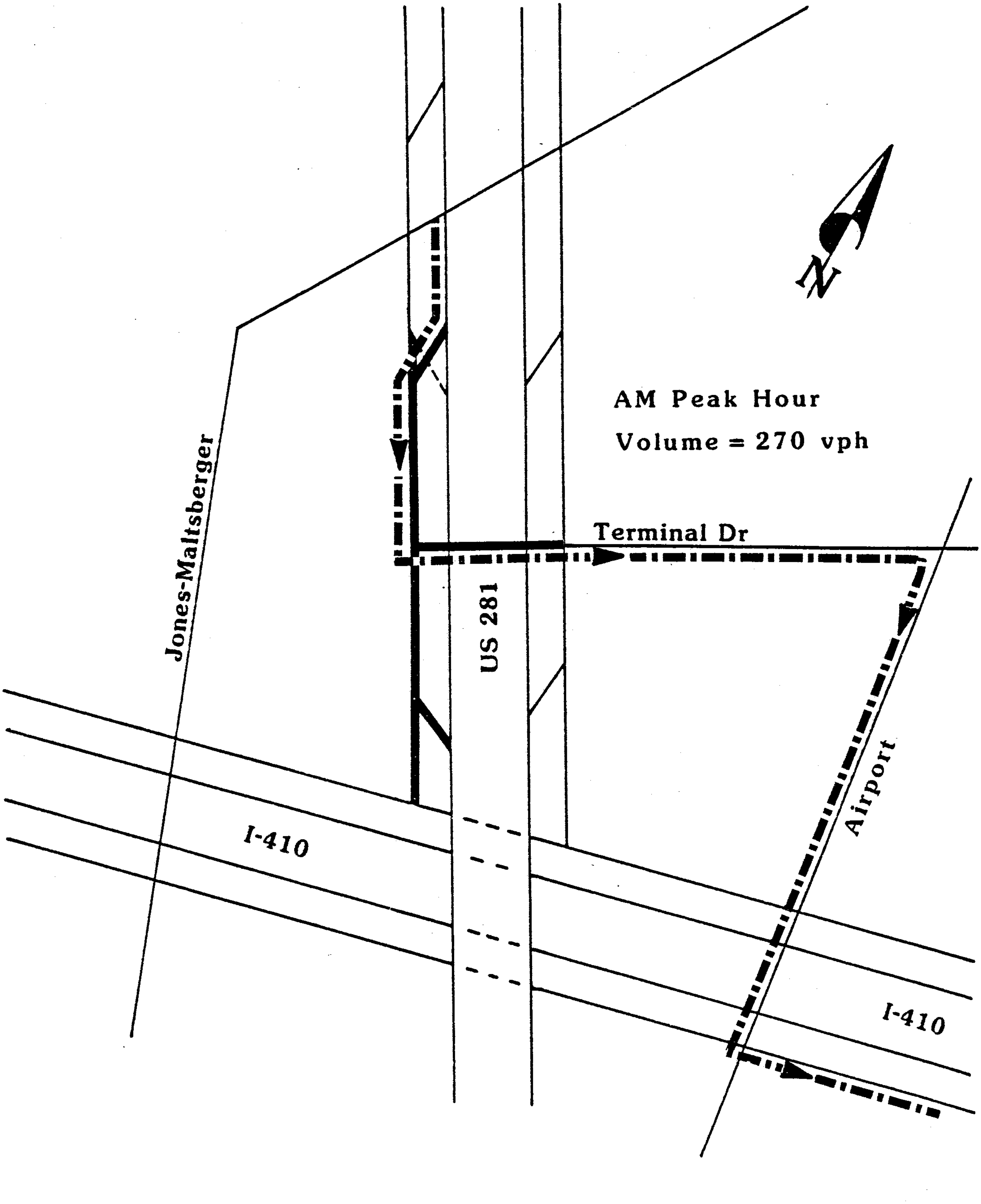


Figure C-8. Southbound U.S. 281 to Eastbound I-410 Routing After Construction

TABLE C-1. INTERSECTION DELAY SUMMARY

Location: Terminal Drive Study Area	AM Peak		PM Peak	
	Before	After	Before	After
Airport/I-410				
Total Volume	5,400	5,170	5,715	6,025
% Delayed Vehicles	75%	71%	69%	77%
Vehicle-Seconds of Delay	160,845	216,450	444,130	436,080
Average Delay per Stopped Vehicle	40	59	112	94
Average Delay per Vehicle	30	42	78	72
Jones-Maltsberger/U.S. 281				
Total Volume Count	2,230	2,480	2,815	3,125
% Delayed Vehicles	67%	62%	72%	70%
Vehicle-Seconds of Delay	29,610	34,560	58,375	76,170
Average Delay per Stopped Vehicle	20	23	29	35
Average Delay per Vehicle	13	14	21	24
Jones-Maltsberger/I-410				
Total Volume	2,140	2,410	2,805	2,970
% Delayed Vehicles	59%	52%	66%	62%
Vehicle-Seconds of Delay	42,735	24,835	64,385	40,710
Average Delay per Stopped Vehicle	34	20	35	22
Average Delay per Vehicle	20	10	23	14
Terminal/U.S. 281				
Total Volume	0	900	0	860
% Delayed Vehicles	-	15%	-	24%
Vehicle-Seconds of Delay	0	1,355	0	1,755
Average Delay per Stopped Vehicle	0	10	0	8
Average Delay per Vehicle	0	1	0	2
Total Study Area				
Total Volume	9,770	10,960	11,335	12,980
% Delayed Vehicles	69%	60%	69%	69%
Vehicle-Seconds of Delay	233,190	277,200	566,890	554,715
Average Delay per Stopped Vehicle	34	42	72	62
Average Delay per Vehicle	24	25	50	43

Substantial volumes used the extension of the southbound U.S. 281 frontage road to the westbound I-410 frontage road, reducing the use of Jones-Maltsberger as a connector between the freeways. The most apparent benefits of this connection occurred at the Jones-Maltsberger/I-410 interchange. Traffic was shifted from southbound Jones-Maltsberger to the previously-underutilized, westbound frontage road. This redistribution of demand largely accounted for a 50 percent reduction in average delay during the morning peak and a 39 percent cut during the evening at this interchange.

User costs based on delay and idling are summarized in Table C-2. The resulting benefit:cost ratio is 0.7:1. However, this analysis understates user benefits in several ways. First, delay savings for only the 12 highest hourly volumes of the day, on weekdays, were included. Second, it is assumed that the annual reduction in delay will remain constant over the entire functional life of the project. As traffic grows and congestion increases, more traffic will be diverted to the new facility. Because the Terminal Drive/U.S. 281 interchange was underutilized relative to other interchanges