CONSIDERATIONS FOR THE INSTALLATION OF U-TURNS AT FREEWAY INTERCHANGES

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TABLE OF CONTENTS

The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Bureau of Public Roads.

INTRODUCTION

One of the major advances in highway design has been the controlled-access highway. By separating through traffic from all cross movement and by controlling the points at which traffic can enter and exit from the through lanes, the freeway designer has provided safety, convenience, comfort, and beauty for users of these facilities. Since the main design effort has been concentrated on the freeway proper, little attempt has been made to adequately design for the traffic which primarily uses parallel frontage roads. Because of the increase in local traffic on these frontage roads, motorists are experiencing more delay, more congestion, and more accidents at points of interchange of a freeway with major arterials. It is neither practical nor possible to eliminate all of the conflicts which occur at these points. However, if certain of the major conflicts could be handled by means which are both practical and economical, it would seem desirable to try to incorporate them into the basic interchange design. This then leads to the obvious conclusion that the analysis and design of freeways must also include simultaneous analysis and design of the cross streets, interchange facilities, and traffic control features.

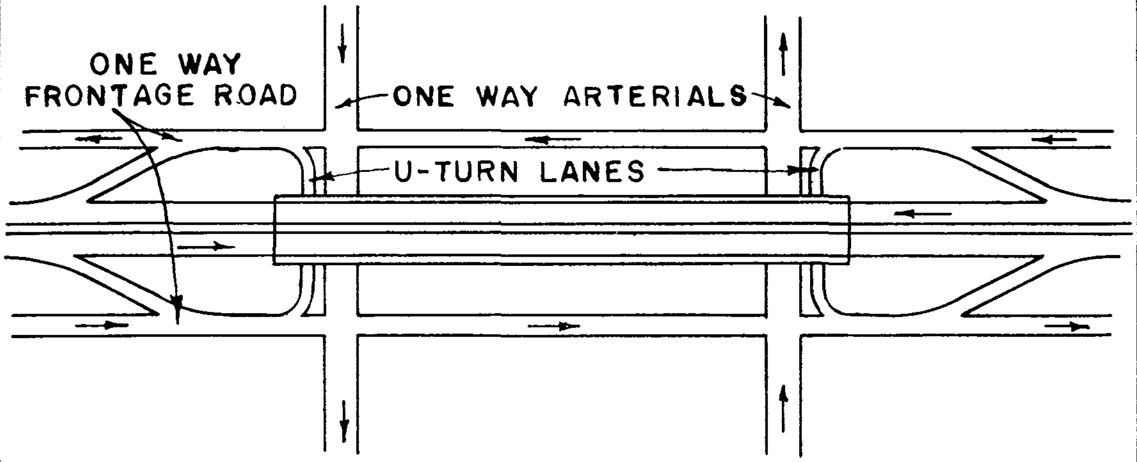
This report will deal mainly with the diamond-type interchange. The use of this design or special adaptation of it in urban areas has become more or less standard due to its minimum right-of-way requirements and its ability to handle large volumes of traffic with proper signal control. Special emphasis has been given to the study of certain movements through these facilities, namely the U-turn maneuver, in order to determine the effect of these movements upon the operating efficiency of the interchange.

Definition of U-Turn

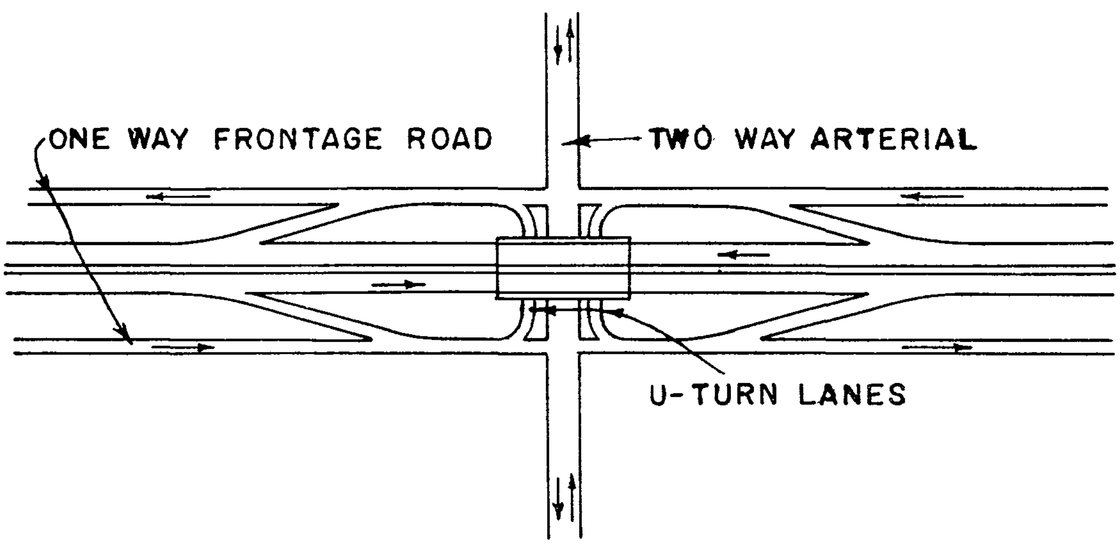
The U-turning maneuver is defined as the movement required to reverse one's direction of travel on a one-way frontage road by use of an intersecting arterial or a special U-turn lane at the interchange. A U-turn lane may be defined as a continuous access lane from one frontage road to the opposite frontage road which eliminates the need to enter either intersection of the frontage roads with the arterial. Illustrations of U-turn lanes are shown in Figure 1.

Study Objectives

It was the objective of this study to investigate the U-turn movement of frontage road traffic in order to determine its effect on the delay produced at signalized intersections and to determine minimum design criteria required to facilitate this movement at freeway interchanges.



SPLIT DIAMOND INTERCHANGE



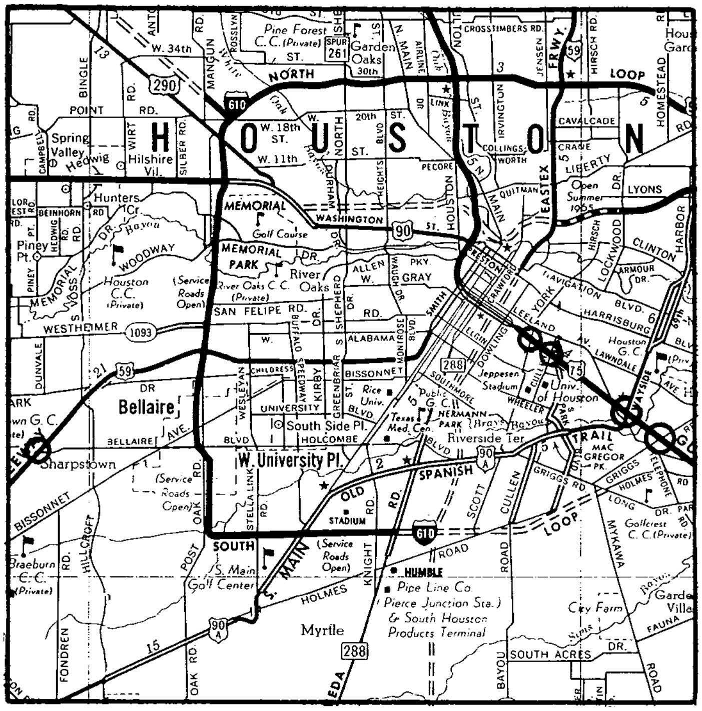
CONVENTIONAL DIAMOND INTERCHANGE

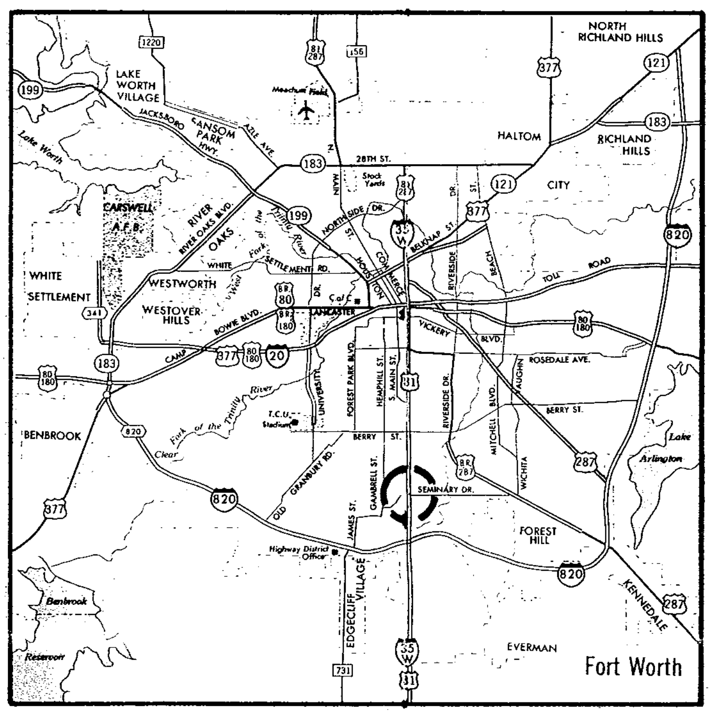
U-TURN FACILITIES

FIGURE 1

Study Sites

The two main study sites used for this study were the Wayside Drive interchange located on Interstate Highway 45 in Houston, Texas, and the Seminary Drive interchange located on Interstate Highway 35 West in Fort Worth, Texas. Several other studies were made at the Scott Street interchange, Cullen Boulevard interchange, and Griggs Road interchange, all located on Interstate Highway 45, and at the intersection of U. S. 59 and Bellaire Boulevard in Houston, Texas. Figure 2 shows location of these study sites within the particular cities mentioned above.





LOCATION OF STUDY SITES

FIGURE 2

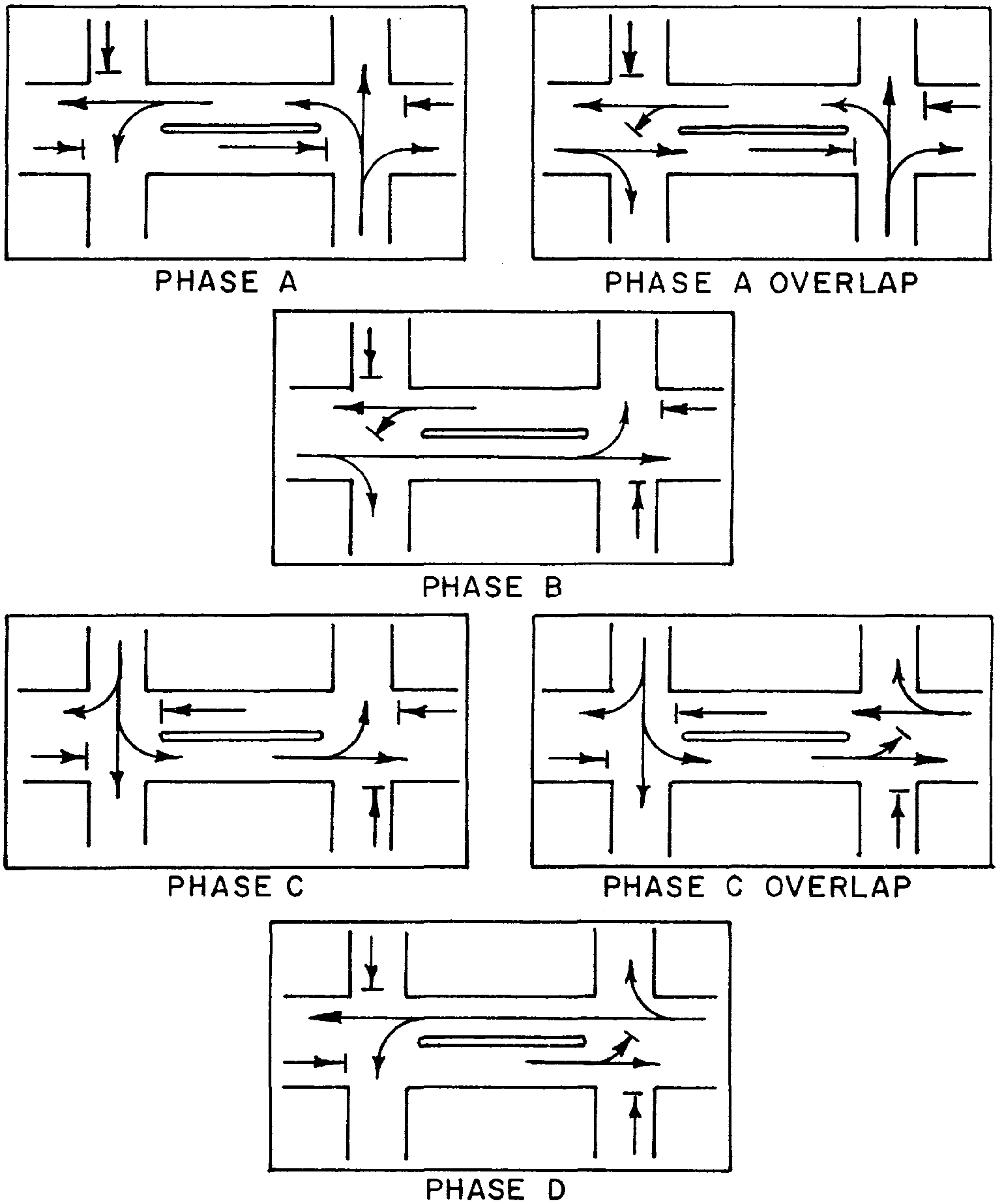
U-TURN EFFECT ON OPERATIONS

The highway designer is constantly striving to put into operation the best set of design criteria in order to make traffic function smoothly under all conditions. Before extensive use of freeways, the designer's job was not too complex because only one set of conditions could exist at a time. In rural areas, the speeds were high and volume relatively low; therefore design was concentrated to fit that characteristic of traffic flow. In urban areas, the reverse was found: high volume and lower speed; and as before, this condition alone was the main design principle. With the advent of the freeways came a mixing of these previously separated types of traffic flow. High volume traffic now moved through the cities at high speeds and few problems arose until this freeway traffic became ready to resume its place on the city street once again. This change could take place only at designated points because of the location of freeway interchanges and the degree of smoothness with which this desired transition took place was simply a function of the design of the interchange itself.

The interchange most often utilized to accomplish this change in traffic form was the diamond interchange. Because of its simplicity in design, this type of interchange has been easily accepted by the motorist. With the addition of continuous frontage roads paralleling the freeways in suburban and urban areas, the interchanges began to mix the short trip frontage road user with the longer trip freeway user who had begun an interchanging pattern. This tended to create congestion in the interchange areas and required that special attention be given to the basic diamond design in order that these problems be resolved. The outcome of this has been the development of the split diamond design and the three level diamond design in the geometric area and a special four-phase overlap signal phasing in the operations area.

Special Four-Phase Overlap Signal Phasing

The use of signals at high volume intersections has become a necessity in order to reduce delay and handle the many different movement desires of the motorists in a safe manner. The widespread use of the diamond interchange on freeways and its characteristic of having two intersections instead of one created a special problem in signal phasing. If the diamond was to continue to be the basic interchange, a method of phasing had to be found that would allow four separated traffic flows to move through an interchange in a minimum amount of time with a minimum of conflict points. Such a finding was made and the basic phase diagram for the diamond interchange can be seen in Figure 3. This phasing allows all traffic from the four approaches to move independently of any other phase and although traffic can be moving at all times, no direct points of conflict are present



RECOMMENDED SIGNAL PHASING

FOR CONVENTIONAL-TYPE DIAMOND INTERCHANGE

FIGURE 3

within the phasing. Utilizing this special phasing in signal operation, the traffic engineer has the ability to move approximately 3400 to 5100 vehicles per hour depending upon the number of lanes provided on the various approaches. These high volumes can be handled at reasonable cycle lengths provided full utilization is made of the special overlap portion of the cycle.

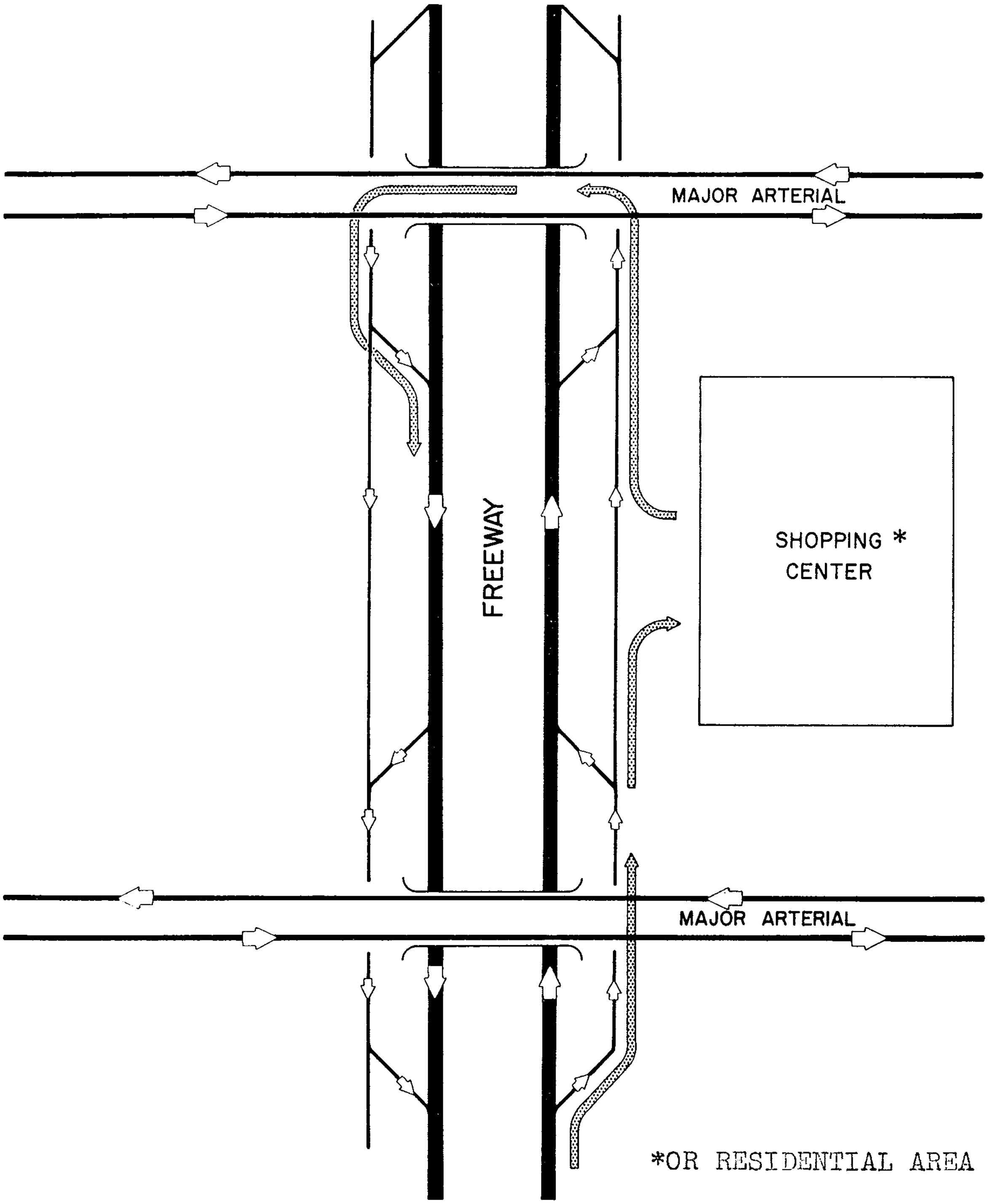
U-Turn Traffic at Diamond Interchange

The signal phasing mentioned above is capable of handling all of the normal traffic movements found at the diamond interchange, but within these normal movements can be found a maneuver which is foreign to the general case. That particular movement is the U-turn--a simple looking movement in the phase diagram, but one that must be investigated quite closely if undue delay is to be eliminated.

The U-turning movement is made up of traffic which is highly influenced by adjoining land use (either abutting the frontage roads or within proximity to them). Figure 4 shows the manner in which traffic attracted to an area, either commercial or residential, can create a demand for U-turn movement at freeway interchanges. Figure 5 illustrates the fluctuation of U-turning volume during the day as a result of land use desires of drivers. Early morning peaks are developed by residential users making their way to the freeway and by workers arriving at work in commercial areas. Noon and evening peaks reflect the homeward trip and peaks during other portions of the day and evening periods show the influence of the commercial generators, such as shopping centers or recreation centers.

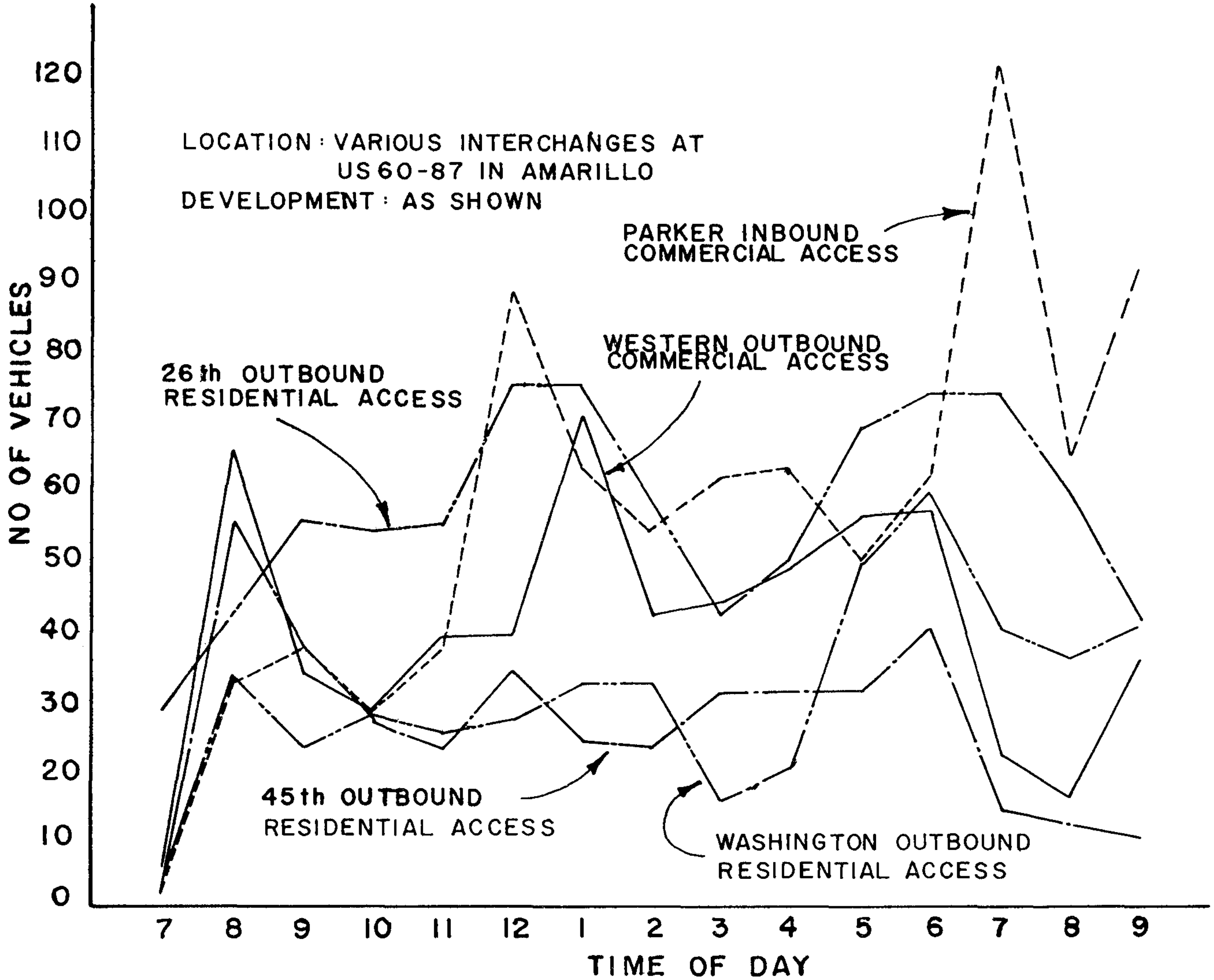
U-turning traffic is also a repetitive type maneuver as similar traces of this volume throughout the week will indicate. Figure 6 shows this characteristic for an area of general commercial land use along both the frontage roads and the arterial. Figure 7 indicates the daily fluctuation of the U-turn volume as affected by a specific commercial land use--the shopping center. The addition of another peaking period during certain days indicates that large shopping centers are capable of generating large volumes at times other than the recognized morning and evening peaks. Figures 5 and 7 also indicate a dual influence of land use as distinct morning peaks are caused by residential traffic and later peaks are caused by the commercial generators in the same area.

As compared to the total approaching frontage road volume, U-turn traffic contributes only a small percentage; but what it lacks in volume, it makes up in delay to the system. This delay cannot be pinpointed to any one particular time period such as the A.M. and P.M. peak conditions. It is entirely a function of the U-turns generated in the interchange area and the efficiency with which this volume is handled, either through design of special U-turning lanes or by adjusted signal operation.



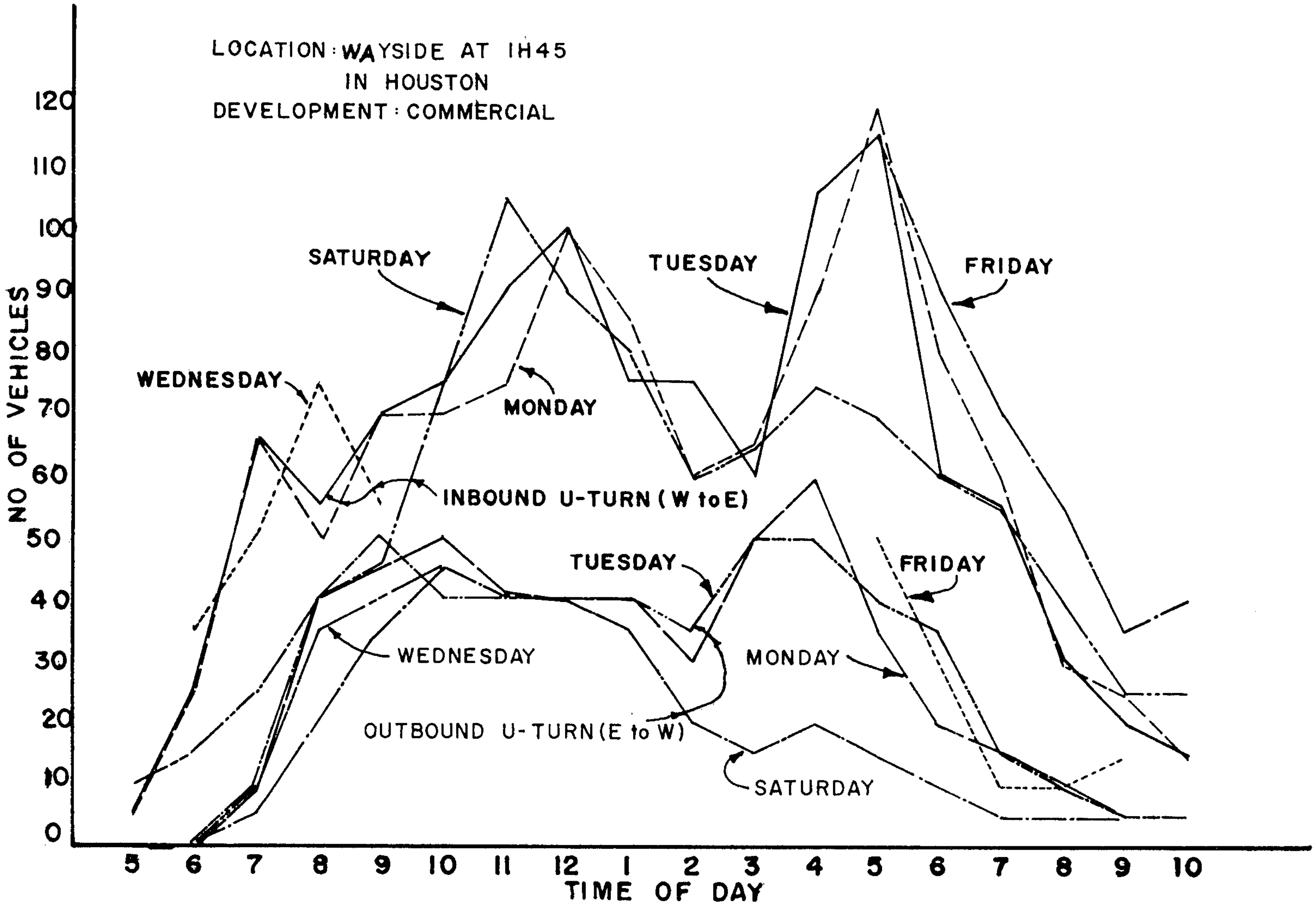
GENERATION OF U-TURN MOVEMENTS

FIGURE 4



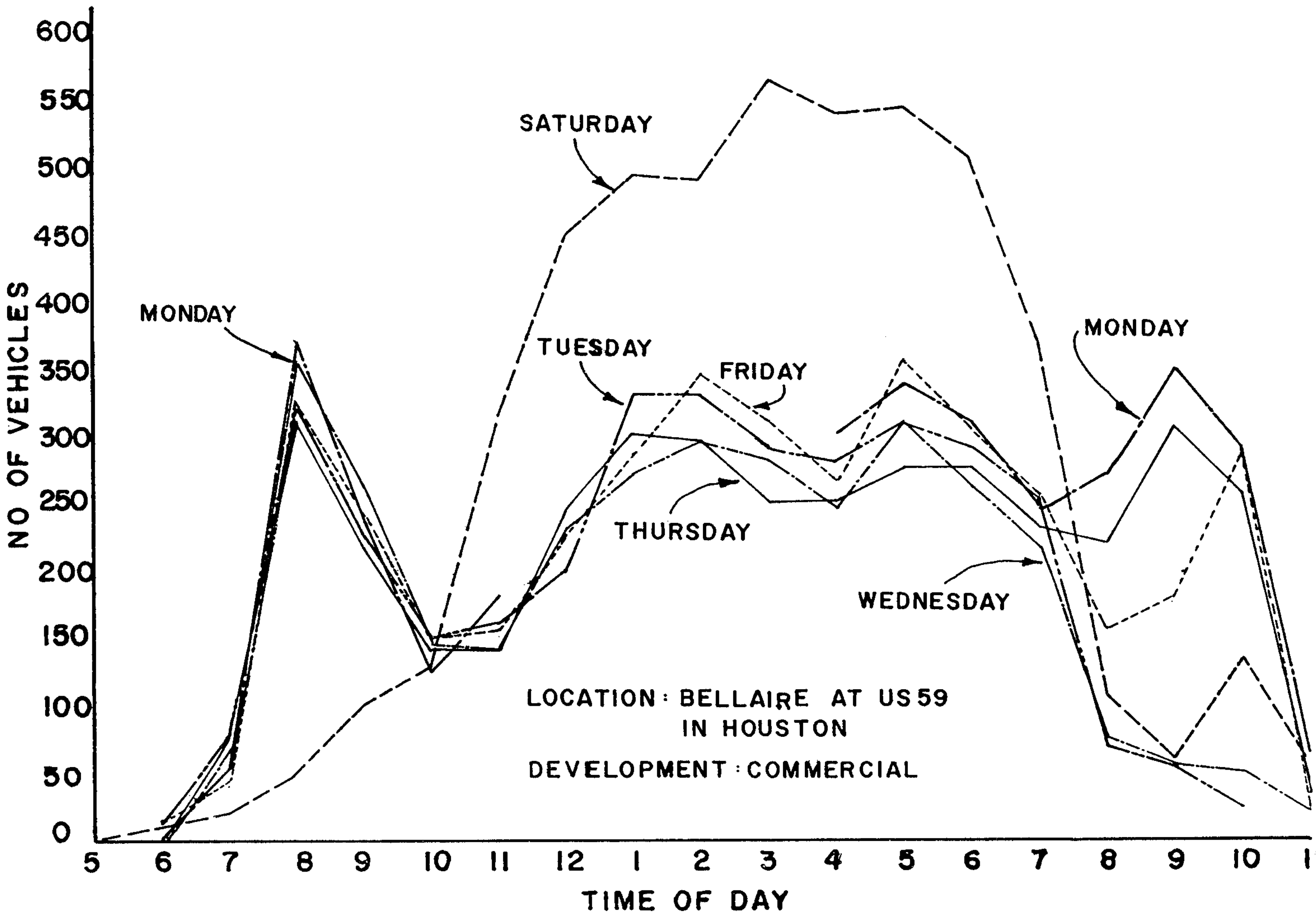
LAND USE EFFECT ON U-TURN VOLUME

FIGURE 5



WEEKDAY VARIATION IN U-TURN VOLUME

FIGURE 6



WEEKDAY VARIATION IN U-TURN VOLUME

FIGURE 7

Problem Areas in Handling U-Turns

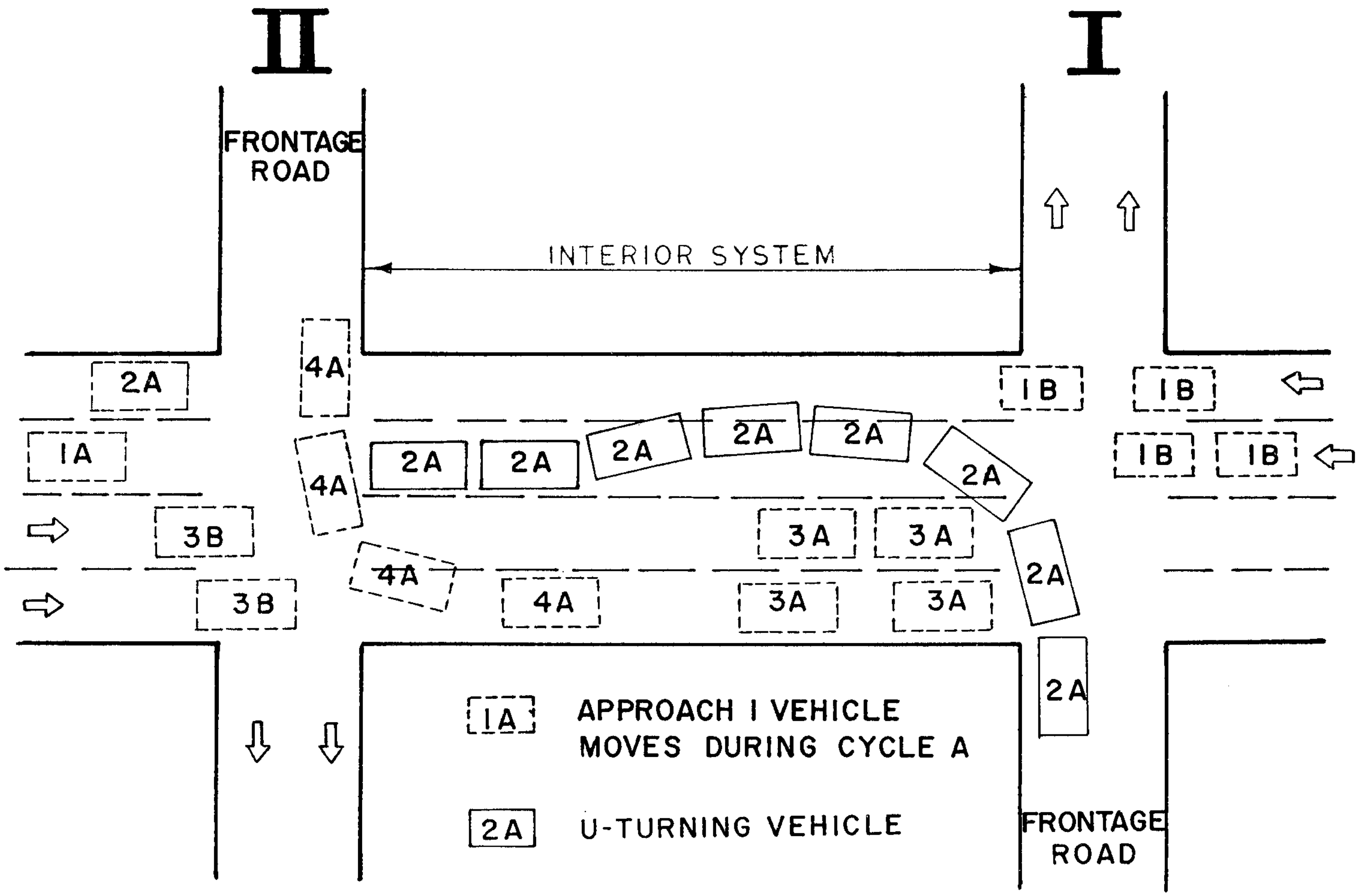
As seen from the signal phasing diagrams, U-turning traffic from each frontage road approach can enter the interior portion between the frontage roads once each cycle. If the distance between the intersections is not too great or the travel time of the particular vehicle in question is not too long, it will be possible for U-turning traffic to make both protected left turns and leave the system. The number of vehicles that can complete this maneuver per cycle depends on the particular length of cycle. During long cycles, a substantial number of U-turning vehicles could be moved through the system. As the cycle lengths are shortened, each percentage phase of that cycle is also decreased. If the full overlap time is maintained regardless of cycle length, the vehicles which move on intervals other than this overlap, such as frontage road traffic, are subject to being held in the system because of insufficient time to clear it. This can lead to an accumulation of vehicles in the interior portion of the system great enough to cause the interchange to become inoperative.

An example of a condition that can develop is shown in Figure 8. The explanation follows:

During phase 1A (Approach 1, Phase A), approach 1 traffic leaves from intersection I and proceeds out of the system. Phase 2A green releases the approach 2 traffic which contains the U-turning vehicles. As mentioned above, some of this U-turning volume may be able to leave the system during its phase but because of the over-lap portion of the cycle, the protected left turn at intersection II is terminated before the green at intersection I. Vehicles 2A are still able to enter the interior system at I until the green is required for the approach 3 traffic now moving on the overlap time. If the 2A traffic turning left wishes to proceed on straight at II it may do so because of its nonconflicting movement with 3A traffic. The 2A traffic wishing to make the U-turn must now stop at II and if sufficient storage is not available to accommodate all U-turning traffic, intersection I can be blocked by it. When the 3A traffic reaches intersection I, it will have a green signal indication but will be unable to proceed because of the waiting 2A U-turn vehicles.

Approach 4A traffic is next released and the left turn portion of it will move into the interior system and complete filling this area with the possiblity of queueing into intersection II. At the end of cycle A, approach 2 U-turning traffic has blocked intersection I; approach 3 straight through and left turning traffic is unable to move through I and is being delayed in the interior system; and approach 4 left turning traffic, having encountered 3A traffic in the interior system, is blocking intersection II.