in the area, increased traffic volumes at that location would be expected to create less marginal delay than at the Airport/I-410 interchange. Therefore, the delay reduction attributable to the project should steadily increase, although the amount is difficult to estimate. Finally, the user benefits represent those measured within the predefined study area. Savings in delay and travel time for vehicles previously using the circuitous Isom Road route discussed earlier were not measured. Therefore, the true benefit:cost ratio over the life of the project should be considerably higher than that calculated.

TABLE C-2. ECONOMIC ANALYSIS - TERMINAL DRIVE STUDY AREA

	Delay and Idling Costs[1]				User Benefits			
	Before[2]		After					
	Daily	Annual	Daily	Annual	Annual	Present[3] Value	Construction Cost	B:C Ratio
Airport/I-410	$8,079	$2,019,800	$7,889	$1,972,200	$47,600	$405,200		
Jones-Maltsberger/ U.S. 281	1,391	347,700	1,549	387,200	-39,500	-336,300		
Jones-Maltsberger/ I-410	1,768	442,000	1,061	265,300	176,700	1,504,400		
Terminal Drive/ U.S. 281	0	0	75	18,800	-18,800	-160,100		
Study Area	$11,238	$2,809,500	$10,574	$2,643,500	$166,000	$1,413,200	$2,011,000	0.7:1

[1]Based on estimated delay during 12 highest-volume hours per day, 250 working days per year.
[2]Delay for before cases adjusted to represent traffic volumes after the improvement
[3]20-year functional life, 10% discount rate

Implications for Other Sites

The operational improvements resulting from the construction of a new interchange are largely dependent on the specific project impact area. The street and highway network, land uses, and travel patterns determine the need for and impact of such a project. Therefore, only general items can be summarized from the study of one case.

First, it is important to consider all possible traffic diversions -- intended and unintended -- which may occur as a result of the project. Generally, congested routes or problem areas would not be considered by motorists as shortcuts. However, the project may intentionally divert enough traffic from a problem area to reduce delay and make it an attractive shortcut for another, unanticipated, routing. This secondary diversion may offset the intended benefits of the project.

It is also important to ensure that the street and highway system supports the proposed improvement. For example, guide signing changes may be required to reinforce the desired travel patterns. Improvements in signalization or local access streets may be necessary to ensure full utilization of new freeway access.