With the starting of cycle B, approach 1 traffic will move through intersection I, if possible, and will get the green indication at Intersection II. Because of the 4A traffic, 1B traffic cannot move through II nor can the 2A U-turns who have a



INTERCHANGE BREAKDOWN

AS CAUSED BY U-TURNING VEHICLE

FIGURE 8

protective left turn indication at this time. If approach 1B traffic was unable to reach intersection II, then it now may be blocking the frontage road at I. This eliminates any 2B traffic proceeding straight out of the system and the same situation will develop at II with 3B and 4B traffic. Sudden death of an interchange--for within the time of two cycle lengths a high volume facility becomes a parking lot. At interchanges with closely spaced frontage roads, some percentage of U-turning traffic, and no U-turn facilities, the above conditions can become the rule and not the exception during peak periods.

Left Turn Lanes and Storage Lanes

One approach to eliminating this U-turning traffic from the through lanes is the addition of left turn lanes between the frontage roads. However, if the distance between these roads is small, very little storage can be obtained. The presence of the islands restricts the turning movement from the frontage road to the interior system as well as producing an accident hazard to through traffic on the arterial.

Another alternative might be the construction of a storage lane for the U-turning traffic in each direction. This requires the interior system to be widened to at least a 6-lane facility, but does little for the overall efficiency of the entire system. The delay is still present but in its own lane. This stored traffic may still affect left turning vehicles from the frontage roads by requiring them to make a tighter turn into the arterial. Left turning traffic from the arterial is delayed to some extent by the presence of stored vehicles at intersection II, the amount varying with the number waiting. This condition reduces the effect of the overlap portion of the cycle as well as increases the potential for rear end accidents from shock waves produced between these two groups of traffic.

Diamond Interchange Capacity

Since U-turning traffic is a function of land use and therefore difficult to predict, it has been overlooked or considered negligible in many early cases of interchange design. Once a freeway is opened, it seems to have a near volatile effect on the development of an area through which it passes. Conditions beyond the most advanced stages ever considered become a reality almost overnight and with these tremendous land use changes comes the traffic. In many cases of this kind, the designer has been caught with an inadequate design for the condition present and must try to make it function with some degree of efficiency by use of design modifications and/or signalization. Regardless of the efficiency of these changes, there is little that can be done for the situation where traffic demand exceeds the available capacity of an existing interchange. The only sure solution to the problem is to provide adequate interchange capacity in the initial design.

Effect of Lane Use Development

An interchange which has experimenced the situation described above is the Seminary Drive interchange located on Interstate Highway 35 West in Fort Worth, Texas. The original design of Interstate Highway 35 West in this section consisted of a 4-lane depressed freeway with the interchanges at grade. The distance between the parallel frontage roads was approximately 216 feet center to center. The Seminary structure crossing the freeway was 60 feet wide and could carry four moving lanes of traffic. Commercial development was limited to the small generator type as well as the service type. The design of the interchange was adequate for the conditions present at that time and for some time into the future provided no large land use change developed.

The decision to place a major shopping center in the northwest quadrant changed the traffic potentials of the area. In an effort to adjust the existing design to handle this projected increase in volume, the southbound frontage road was relocated some 200 feet to the west, thus extending the distance between the intersections to almost 500 feet. This increased the storage area between the frontage road by approximately 20 vehicles per direction on the arterial. The interchange has been signalized with full actuated volume density equipment, minor movements, skip phase equipment, and pedestrian timers in order to produce an efficient use of signal time through the special four phase overlap phasing.

Even with these two improvements, the basic design capacity of this interchange has not been substantially increased over the initial design. The increased storage may be sufficient to hold the present U-turns and the signalization may be able to move the traffic with efficiency, but as volumes increase, the delays experienced in the interchange are expected to become too great to tolerate. As mentioned before--the only way to keep U-turn traffic from affecting other movements in an interchange is to separate them completely. In the Seminary Drive case this will necessitate the widening of the bridge structure or a separate bridge structure for each U-turn lane. This is an expensive way to eliminate a problem, but as congestion, delay, and accidents increase, the price of correction comes more in line.

Chapter IV in this study deals specifically with the delay at the Seminary Drive interchange as affected by the U-turns generated at the shopping center. The results of the study will show that U-turning vehicles are prime causes of delay at any time they are present in traffic flow.

Conclusion

It would appear that since prediction of future volume is uncertain, the only way to assure adequate capacity is to provide flexibility in the initial design. In the case of the U-turn lanes, if such were not desirable in the first construction phase, all other facilities which are affected by the provision of these lanes should be adequately designed to provide the minimum requirements when they are added at a later date. This means adequate right of way, bridge length, vertical and horizontal clearances, and drainage considerations must be planned for and built as if the ultimate design were to be the first and only considered project. This will necessitate a higher initial construction cost, but the later savings in time and cost will more than offset any original expenditure incurred.

WAYSIDE DRIVE FIELD STUDY

The intersection of Wayside Drive with the frontage roads of Interstate 45 has been the object of intense study in the systems analysis of traffic flow on the Gulf Freeway because of its restrictive capacity. In order that freeway traffic control systems function properly, the capacity of this interchange had to be increased to handle diverted freeway traffic and the methods proposed were outlined in Research Report 24-9, "Capacity-Demand Analysis of the Wayside Interchange on the Gulf Freeway." The methods suggested for increasing the capacity were two-fold, namely: (1) improved signal phasing and timing and (2) the addition of U-turn lanes on both sides of the existing interchange.

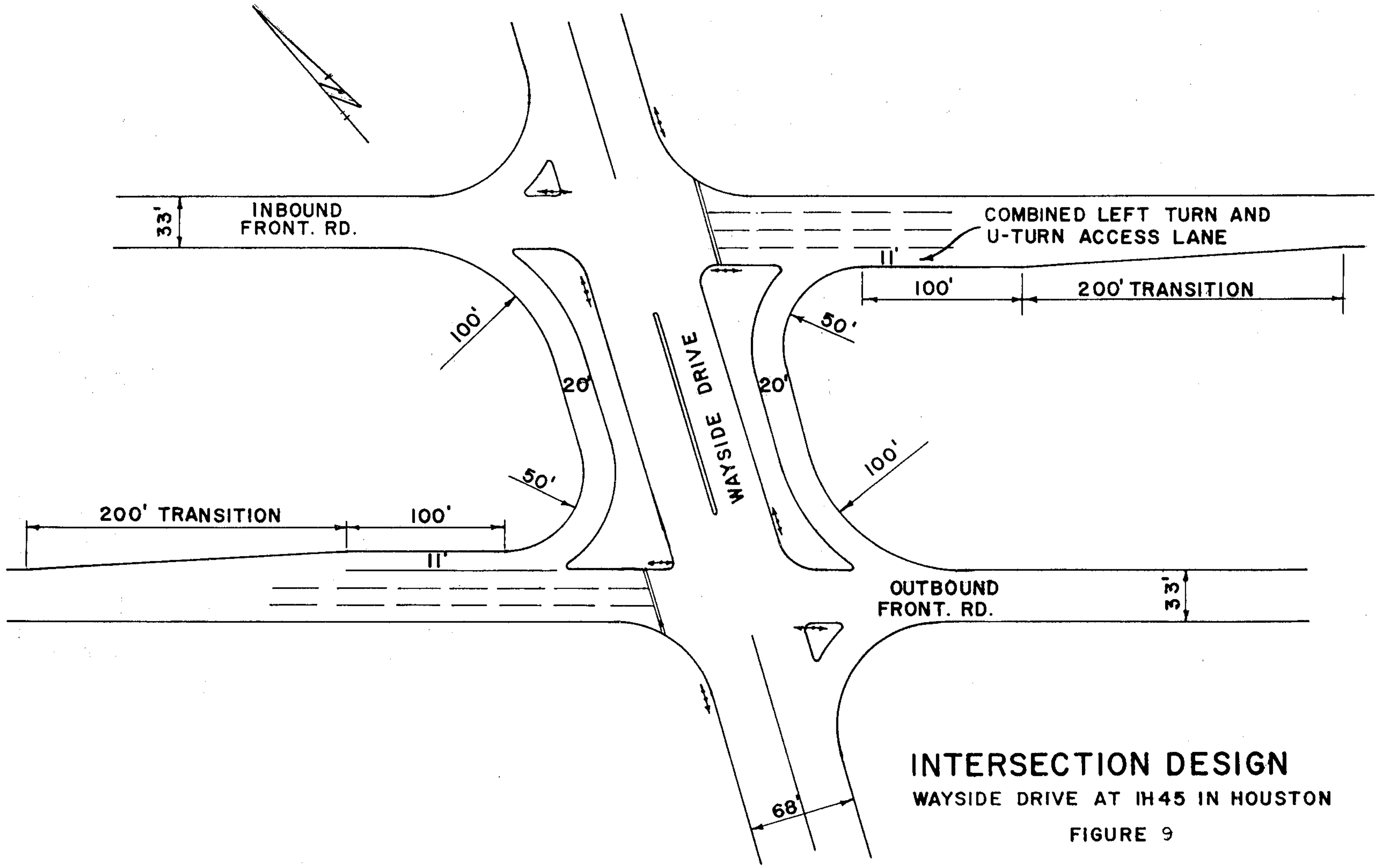
The recommended signal changes were made as proposed and the results were reported in the above mentioned report. The second change was approved by the Texas Highway Department and the U-lanes were constructed by maintenance forces and opened to traffic in October, 1965. A diagram of this completed facility as built can be seen in Figure 9. Original construction of the freeway was such that adequate space was available for placement of the U-turn access lanes and the U-turn lanes without major construction changes to the fill slopes or the bridges. The access lanes and U-turning lanes are identical on both approaches with the exception of the westbound frontage road. Because of the high left turn volume from this approach, the access lane was extended to the intersection in order to provide additional capacity for this movement. The approach now provides two lanes for each maneuver by permitting straight and left or right turns from the inside and center lanes respectively. Signalization of this interchange is controlled by multi-dial fixed time equipment which provides the special diamond signal phasing with the overlap.

Method of Study

In order to determine whether or not the addition of the U-turn lanes was successful in increasing the efficiency of the interchange system an input-output study was made. This type of study was used to determine the total amount of system delay in the interchange in "before" and "after" conditions. Comparative studies were made and a comparison of the results will be discussed in a later portion of this chapter.

Input-Output Study Procedure

The input-output study is a method used to determine amounts of delay to traffic in any situation dealing with arrivals and departures in some type of system during an increment of time. This system may be a straight section of freeway, some service area, or as in the cases cited above, an intersection. The system can be described to include the total intersection, a part of a particular intersection, or a



INTERSECTION DESIGN

WAYSIDE DRIVE AT IH45 IN HOUSTON

FIGURE 9

series of intersections. The important thing to remember is that if results are to be accurate, the system must be operated under a closed condition. This is to say that all traffic within the system must be accounted for throughout the study.

In order to operate a closed system, all possible entry and exit points must be covered in some manner—either by observer, counting device, or barricade. The larger and more complex the system, the greater will be the coverage responsibility.

In the studies conducted at the Wayside interchange, the system was defined as the four avenues of approach to and exit from the interchange: inbound frontage road, outbound frontage road, southbound Wayside Drive, and northbound Wayside Drive. On each approach, the minimum points of input into the system and the points of output from the system were established. This minimum input point was selected as some point which would be in advance of the normally experienced off-peak traffic queues. During peak conditions, this minimum position was adjusted upstream with the increased queue lengths in order to determine when an approaching vehicle joined the queue. The output point remained fixed for each approach but the particular maneuver made by the leaving vehicle was recorded, such as straight, left turn, right turn, U-turn.

Time increments for determining the delay were established to allow the conditions of the system to be found at some particular time. For the Wayside studies, this time increment was a one-minute interval. In studies of other types, a different time increment might be more satisfactory to use; therefore the interval that produces the desired results should be the one used throughout the study.

The actual counts were made manually by observers stationed at the selected points mentioned above and the results were recorded on prepared survey sheets in the one-minute intervals as selected. It was important to have all time pieces synchronized in order that the results be recorded at approximately the same time at all stations. The time keeping responsibility was one of the major problem areas for the individual recorder. He was sometimes not able to record the information at exactly the minute because of having to count a moving stream of traffic or sometimes just forgot. One method used to eliminate this problem was the use of a central time keeper and two-way radios at the count stations. Another method was tried in the Seminary Drive study and will be discussed in another section of the publication.

To get the input-output study started and closed, the number of vehicles in the system at these times must be determined. The method used was to have the input recorder on each approach count the number in the eueue at the starting time and have the output recorder do the same thing at the close of the count.

In summary the basic steps of this type of study are:

1. Define the system

2. Establish the input-output stations within the system

3. Determine the number in the system at the starting time

4. Begin timing

5. Record the volumes at the designated time intervals

6. Stop timing

7. Determine the number in the system at the completion time.

The results of the studies were analyzed by hand and the volumes and amounts of delay were determined for each approach and the entire system. The volume was calculated directly from the counts and the delay was calculated by finding the difference between the input volume and output volume on the same approach during each minute increment. This difference would be the number of vehicles delayed on that particular approach for that one minute of time. Summing these one-minute delays gave the total delay for the study period in vehicle minutes which was then converted into vehicle-hours of delay. For study time periods greater than one hour in length, a maximum hour or peak hour delay was also determined.

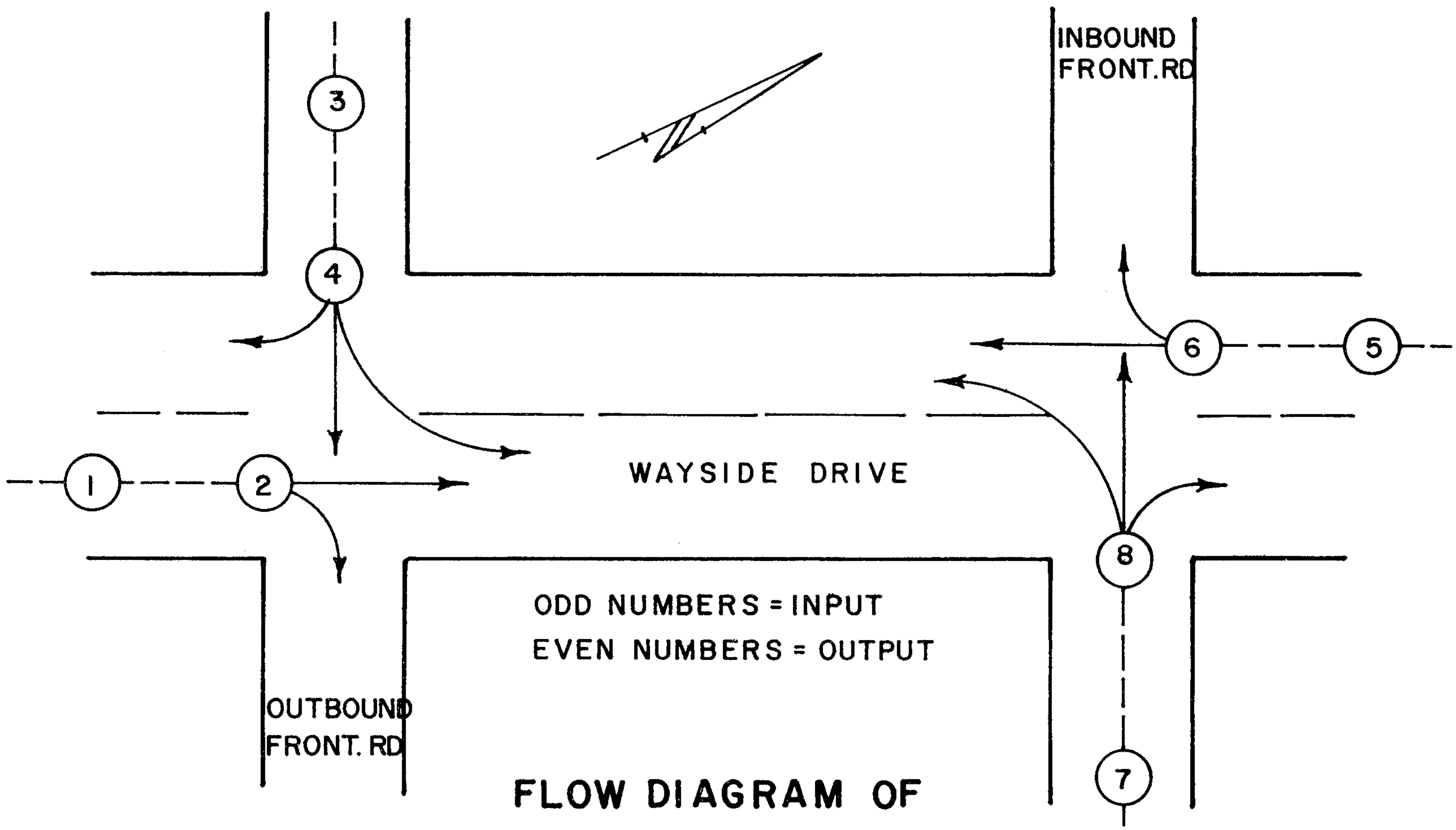
Another method of finding the delay was to plot the one-minute lengths and then find the area under the curve by some acceptable manner. Examples of the graphs are shown in Figure 15, 16 and 17.

Figure 10 shows the flow diagram of the input-output for this interchange. Traffic was counted out of the system at the intersection because of the fact that the special overlap signal phasing assures the continuous movement from the interchange once the approach intersection has been cleared.

Discussion of Results

Before and After Study

As mentioned above the construction of the U-turn lanes was part of a two-fold recommendation for improving the capacity of the Wayside Drive interchange. After completion of these lanes, another input-output study was made in order that some comparison be made with the "Before" results and the "After Signal Timing" results. The results of all three studies plus an additional signal efficiency study results are shown in Table 1. The results show that the improvement of signal efficiency was the most noticeable accomplishment. The May, 1965 study shows an increase in volume with only a small increase in delay. The November study was made after the U-turn lanes were opened and showed an increase in delay of approximately



FLOW DIAGRAM OF
WAYSIDE INPUT-OUTPUT STUDY

FIGURE 10

TABLE 1

SUMMARY OF RESULTS
MORNING STUDY
A. M. PEAK PERIOD

Intersection Approach	Before Signal Timing*	After Signal Timing*	May 1965	Nov. 1965
Wayside Southbound:				
Total Number of Vehicles			2017	
Peak Hour Volume	1057	1127	1030	
Total Vehicle Delay			33.4	72.8
Peak Hour Delay	54.6	24.8	16.6	55.5
Wayside Northbound:				
Total Number of Vehicles			2163	
Peak Hour Volume	1187	1180	1185	
Total Vehicle Delay			10.2	14.0
Peak Hour Delay	5.9	9.0	6.3	9.5
Frontage Road Eastbound:				
Total Number of Vehicles			931	759
Peak Hour Volume	498	471	526	465
Total Vehicular Delay			15.4	9.4
Peak Hour Delay	8.2	6.7	9.2	5.8
Frontage Road Westbound:				
Total Number of Vehicles			1703	1834
Peak Hour Volume	974	868	995	1169
Total Vehicular Delay			29.3	17.8
Peak Hour Delay	25.7	10.7	22.3	11.9
Totals:				
Peak Hour Volume	3716	3646	3736	
Peak Hour Delay	94.4	51.2	54.4	79.7

*Average values for studies reported in Report 24-9.

50 per cent over the signal timing studies. This value seems to be quite high and it is not known whether this was some function of signal inefficiency, accident, or misrecorded data on the southbound Wayside approach. The influence of the additional capacity on the inbound frontage road can be seen as that particular volume had increased 18 per cent over the May study and the delay was reduced by 50 per cent. No volumes were taken during the November study on the Wayside approaches; therefore no comparison was made with prior studies.

Table 2 gives the results of three studies made during the P. M. peaking condition after the U-turn lanes were opened. As will be noticed, the volumes are all increased over the morning peak volume but the delay is substantially smaller.

The U-turning volumes and left turn volumes taken before and after the opening of the U-turn lanes are shown in Tables 3, 4, 5, and 6. Both morning and evening peaks are included in these counts. The results bear out the fact that U-turning volumes are very consistent and show little change from volumes taken during earlier studies.

Total approach volumes before and after the design change took place are shown in Tables 7 and 8. All volumes show some increase with the exception of the outbound frontage road.

Conclusions

With the opening of the U-turn lanes at the Wayside interchange last year, the recommendations set forth in Report 24-9 were fulfilled. The results obtained by studies before and after these changes were made indicate that substantial improvement has been made in the operating efficiency of the interchange. The signalization change shows to have been the source of greatest improvement to the entire system with a reduction in delay by almost 55 per cent. The addition of the U-turn lanes has improved operation efficiency of the frontage road approaches by approximately 50 per cent.

The results show that fixed time equipment, when properly timed, is a most efficient means of handling traffic at a diamond interchange. In the afternoon study a total volume of almost 4000 vehicles were moved with a delay of only 37 vehicle-hours. When compared to the other studies (Seminary and Bellaire) made, the above figures indicate that twice the volume was moved at the Wayside interchange with only a 10 per cent increase in delay. It was this same equipment improperly timed which was the main source of delay in the initial studies at this interchange.

TABLE 2

SUMMARY OF RESULTS
AFTERNOON STUDY
P. M. PEAK PERIOD

Intersection Approach	Study No. 1	Study No. 2	Study No. 3	Average
Wayside Southbound:				
Total Number of Vehicles (Veh)	2511	2661	2450	2541
Peak Hour Volume (Veh/Hr)	1386	1380	1345	1370
Total Vehicular Delay (Veh-Hrs)	18.4	30.9	17.9	22.4
Peak Hour Delay	10.5	17.7	10.4	12.9
Wayside Northbound:				
Total Number of Vehicles (Veh)	2140	2172	2055	2122
Peak Hour Volume (Veh/Hr)	1165	1186	1102	1151
Total Vehicular Delay (Veh-Hrs)	22.3	28.7	12.6	21.2
Peak Hour Delay	12.2	16.8	6.6	11.9
Frontage Road Eastbound:				
Total Number of Vehicles (Veh)	1552	1691	1765	1669
Peak Hour Volume (Veh/Hr)	830	883	978	897
Total Vehicular Delay (Veh-Hrs)	13.7	10.4	18.1	14.1
Peak Hour Delay	7.7	6.1	9.5	7.8
Frontage Road Westbound:				
Total Number of Vehicles (Veh)	1135	1115	996	1082
Peak Hour Volume (Veh/Hr)	611	589	536	579
Total Vehicular Delay (Veh-Hrs)	8.7	10.7	8.0	9.1
Peak Hour Delay	4.8	6.3	4.3	5.1
Totals:				
Peak Hour Volume (Veh/Hr)	3892	4038	3961	3997
Peak Hour Delay (Veh-Hrs)	35.2	46.9	30.8	37.7

TABLE 3

U-TURN STUDY—WAYSIDE INTERCHANGE
JUNE 17, 1965

Time	U-Turn from Inbound Frontage Road to Outbound Frontage Road	U-Turn from Outbound Frontage Road to Inbound Frontage Road
4:00-4:30 P.M.	40	27
4:30-5:00	52	28
5:00-5:30	74	22
5:30-6:00	31	18
6:00-6:30	26	17
Total 4:00-6:00 P.M.	197	95
Total Peak Hour	126	50

TABLE 4

U-TURN STUDY—WAYSIDE INTERCHANGE
JUNE 18, 1965

Time	Left Turn from Inbound Frontage Road to Wayside	U-Turn from Inbound Frontage Road to Outbound Frontage Road
6:30-7:00 A.M.	161	16
7:00-7:30	214	15
7:30-8:00	182	24
8:00-8:30	174	23
Total 6:30-8:30 A.M.	731	78
Total Peak Hour	396	39

TABLE 5

U-TURN STUDY—WAYSIDE INTERCHANGE
NOVEMBER 8, 1965
NOVEMBER 22, 1965
APRIL 24, 1966

Time	U-Turn from Inbound Frontage Road to Outbound Frontage Road			U-Turn from Outbound Frontage Road to Inbound Frontage Road	
	Nov. 8	Nov. 22	Apr. 24	Nov. 8	Nov. 22
3:00-3:30 P.M.	37	28		22	30
3:30-4:00	36	27		15	27
4:00-4:30	43	42	50	19	15
4:30-5:00	49	55	71	22	14
5:00-5:30	72	63	53	14	13
5:30-6:00	40	51	31	15	11
Total 4:00-6:00 P.M.	204	211	205	70	53
Total Peak Hour	121	126	124	41	57

TABLE 6

U-TURN STUDY—WAYSIDE INTERCHANGE
NOVEMBER 8, 1965

Time	U-Turn from Inbound Frontage Road to Outbound Frontage Road	U-Turn from Outbound Frontage Road to Inbound Frontage Road
6:30-7:00 A.M.	22	2
7:00-7:30	20	4
7:30-8:00	28	8
8:00-8:30	23	14
Total 6:30-8:30 A.M.	93	28
Total Peak Hour	51	22

TABLE 7

A. M. PEAK HOUR VOLUMES
WAYSIDE DRIVE AND GULF FREEWAY FRONTAGE ROADS

Approach	Before Design Changes	After Design Changes
Inbound Frontage Road	995	1169
Outbound Frontage Road	526	465
Southbound Wayside	1030	
Northbound Wayside	1185	
Totals	3736	

TABLE 8

P. M. PEAK HOUR VOLUMES
WAYSIDE DRIVE AND GULF FREEWAY FRONTAGE ROADS

Approach	Before Design Changes Apr. 1965	After Design Changes Nov. 1965
Inbound Frontage Road	795	842
Outbound Frontage Road	650	598
Southbound Wayside	1244	1369
Northbound Wayside	1036	1142
Totals	3725	3951

SEMINARY DRIVE FIELD STUDY

The Seminary Drive interchange is located on Interstate Highway 35 West in southwest Fort Worth, Texas. The freeway in this area is a depressed type facility with all cross streets at grades. For this reason the Seminary Drive interchange makes use of a bridge structure for a segment of the arterial roadway and does not have U-turning lanes on either frontage road approach. A geometric layout of the interchange is shown in Figure 11. The signal equipment used is the Automatic Signal volume density controller with minor movements and special timers to produce the diamond type phasing.

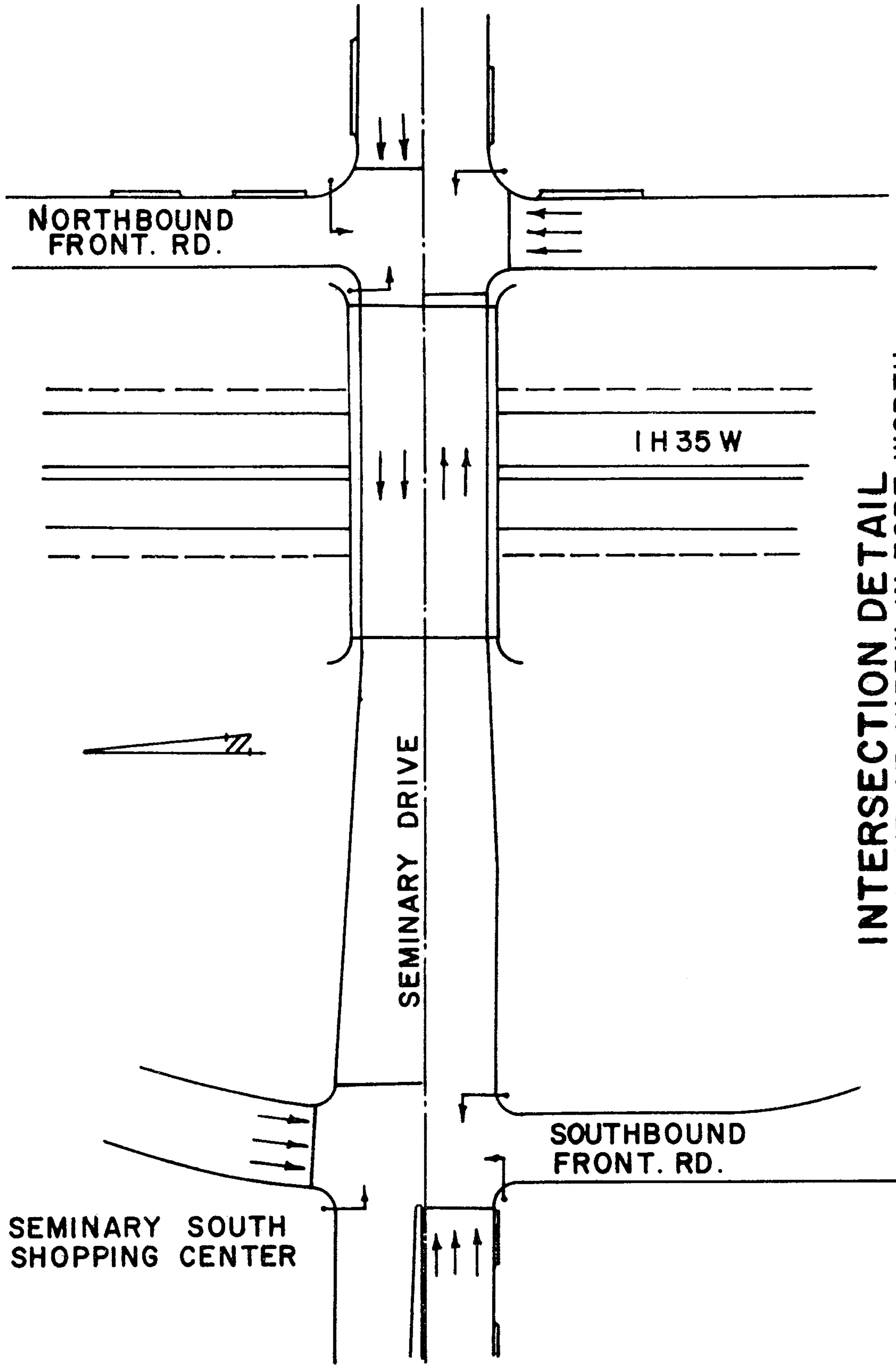
The area along the frontage roads and Seminary Drive in the vicinity of the interchange is of commercial development with a major shopping center in the northwest quadrant. Traffic enters and leaves the parking areas from the southbound frontage road and Seminary Drive. One of the primary movements is the U-turning movement of persons in the shopping center desiring to return to the north on the freeway.

A characteristic of the modern day shopping center is its scheduling of open hours to fit the convenience of the shoppers, namely the evening shopping periods on certain weekday nights. The study at Seminary Drive was made to coincide with one of these particular shopping days in order to study the effects caused by the generation of the U-turns.

Method of Study

Because of the Layout of the Interchange, a revised method of the previously used input-output study was tried. Since the main interest was in the U-turn traffic from the shopping center, only two approaches of the interchange were used, the southbound frontage road approach and the eastbound Seminary Drive approach. The input and output portion of this study on this approach was the same as used previously. From this point the system was extended to include the eastbound traffic at the northbound frontage road. A sketch of the system can be seen in Figure 12. The left turning and U-turning vehicles (2a) from the southbound frontage road and the straight through vehicles (4a) from eastbound Seminary Drive became the input volume for this interior section of the system. The output was then taken as the left turn (6a) to the northbound frontage road and the straight (6b) through on Seminary Drive. The left turn was broken down into the U-turn from the southbound frontage road and the left turn from Seminary Drive from the west side of the interchange.

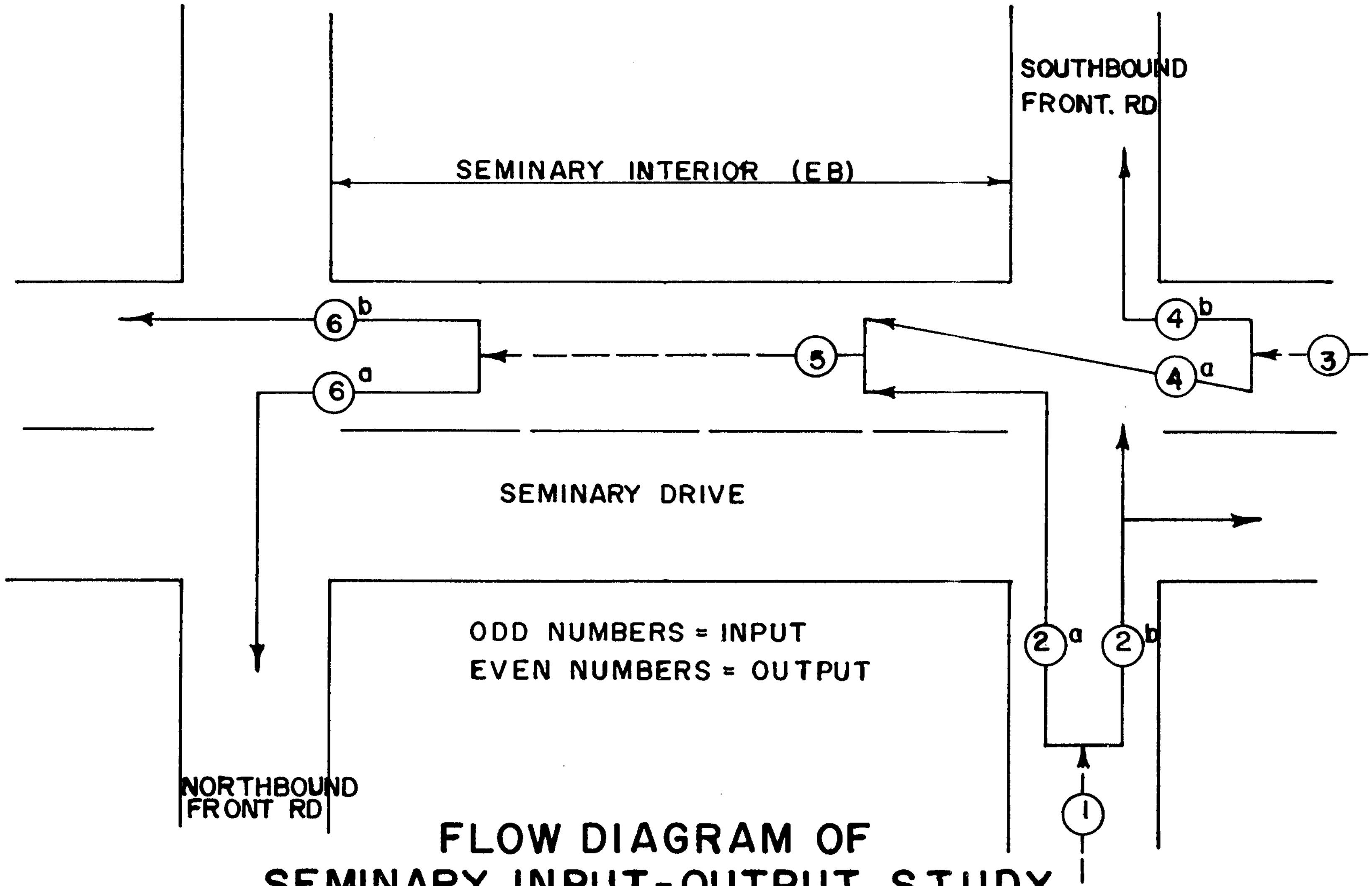
Total delay in the system was now comprised of three elements for the half of the interchange studied. These are the delay on the frontage road approach, the delay on the arterial approach and the delay on the interior portion at the north frontage road. This delay was determined for different times during the afternoon and evening to see if any variation could be found. A total of four different studies were made between the hours of 2:30 P. M. to 9:30 P. M. and the length of study varied from one hour to one and one half hours depending on the traffic condition during the test period.



INTERSECTION DETAIL

SEMINARY DRIVE AT IH35W IN FORT WORTH

FIGURE 11



FLOW DIAGRAM OF SEMINARY INPUT-OUTPUT STUDY

FIGURE 12

Revised Input-Output Procedure

In order to eliminate the time keeping process throughout the study, a 20-pen Esterline Angus recorder was used with a constant speed chart drive. Switches were used at each point of input and output and the results were automatically recorded on the chart as it was turned on and a finishing time was turned on and a finishing time was recorded at the completion of the study. At the beginning of the time period, the person counting the input volumes on each approach recorded the number in the system and upon closing the study the person counting the output volumes on each approach counted the number in the system. This assured that the system was properly charged and cleared at the beginning and end of a study period respectively. The analysis of the data was handled in the same manner as the Wayside Drive study.

Discussion of Results

The results of the four studies are shown in Table 9. The studies represent four different traffic conditions: midafternoon, peak afternoon period, mid-evening, and closing peak condition as affected by a large generator. Due to the absence of U-turn lanes and the inability of the signals to adequately handle U-turning traffic, the effect of this one maneuver can be seen in the increased delay at all three points checked. U-turning traffic seemed aware that they could not easily make the signal at the northbound frontage road, so their departure from the southbound frontage road was quite slow in most cases. During the peak hour conditions, the U-turn queue waiting on the bridge would contain as many as eight vehicles. Eastbound Seminary Drive traffic wishing to turn left at the north frontage road would be released by the overlap portion of the signal cycle and have to stop again due to this waiting queue. Several times during this study, eastbound traffic blocked the intersection of the southbound frontage road. This congestion can easily be seen from the results of the 8:30-9:30 P.M. study made. The presence of these vehicles at the interior intersection also caused a restrictive type left turn from the frontage road approaches. During this time, the U-turn volume comprised nearly 30 per cent of the frontage road approach volume and although the closing peak volume was 35 per cent less than the afternoon peak, the delay was within 13 per cent of the peak hour delay. The main portion of the delay was concentrated on the interior section of the study area.

Conclusion

A main source of delay was found to occur in the interior system. This was mainly because of the U-turning traffic not being able to clear the second interchange in the time portion allowed for its movement. Because of the use of actuated equipment, this condition took place on almost every cycle; the exception being when long queues formed on the frontage roads and extended the

TABLE 9

SUMMARY OF RESULTS
SEMINARY DRIVE

Intersection Approach	2:45-3:45 P.M.	4:30-6:00 P.M.	6:45-8:05 P.M.	8:30-9:30 P.M.
Seminary Eastbound:				
Total Number of Vehicles (Veh)	743	1356	1030	762
Peak Hour Volume (Veh/Hr)		927	672	
Total Vehicular Delay (Veh-Hrs)	14.1	20.0	12.8	9.2
Peak Hour Delay		12.6	9.5	
Frontage Road Southbound:				
Total Number of Vehicles (Veh)	686	1414	817	666
Peak Hour Volume (Veh/Hr)		1005	567	
Total Vehicular Delay (Veh-Hrs)	5.3	13.9	10.2	6.5
Peak Hour Delay		10.0	6.0	
Seminary Interior Eastbound:				
Total Number of Vehicles (Veh)	938	1728	1212	930
Peak Hour Volume (Veh/Hr)		1129	826	
Total Vehicular Delay (Veh-Hrs)	3.9	16.9	10.5	12.6
Peak Hour Delay		9.3	7.1	
U-Turn Volumes (Max. Hour):	162	171	117	181
Per Cent of Frontage Road Volume	24	12	14	27
Totals:				
Max. Hour Volume (Veh/Hr)	1429	1932	1239	1428
Max. Hour Delay (Veh-Hrs)	23.3	31.9	22.9	28.3

time beyond that required to allow U-turning traffic to clear the system. Over-all signal efficiency seemed to be good and traffic was handled very smoothly in all phases.

Although the major portion of concern has been concentrated on the afternoon peak period in most cases, it can be easily shown that comparable amounts of delay can be encountered at lower volumes due to the fact that certain movements contribute such high delay to a system.

BELLAIRE BOULEVARD FIELD STUDY

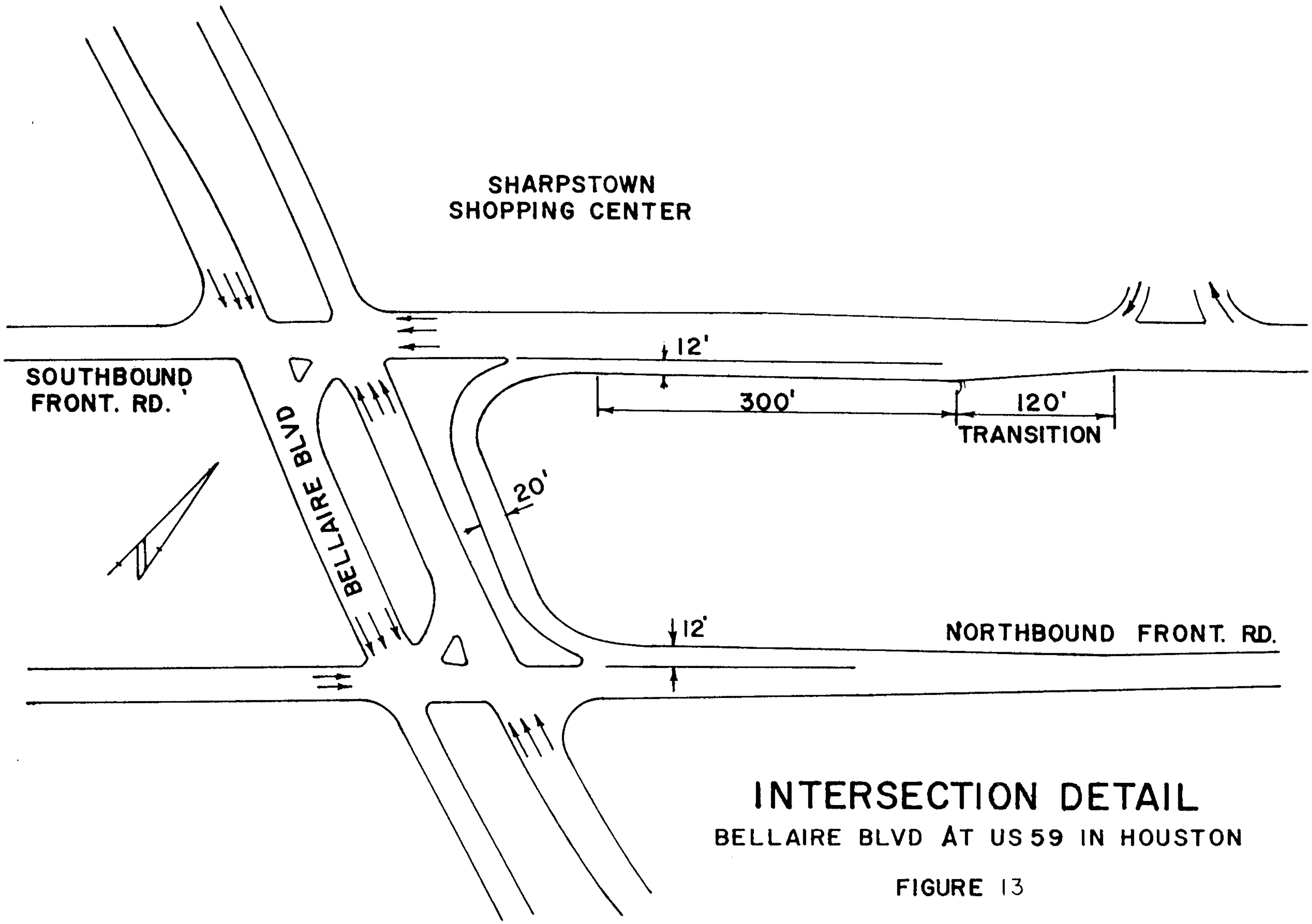
This study was conducted at the intersection of Bellaire Boulevard and the U.S. 59 frontage roads in the Sharpstown area of Houston, Texas. This site was chosen because of its similarity to the Seminary Drive interchange located in Fort Worth, Texas. It is in a commercial land use area and contains a major shopping center in the same quadrant as the Fort Worth interchange. The primary difference between the two locations is the provision of the U-turn lane on the southbound frontage road approach at the Bellaire interchange. A geometric layout of this interchange is shown in Figure 13. Signalization control is of the Crouse-Hinds Diamond Vehicle Actuated type which is capable of providing the special diamond phasing under certain conditions.

Method of Study

The same procedure used in the Seminary Drive study was also employed in this study in order that a more accurate comparison of results could be made. The study was made throughout one afternoon and into the evening until the shopping center had closed and the majority of shoppers and employees had left the parking lot. The only major change in procedure was the manner in which the U-turning vehicles were handled in the input-output portion of the frontage road approach system. A diagram of the input-output arrangements for the portion of the interchange studied is shown in Figure 14. It will be noted that the U-turn was still considered in the system until it left the U-turn lane. Although this was a free flowing movement most of the time, some small amounts of delay were experienced due to traffic moving on the inbound frontage road.

Discussion of Results

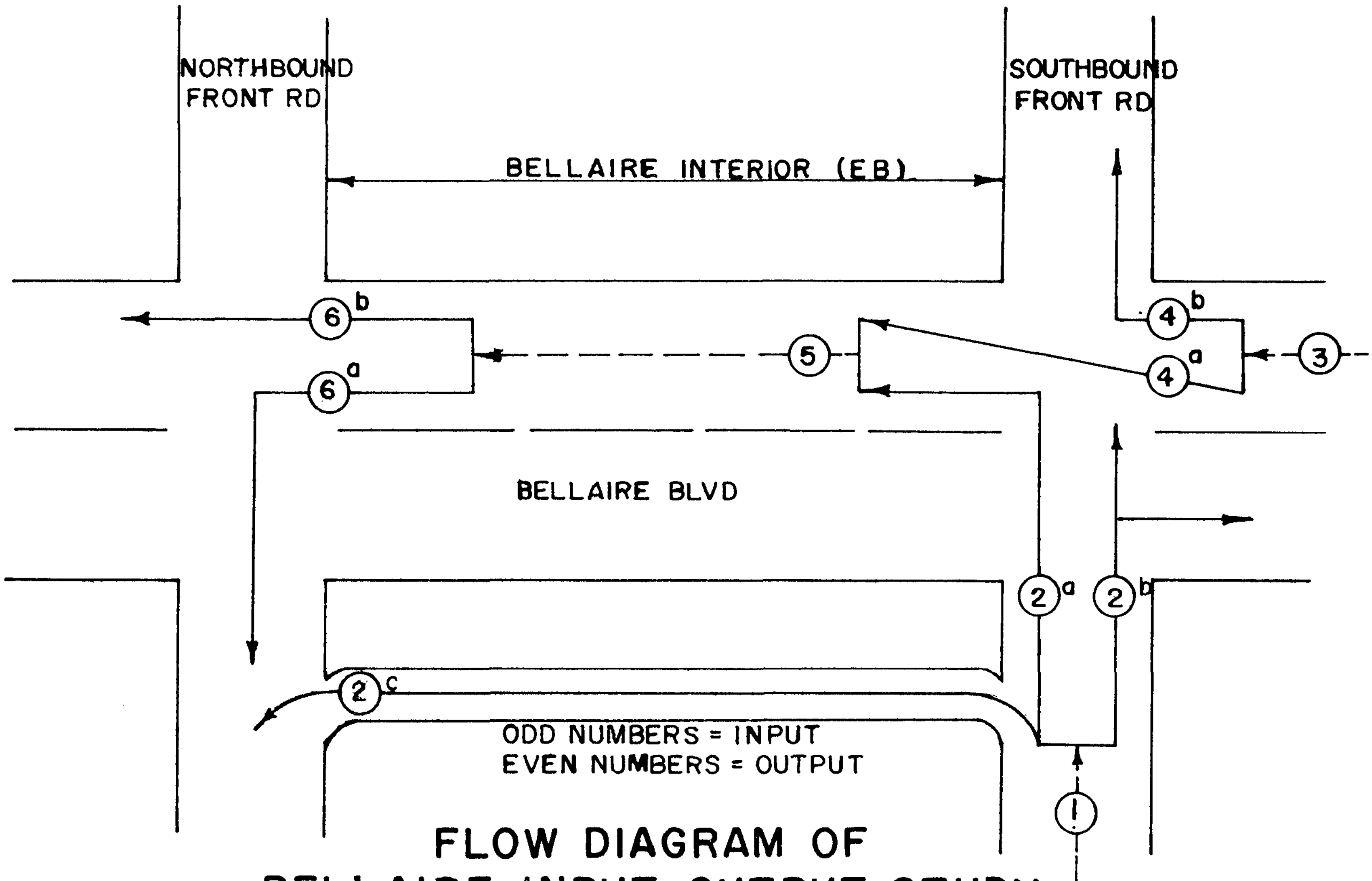
The complete summary of results can be found in Table 10. At this location U-turning traffic accounts for a sizeable amount of the frontage road traffic, but because of the fact that this traffic can be completely separated from the remainder of the frontage road traffic, it contributes very little, if any, delay to the system. The major portion of the delay in the system seemed to be caused by a signal deficiency which produced high delay on the interior approach. Once again it can be shown that substantial amounts of delay are encountered at times not usually thought of as peak hours. The results show that the last hour of the study was the highest volume level during the time the studies were being made.



INTERSECTION DETAIL

BELLAIRE BLVD AT US 59 IN HOUSTON

FIGURE 13



FLOW DIAGRAM OF
BELLAIRE INPUT-OUTPUT STUDY
FIGURE 14

TABLE 10

SUMMARY OF RESULTS
BELLAIRE BOULEVARD

Intersection Approach	3:00-4:00 P.M.	4:40-6:02 P.M.	6:36-7:39 P.M.	8:06-9:27 P.M.
Bellaire Blvd. Eastbound:				
Total Number of Vehicles (Veh)	746	1072	741	1139
Peak Hour Volume (Veh/Hr)		720		881
Total Vehicular Delay (Veh-Hrs)	5.1	16.6	4.6	10.1
Peak Hour Delay		8.3		8.3
Frontage Road Southbound:				
Total Number of Vehicles (Veh)	927	1527	765	1443
Peak Hour Volume (Veh/Hr)		1009		1082
Total Vehicular Delay (Veh-Hrs)	5.6	19.1	3.2	21.7
Peak Hour Delay		13.0		17.1
Bellaire Blvd. Interior Eastbound:				
Total Number of Vehicles (Veh)	899	1409	897	1289
Peak Hour Volume (Veh/Hr)		937		990
Total Vehicular Delay (Veh-Hrs)	8.9	22.0	13.3	5.5
Peak Hour Delay		14.0		3.7
U-Turn Volume (Max. Hour):	418	361	300	514
Per cent of Frontage Road Volume	45	36	39	48
Totals:				
Max. Hour Volume (Veh/Hr)	1673	1729	1506	1963
Max. Hour Delay (Veh-Hrs)	19.6	35.3	21.1	29.1

Comparison of Results

Before a comparison as such is made between the results of the Seminary study and the Bellaire study, it would be best to point out the major interchange differences so as not to accept per se the study results as they appear in the tables. The distance between frontage roads at Bellaire is approximately 280 feet while at the Seminary frontage roads this distance is almost 500 feet. This would tend to increase the delay in the Seminary system simply because of the interchange itself being larger.

Signal control equipment is another major difference. The Bellaire system is a full actuated, 4-phase type with the ability to supply or delete the special overlap portion of the signal cycle. The Seminary system has the full actuated, volume density type equipment which supplies the overlap each cycle

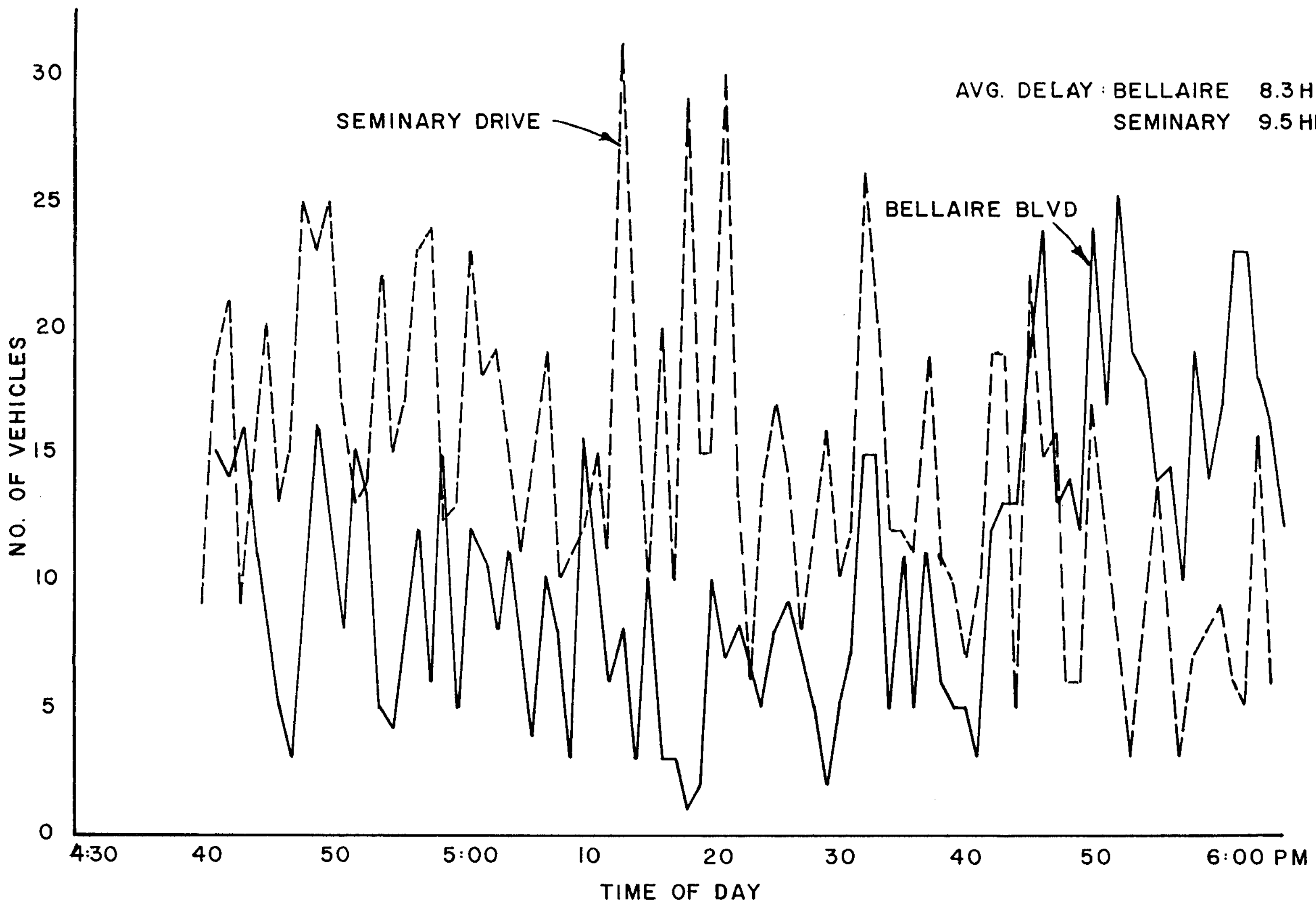
The U-turn access lane and U-turn lane were especially designed to handle the U-turning traffic from the shopping center parking lot and this traffic should actually have little effect on the frontage road traffic. However, the delays are very close to the Seminary values. It is not felt that the differences in volumes between the two study sites could be the cause of this increased delay at Bellaire. Observations in the field showed an inefficient handling of traffic through the two intersections of the interchange. The results tend to bear this out as three of the four studies had the high delay on the interior approach in the system. The other study had the high delay value on the frontage road approach partly because of the higher volume present on it, but also due to the manner in which westbound Bellaire traffic was handled at the interior approach signal. This particular movement was not part of the system being studied, but its presence in the interchange had a great effect on the signal time split at the southbound frontage road. It was released from the westbound exterior approach and would proceed to the westbound interior approach where it had to come to a complete stop for approximately 10 seconds and during this time no vehicles were allowed to move at this intersection. This stop condition to Bellaire traffic was caused by the absence of a vehicle detector call on that approach until the traffic arrived at the intersection. If some traffic were left in the interior system and placed a call on the approach for the next cycle, then the coordinating unit of the equipment would supply the required overlap feature and the westbound traffic could move through without being stopped. This feature, however, was supplied at the expense of delayed traffic because the overlap feature was always supplied on the upcoming cycle after the call was placed. In a sense this feature presented an alternating effect, being available only every other cycle. The eastbound interior approach delay was reduced because the northbound frontage road volume was very light during the last study and this allowed the overlap green at that intersection to remain on throughout most of the study.

The interior delay at Bellaire must be considered as unnecessary and undesirable. As compared to the Seminary interior delays, where the actual U-turn traffic is stored on this interior portion, it was found that the Seminary system more efficiently handled a higher volume of traffic with less delay than did the Bellaire system. It was also felt that the main reason for this was the inability of the signal equipment to provide the needed overlap portion each cycle.

Graphs showing the queue lengths per approach during the peak hour at Seminary and Bellaire can be seen in Figure 15, 16, and 17. The average delays for the approaches are shown for both interchanges on each graph. These delays represent the areas under the respective curves for the particular approach.

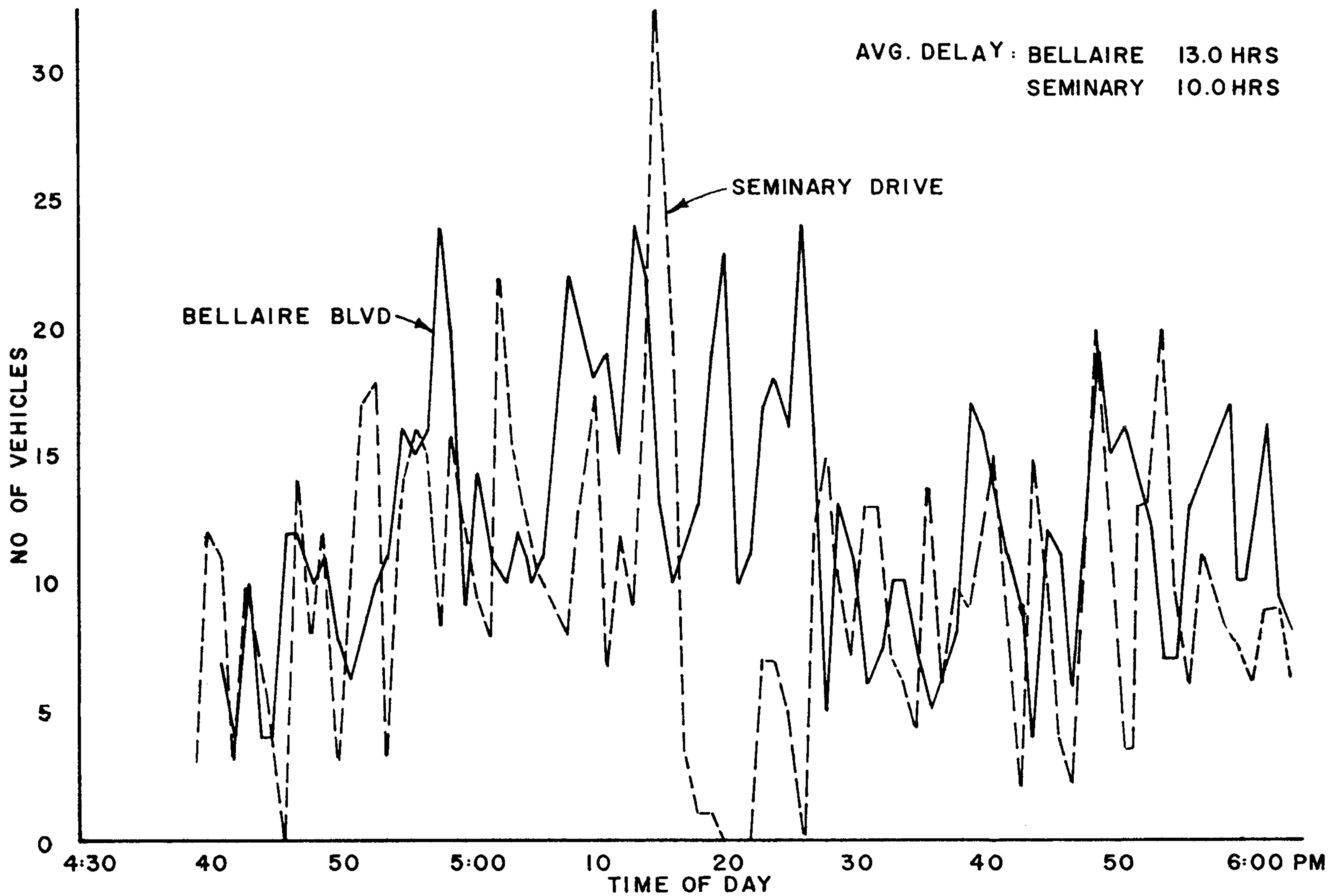
Conclusions

The inability of any signal equipment to supply the overlap feature to the diamond interchange type signal phasing should be considered as a major source of delay to the system. This has been clearly shown by the studies made at the Bellaire interchange during both peak and off peak conditions. If low traffic volumes must be subjected to high delay because of signal efficiency, it is readily apparent that the delay to large traffic volumes could not be tolerated. It has been shown that these high volumes are not just concentrated at the now recognized morning and evening peaks, but can be found at almost any time throughout the day.



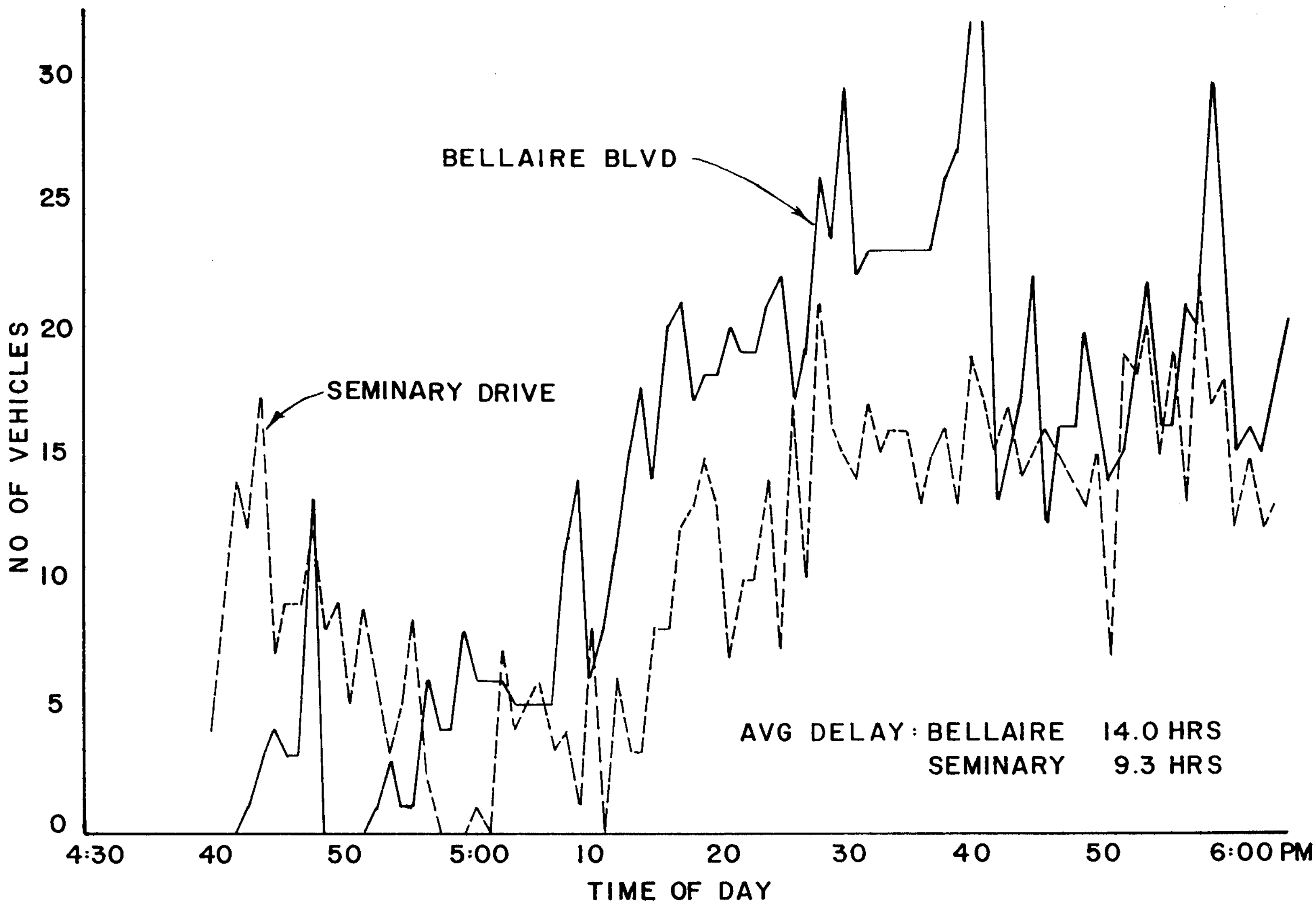
EXTERIOR APPROACH QUEUE LENGTHS

FIGURE 15



FRONTAGE ROAD APPROACH QUEUE LENGTHS

FIGURE 16



INTERIOR APPROACH QUEUE LENGTHS

FIGURE 17

SCOTT, CULLEN, AND GRIGGS FIELD STUDY

The three interchanges in this study are located on the Gulf Freeway, Interstate Highway 45, in Houston, Texas. The Scott Street and Cullen Boulevard interchanges are located downstream of the Wayside interchange and the Griggs Road interchange is upstream of it. Signal equipment at these interchanges is the fixed-time type similar to the type installed at Wayside Drive.

Due to the heavy frontage road volumes carried through the Scott and Cullen interchanges, U-turns have been built at these locations in order to increase the capacity of these interchanges. Griggs Road interchange is located at a discontinuity in the frontage road which forces all traffic to make some turn maneuver. A U-turn lane is going to be installed at the Griggs interchange and will only necessitate construction of the lane itself due to the fact that ample clearance is available, both vertical and horizontal.

Method of Study

In order that the results of these studies could be compared with the Wayside afternoon study, the field studies were made during the peak hour on the outbound frontage road and the eastbound arterial approach.

The input-output manual count was used but without the recorder. Total volume on the two approaches at each site was taken and queue lengths were counted at the end of each minute. The counts were of two-hour duration and the peak hour within the study was broken out in order to find the maximum volume and maximum delay at the intersection.

Discussion of Results

The results of the three studies are found in Table 11. Scott and Cullen show very similar results because of the near duplication of traffic conditions at each location. Frontage road traffic seems to be much heavier, probably because of the extensive use of these facilities as a means of relieving some congestion from the freeway during this peak period.

Griggs Road interchange shows almost equal volume between frontage road and arterial and the same type split with the delay on these approaches. As compared to the Bellaire results, it is within five per cent of the total volume but it has about three times less delay.

TABLE 11

SUMMARY OF RESULTS
FREEWAY INTERCHANGES

Intersection Approach	Way-side	Scott	Cullen	Griggs	Seminary	Bellaire
Major Streets:						
Total Number of Vehicles (Veh)	2541	896	915	1452	1356	1072
Peak Hour Volume (Veh/Hr)	1370	484	556	801	927	720
Total Vehicular Delay (Veh-Hrs)	22.4	6.7	5.3	4.8	20.0	16.6
Peak Hour Delay	12.9	4.3	3.7	3.2	12.6	8.3
Frontage Road:						
Total Number of Vehicles (Veh)	1669	2068	2391	1570	1414	1527
Peak Hour Volume (Veh/Hr)	897	1327	1426	889	1005	1009
Total Vehicular Delay (Veh-Hrs)	14.1	15.5	15.6	6.1	13.9	19.1
Peak Hour Delay	7.8	11.5	11.0	3.4	10.0	13.0
U-Turn Volume (Peak Hour)	49	19	47	90	171	361
Per cent of Frontage Road Volume	5	2	3	7	17	36
Totals:						
Peak Hour Volume (Veh-Hr)	2267	1811	1982	1690	1932	1729
Peak Hour Delay (Veh-Hrs)	20.7	15.8	14.7	6.6	22.6	21.3
Signal Control (Type)	Fixed Time	Fixed Time	Fixed Time	Fixed Time	Actuated	Actuated
U-Turn Lanes	Yes	Yes	Yes	No	No	Yes

Conclusions

Interchanges which have similar geometrics, similar turning movements, and are subjected to comparable volumes will tend to have like amounts of delay. Fixed time signal control with proper phasing is an effective method of controlling traffic with a minimum of delay to the system. This is due in part to the fact that with a fixed-time control, every phase will be handled as the one before and the special overlap portion is always present in the cycle.

DESIGN CONSIDERATION

As pointed out at the beginning of this publication, it is of real importance to adequately design an interchange in order to provide the capacity which may someday be demanded of it. Realizing that this is a necessary step in the design procedure should also cause the designer to be more alert as to good design geometrics. It is important to determine the basic requirements of an interchange by use of the capacity-demand approach, but such a study is useless if improper geometrics are allowed to be used at the interchange. Design features which are considered important to the proper functioning of the interchange in relation to the U-turn movement have been studied and will be discussed in the following paragraphs.

Slope

The designer usually must decide whether or not to take the main freeway lanes over the existing cross street or under it. In either case the amount of side slope used will have a direct bearing on width of right of way required, the length of bridge required, and the amount of sight distance available to traffic in the intersection. The effect of varying slopes can be seen in Figures 18, 19, and 20. It can be seen that as the side slopes are flattened the amounts of right of way and bridge end span requirements increase considerably over the steeper slopes. As seen in Figure 20, the greatest amount of available sight distance is produced by the flatter slopes. The sight distances for the varying slopes were calculated by using the equation developed by Leisch. The equation for determining the available sight distance is:

$$d = \frac{c(m + k)}{m - a}$$

where

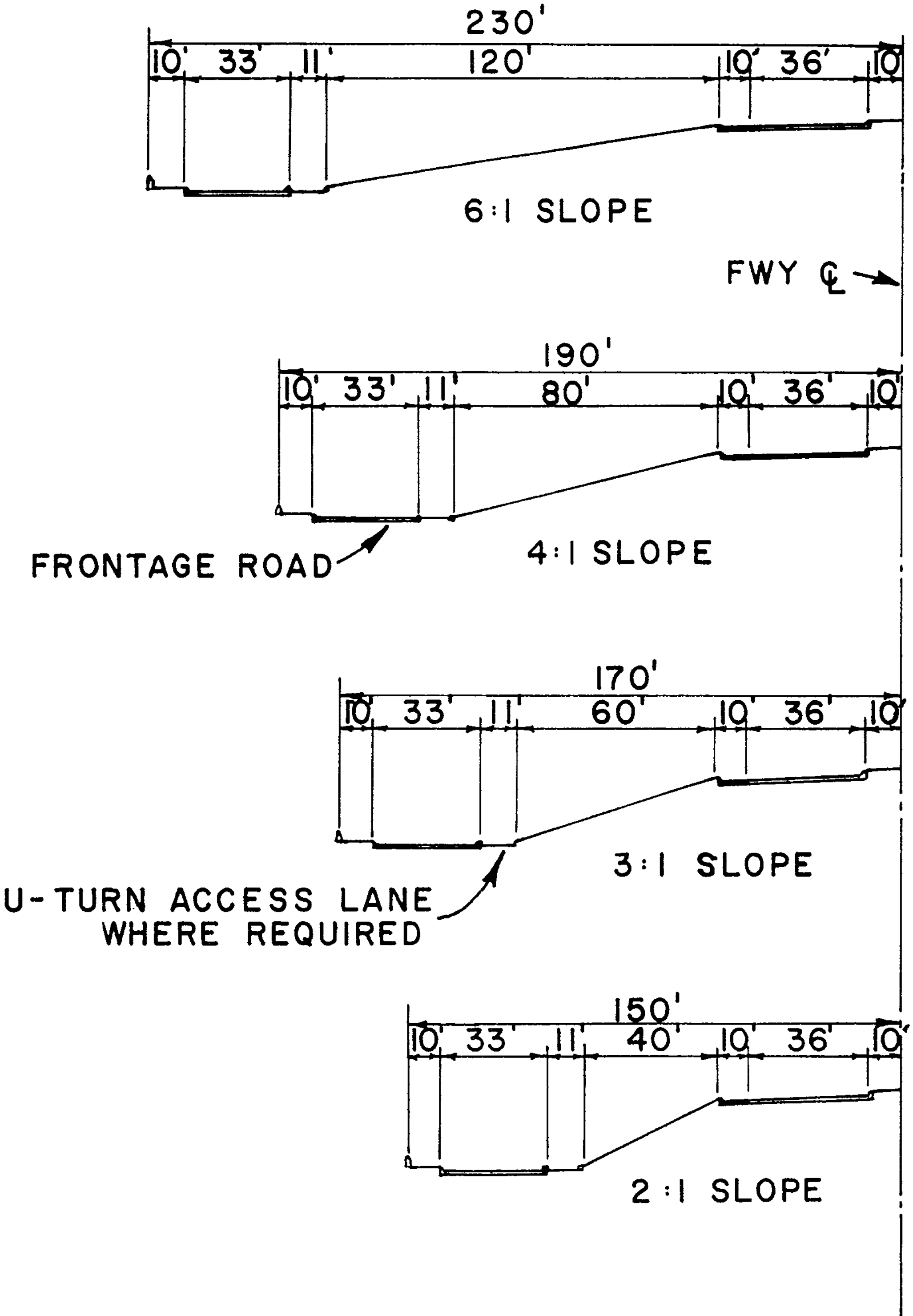
d = available horizontal sight distance measured from a stopped frontage road vehicle to an oncoming vehicle on the arterial

a = horizontal clearance between edge of travel way and obstruction in question ($a = 6$ in this example)

c = distance between frontage road and end of a grade separation structure, varies with slope of fill

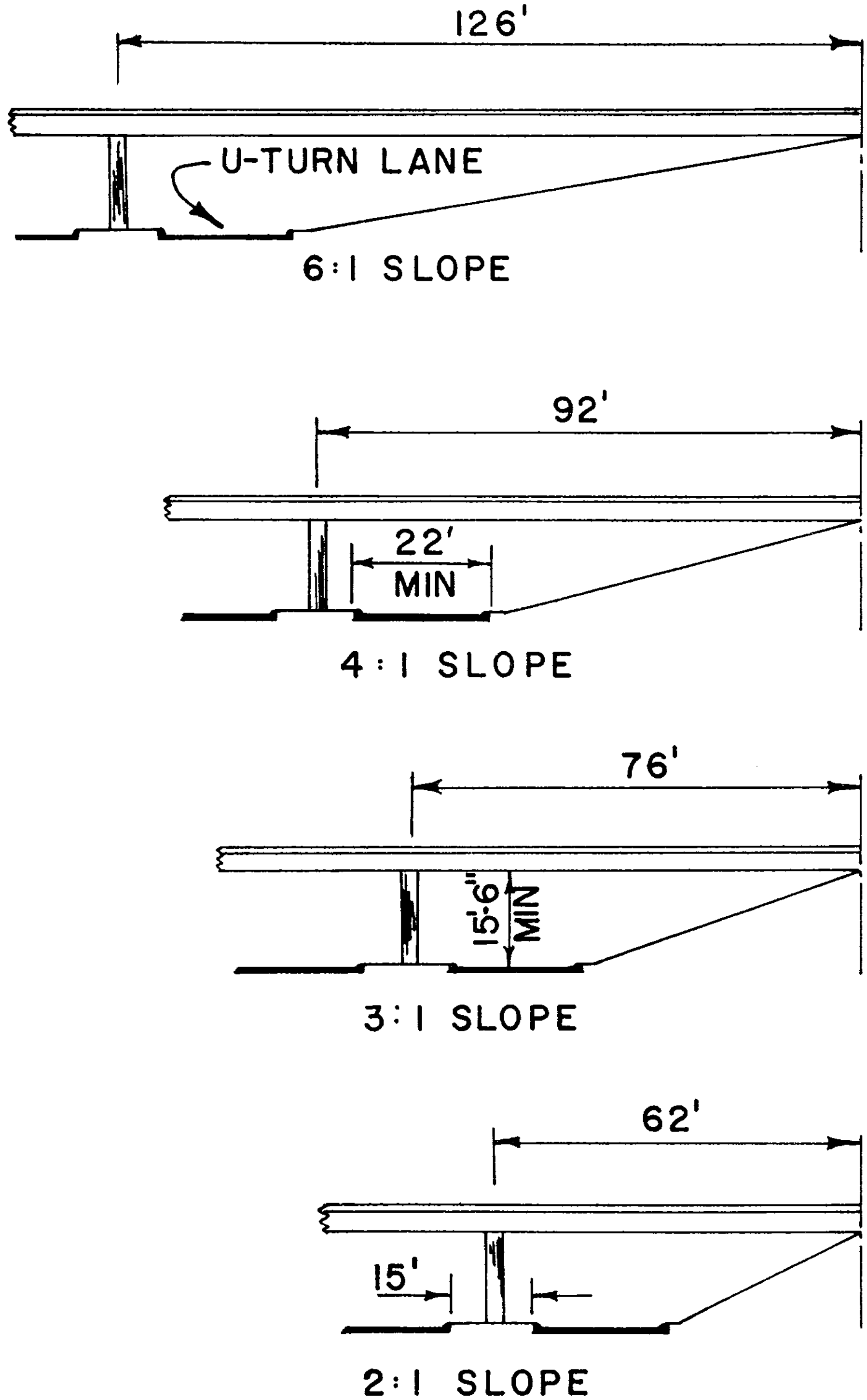
m = distance from edge of traveled way to eye of driver in stopped vehicle ($m = 10$, 12, 15 in example)

k = distance between edge of traveled way and left side of the oncoming vehicle on the arterial ($k = 8$ in example)



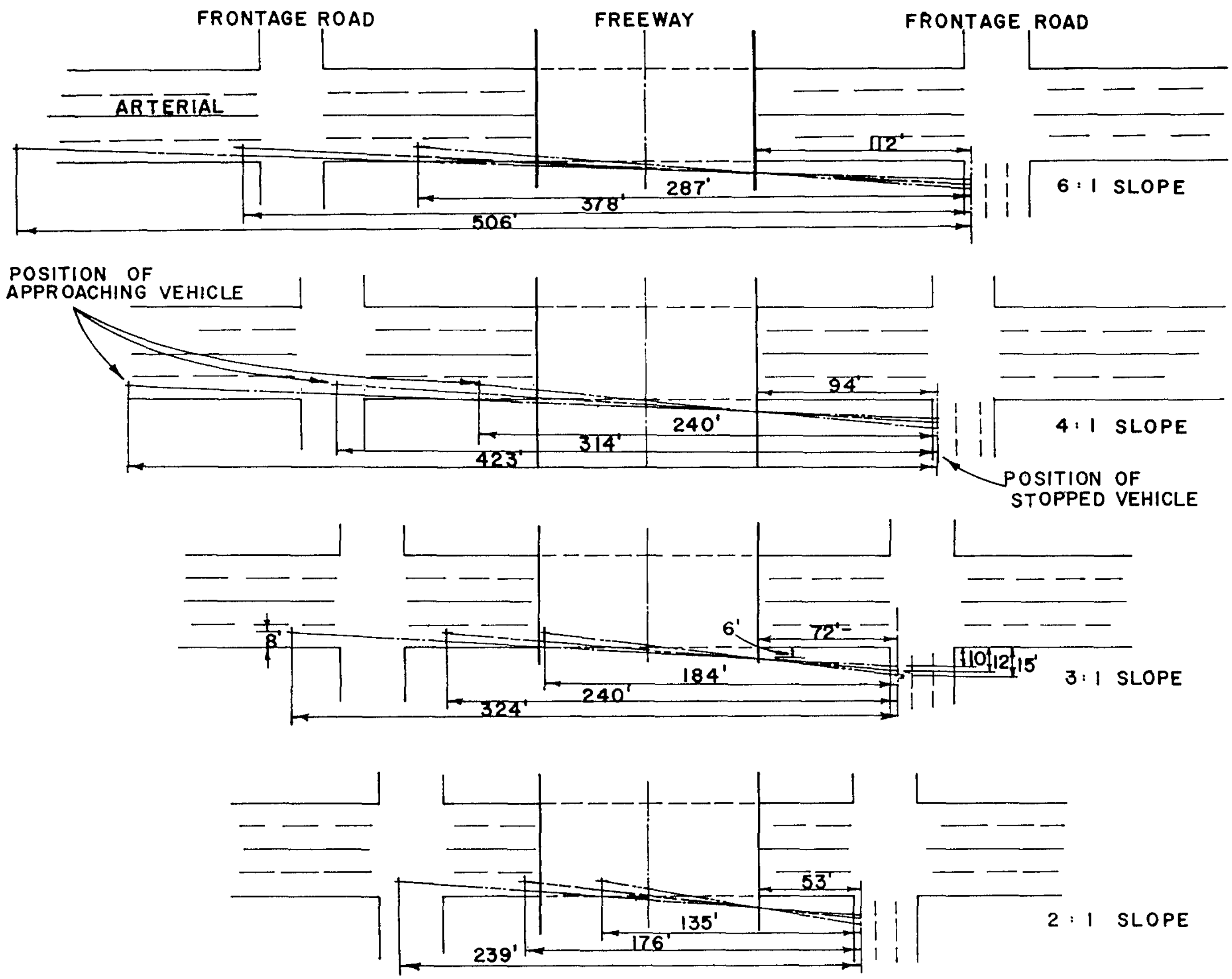
MINIMUM RIGHT-OF-WAY FOR VARIOUS SIDE SLOPES

FIGURE 18



MINIMUM BRIDGE SPAN FOR VARIOUS END SLOPES

FIGURE 19



AVAILABLE SIGHT DISTANCE FOR VARIOUS SIDE SLOPES
FIGURE 20

The "a" and "c" dimensions will vary from interchange to interchange because these values are decided upon by the designer in the development of the interchange geometrics. The values of "m" and "k" depend on the driver's choice and the values selected for a design on the basis of vehicle size and experience in traffic operations.

It is not fully realized the extent to which adequate sight distance can influence the operational efficiency of an interchange. In most cases of design, the interchange geometrics are chosen and then the sign distances are checked.

The final selection of a side slope for fill sections must be made by the designer after he has placed all of the restrictions as governed by available right of way, bridge costs, fill cost, maintenance costs, and minimum acceptable sight distance on the interchange design. The typical cross section chosen for the remainder of this design section is shown in Figure 21. It represents a compromise of reasonable right of way requirements, satisfactory bridge end span, easily maintained side slopes and an acceptable range of sight distances for speeds usually found in the interchange areas.

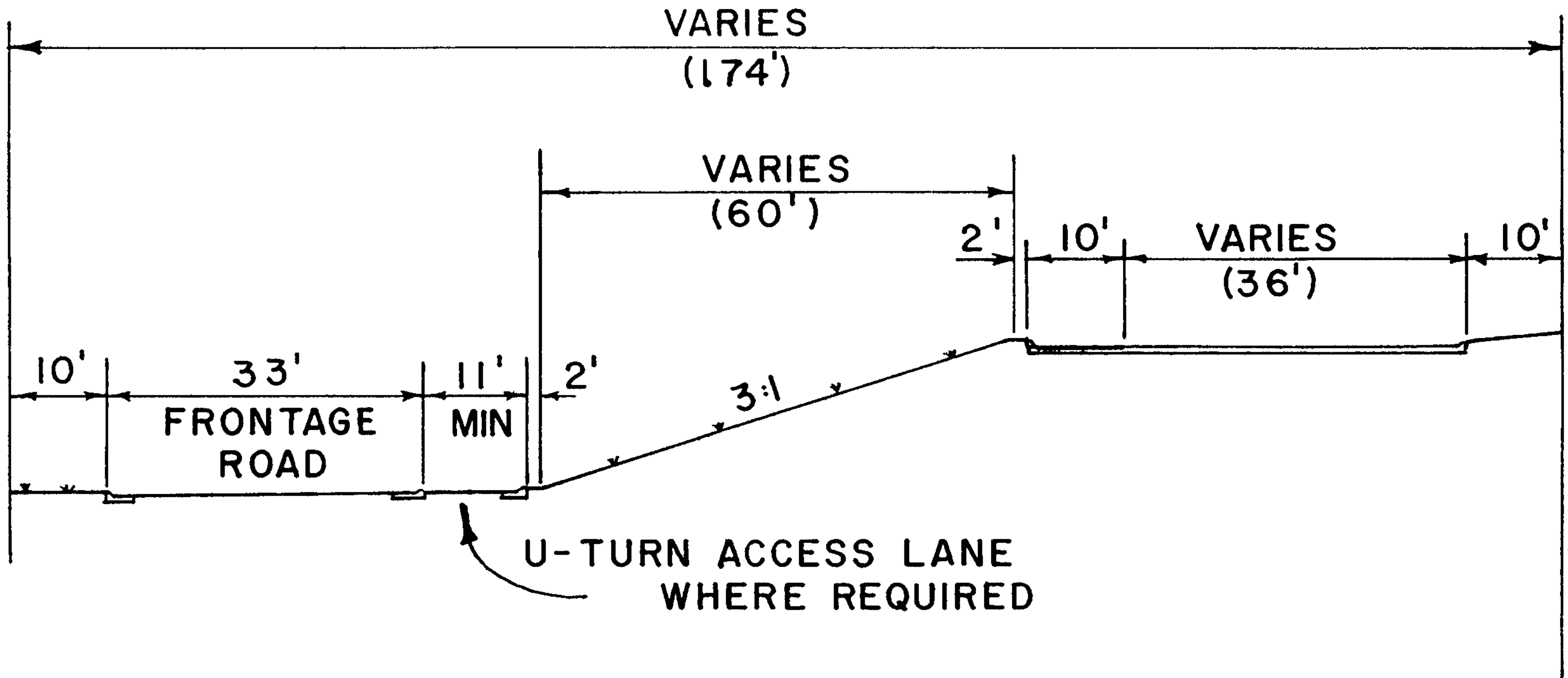
U-Turn Lane

It has been shown that land use is subject to change in the freeway vicinity and therefore, design of the proposed interchanges should be flexible enough to permit handling situations which may develop from the introduction of high volume generators. For this reason the semi-trailer truck is to be used as the design vehicle for the minimum requirements of the U-turn lanes. Although this type of traffic is only a small per cent of the total traffic, its ability to disrupt free flowing traffic conditions because of insufficient design of turn radii, storage facilities, and lateral and vertical clearances should be of prime concern to the designer if he plans to provide free flowing conditions under even extreme circumstances.

This vehicle requires a minimum radius of 50 feet and a pavement width of 25 feet if curbs are to be used on both sides of the roadway. This would provide for one-way operation with no provision for passing.

Lateral and Vertical Clearances

A minimum lateral clearance of six feet should be established from all columns and abutments where possible. This will provide an island of 15 feet between the arterial and the U-turn lane.



TYPICAL HALF-SECTION OF FREEWAY

FIGURE 21

A minimum of 16 feet 6 inches vertical clearance should be provided between the high point on the U-turn lane and the lowest point of the structure which can affect the travel under the structure. This clearance should be increased to 17 feet 6 inches if the roadway is to handle any interstate traffic. Figure 22 shows these features of the U-turn roadway.

Bridge Span Required

When the above design requirements are applied to an interchange, it necessitates the lengthening of the structure by an additional 40 feet for each U-turn lane proposed. It is recommended that an 80-foot span be used instead of two 40-foot spans. Figure 22 shows this desired condition of the end span of the bridge.

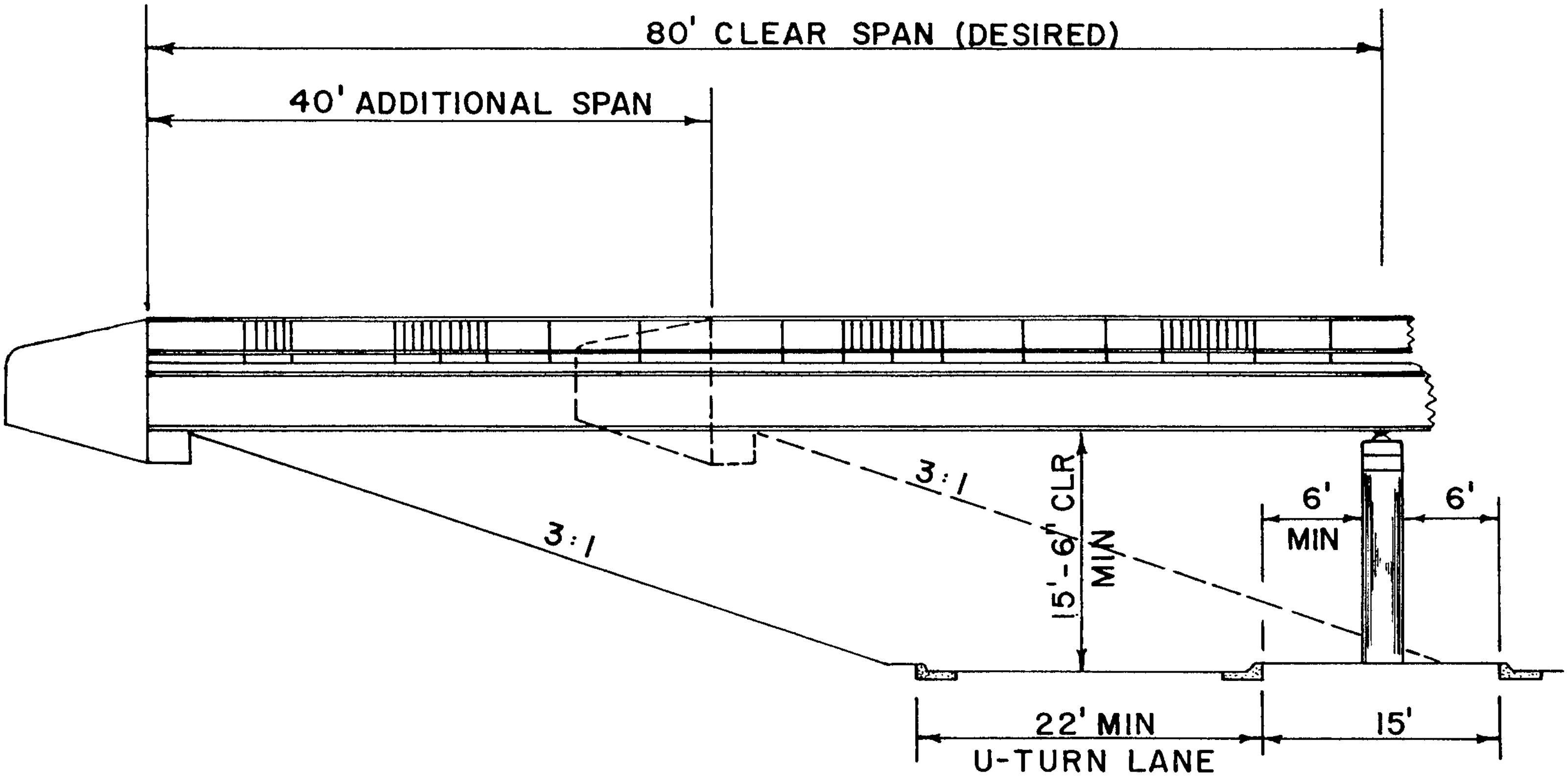
It is desirable to have as open an area as possible at all interchanges. Being able to see the other traffic gives the driver a greater sense of confidence and as a result the operating efficiency of the interchange is increased. Where short spans are used on bridges, the number of columns blocking the view of the driver is increased. If some degree of skew is also present, the staggered columns tend to create a massive front of concrete as the sight distance is reduced. The structure produces the effect of driving into a tunnel and because the driver cannot see what might be behind this forest of concrete columns he is forced to enter the area with caution by reducing speed and increasing headway. He may even be so engrossed in looking for approaching vehicles that he is not fully aware of his upcoming signal indication at the second intersection. These situations seem insignificant in the design stage, but once the condition exists in the field, the result is reduced capacity.

If the structure happens to be skewed with reference to the cross street, it will not always be possible to provide the desired 80-foot clear span as mentioned above. If such is the case, the required number of spans should be determined, but the desire to keep the area under the structure as open as possible should influence the decision as to the number and arrangement of spans used.

Travel Paths on U-Turn Lanes

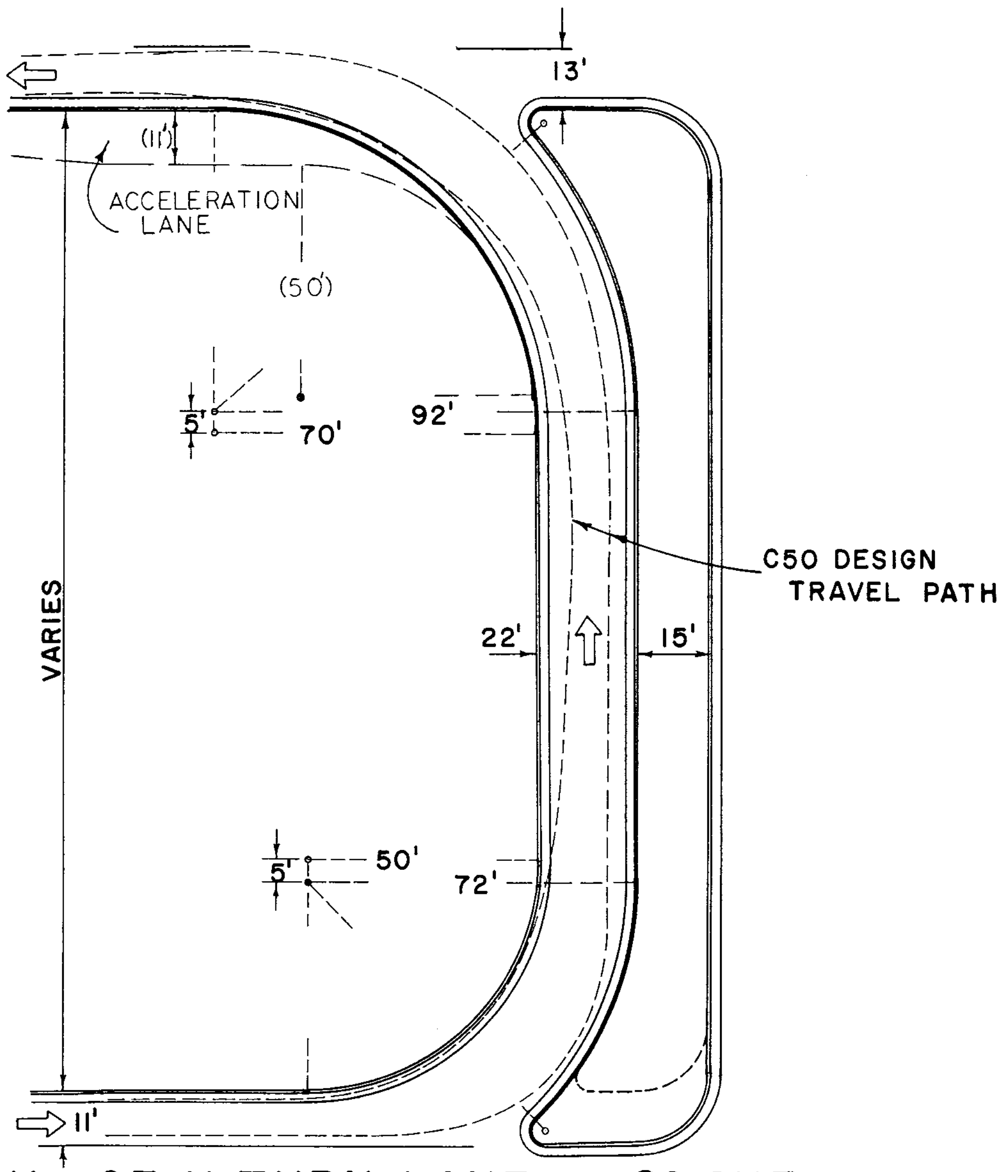
The minimum travel path requirements for the various degrees of bridge skew are shown in Figures 23 through 27. The vehicle used is the C50 type and the radii are such that the vehicle does not have to swing wide to enter the U-turn lane or leave but can remain in the inside lane of either frontage road.

When conditions other than the minimum exist at an interchange, the travel path of the U-turning vehicle can be adjusted to relax the tight radius required to negotiate these particular curves. Because of the infinite variety of interchange conditions, the U-turn travel path can range from a near straight line to a semi-circle.

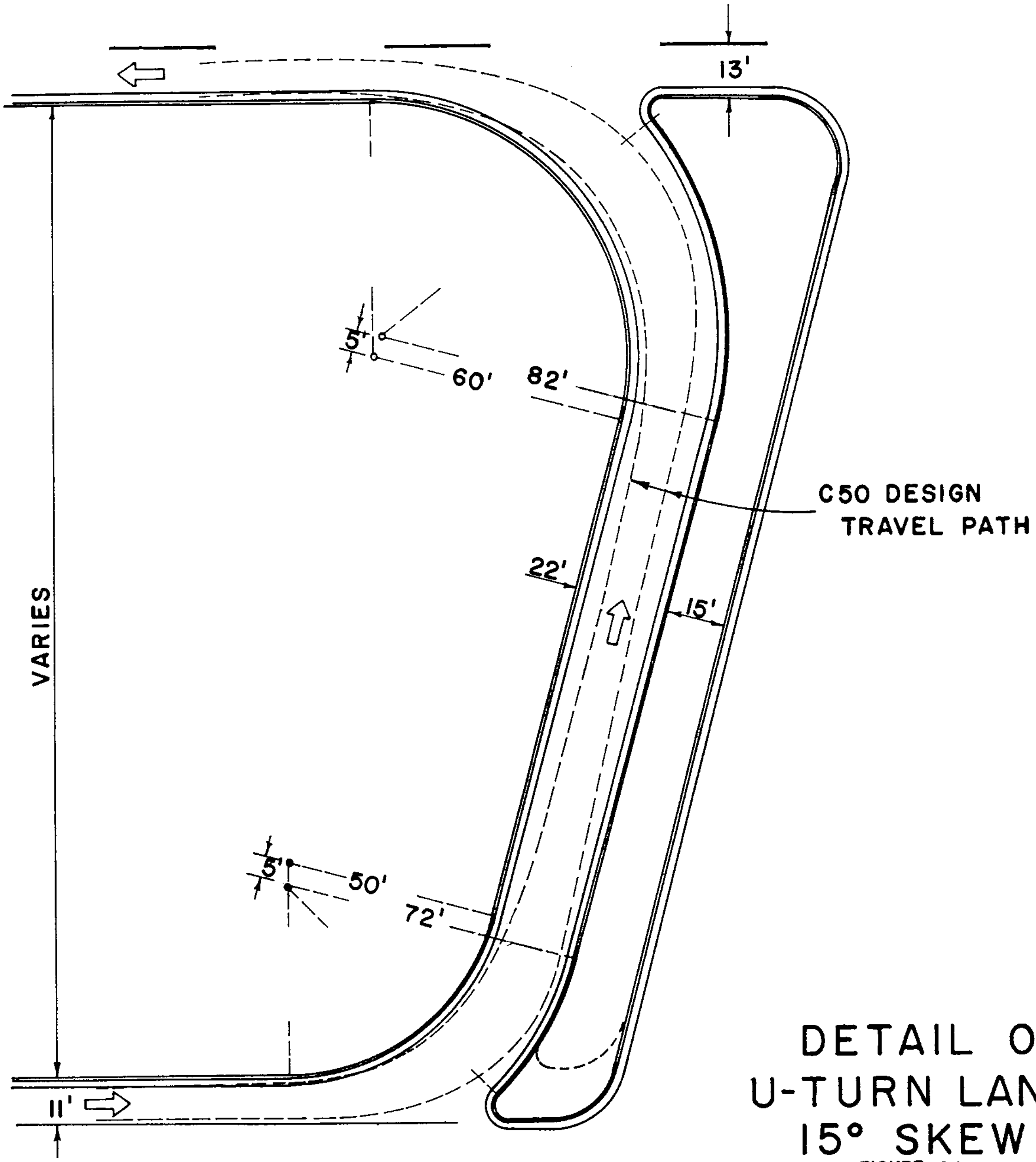


INCREASED BRIDGE REQUIREMENTS
FOR ADDITION OF U-TURN LANES

FIGURE 22

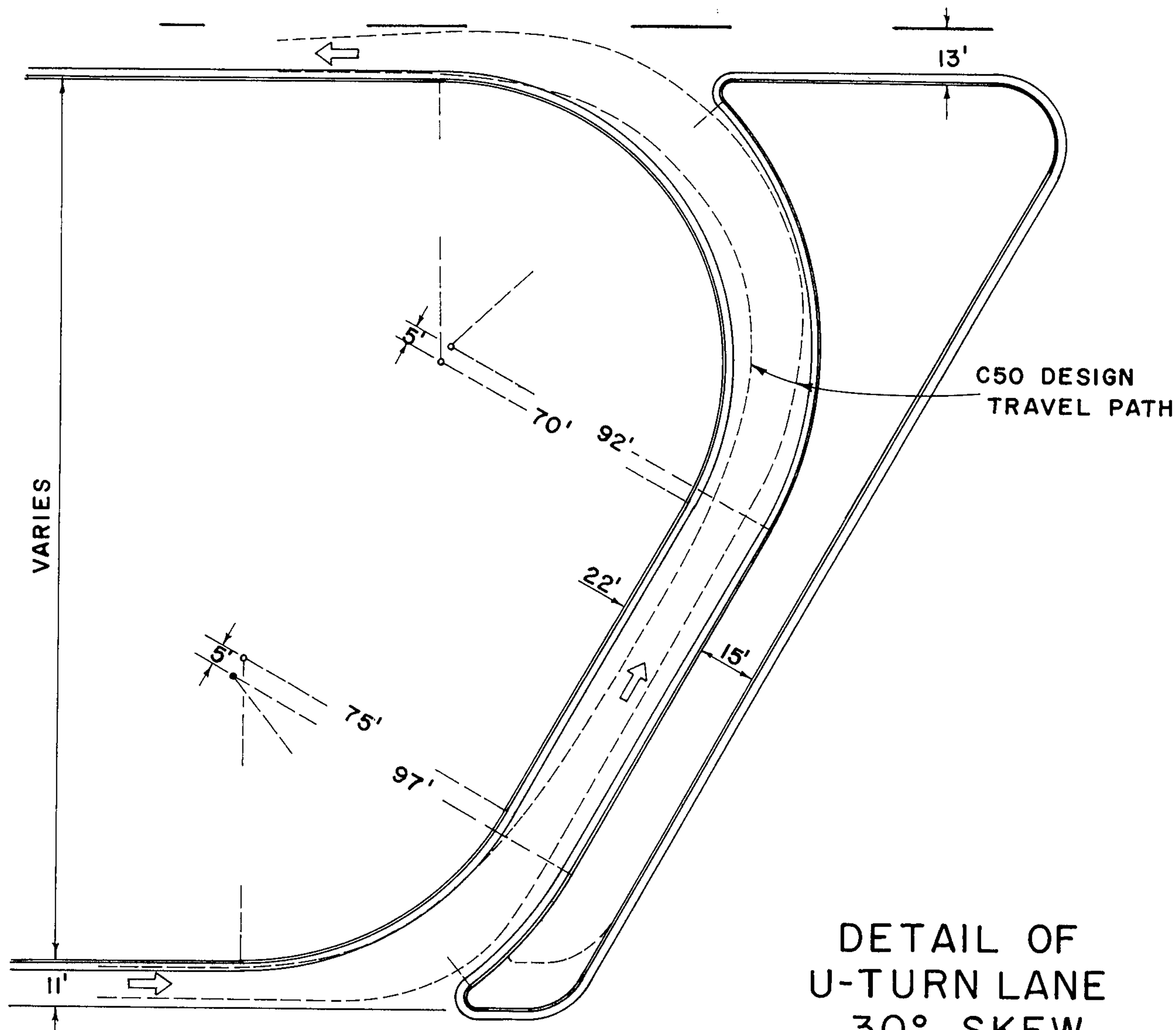


DETAIL OF U-TURN LANE 0° SKEW
FIGURE 23



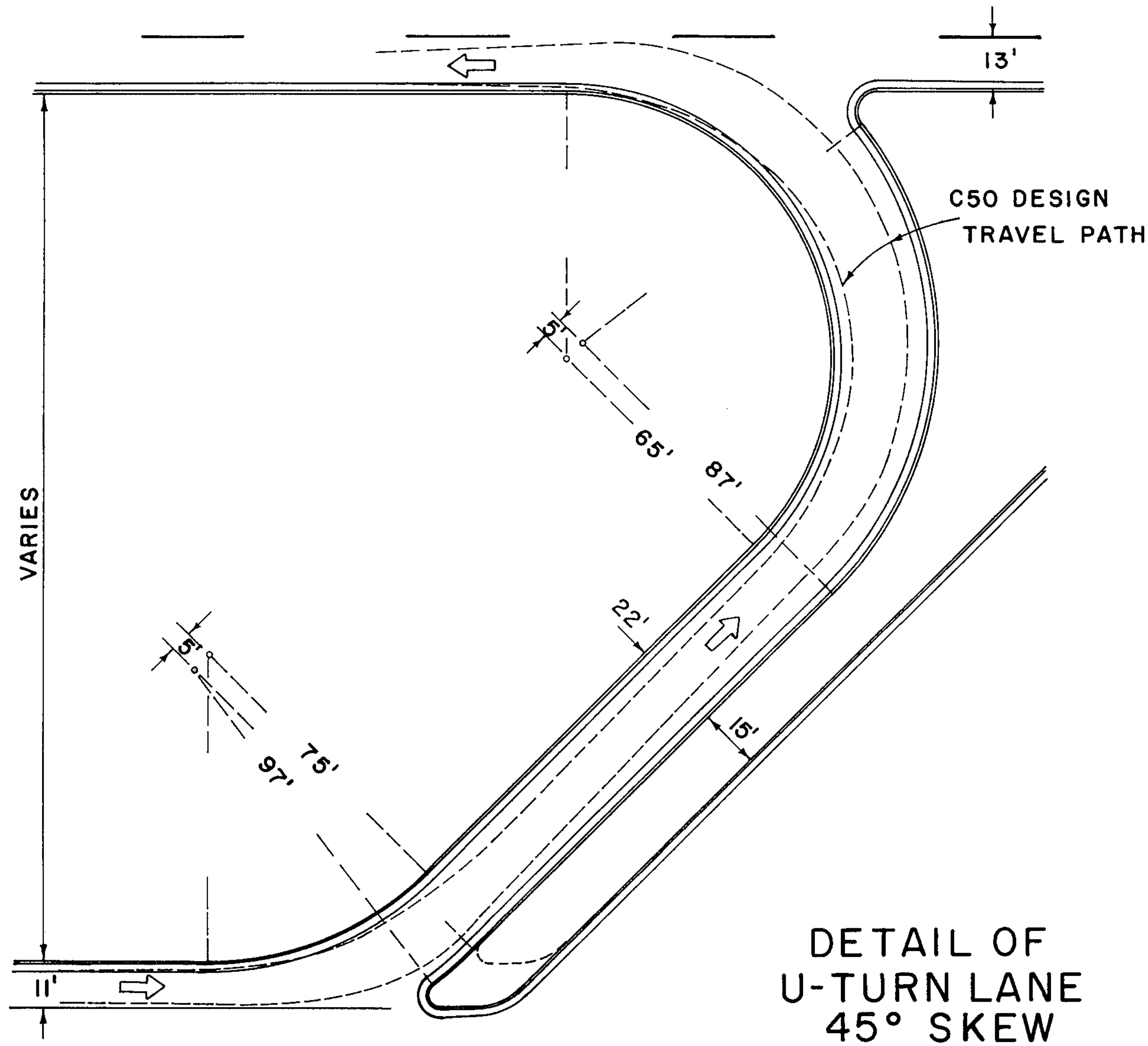
DETAIL OF
U-TURN LANE
15° SKEW

FIGURE 24

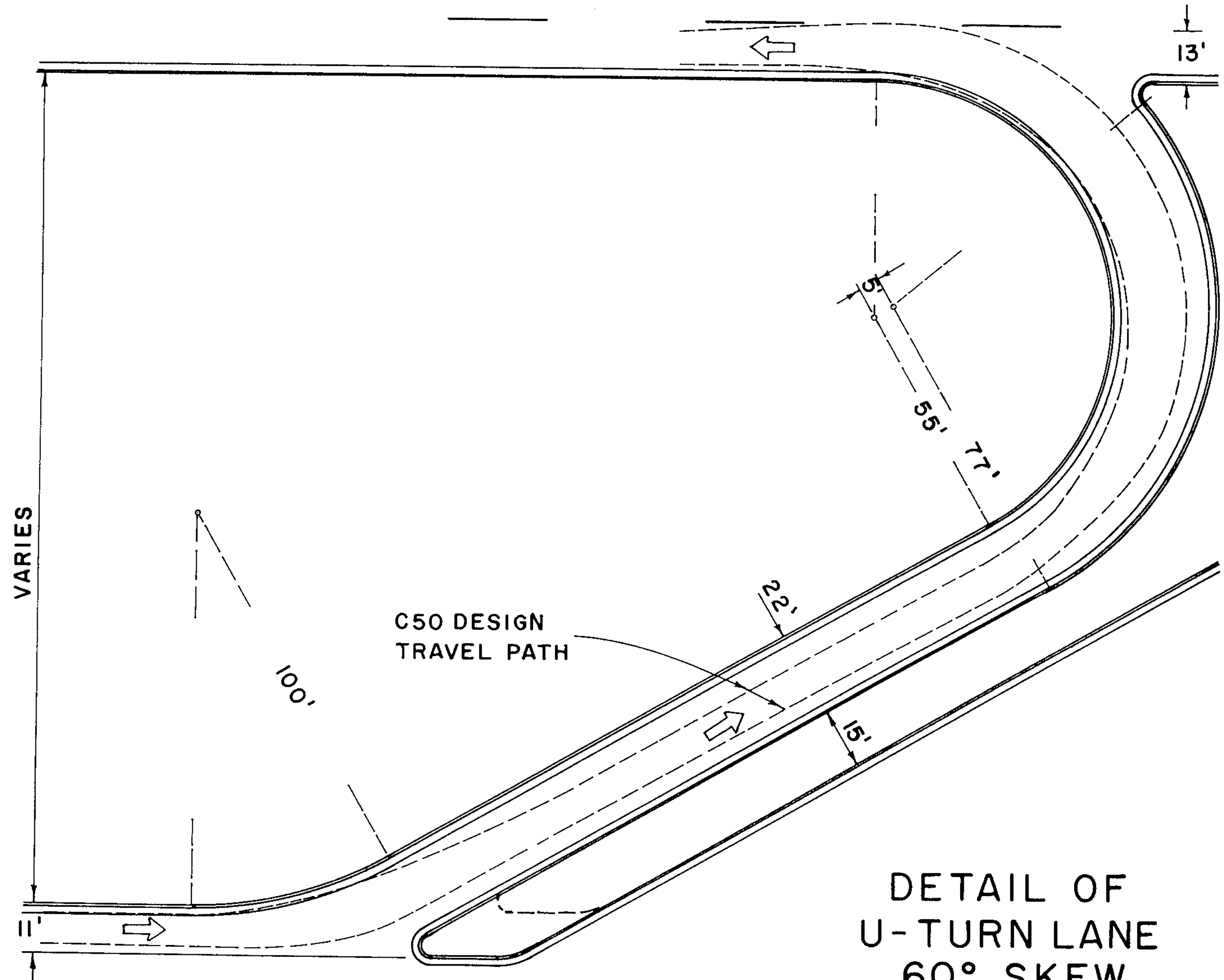


DETAIL OF
U-TURN LANE
30° SKEW

FIGURE 25



DETAIL OF
U-TURN LANE
45° SKEW
FIGURE 26



DETAIL OF
U-TURN LANE
60° SKEW

FIGURE 27

U-Turns at Existing Facilities

It may be advisable in some cases to add a U-turn lane at an existing structure. Since the bridge cannot be lengthened economically, the next best thing is to excavate the end slope between the columns and abutment under the end span. As seen in Figure 28, a retaining wall is required to replace the removed slope. Such attempts at installing U-turn lanes are quite expensive and the results are not always the best because of the restricted area in which they are placed, but if the makeshift project can fulfill its purpose of eliminating significant delay, it is considered worth the expenditure.

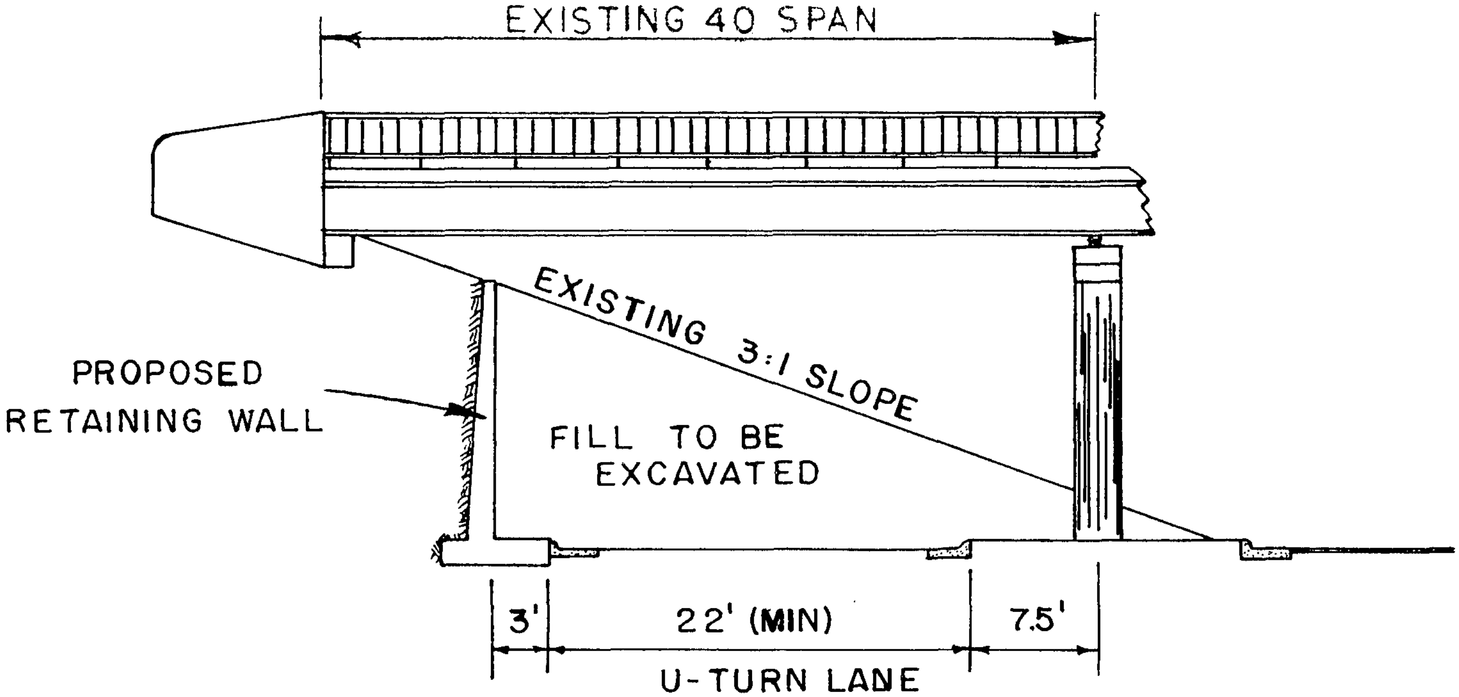
U-Turn Access Lanes

Early attempts at installing U-turn lanes at interchanges were concentrated mainly on providing some access to the opposite frontage road without having to go through the two intersections. The lanes were located close to the arterial and entrance was gained into them from the inside lane of the frontage road. As the volume increased at these intersections, this inside lane was in demand by both the left turn and the U-turn vehicles. The conflict caused a shutdown in the free flow of the U-turn because of left turn vehicles waiting for the signal. Left turn vehicles were delayed because of U-turning traffic allowing the actuated signal to gap out on short detector placements. All other phases were delayed when detectors were located at greater distances back because of U-turning traffic extending the green phase but not utilizing the intersection.

If any traffic movement is worthy of coming under specific design analysis and is found to require a special lane to handle that movement, the design of that lane, be it left turn, right turn, straight, or U-turn, should be adequate in all respects. In the case of the U-turn lane, this would mean not only the minimum turning radii and width of lane, but also the ability of traffic to get into the lane quickly and smoothly and without undue delay to either the potential user or the frontage road user.

Since bridge lengths are kept to a minimum in most standard designs, this means that the U-turn lane will be in proximity of the arterial and traffic desiring to enter the lane could be delayed due to frontage road traffic being queued up at the signalized intersection. To maintain a free flow condition, an access lane may be constructed for the traffic desiring to get into the U-turn lane. The length of this lane is dependent upon several factors, such as amount of U-turn volume, average queue lengths and type of signalization control at the intersection. If the U-turn volume is quite high, a design similar to the Bellaire interchange might be preferred as it provides an access lane from the closest parking lot exit to the U-turn lane. In most cases, this length of access lane would not be required.

Figure 29 shows a minimum length of lane which consists of a 50-foot minimum tangent section and a 120-foot tapered section. For this design, an available



INSTALLATION OF U-TURN LANE TO EXISTING FACILITY

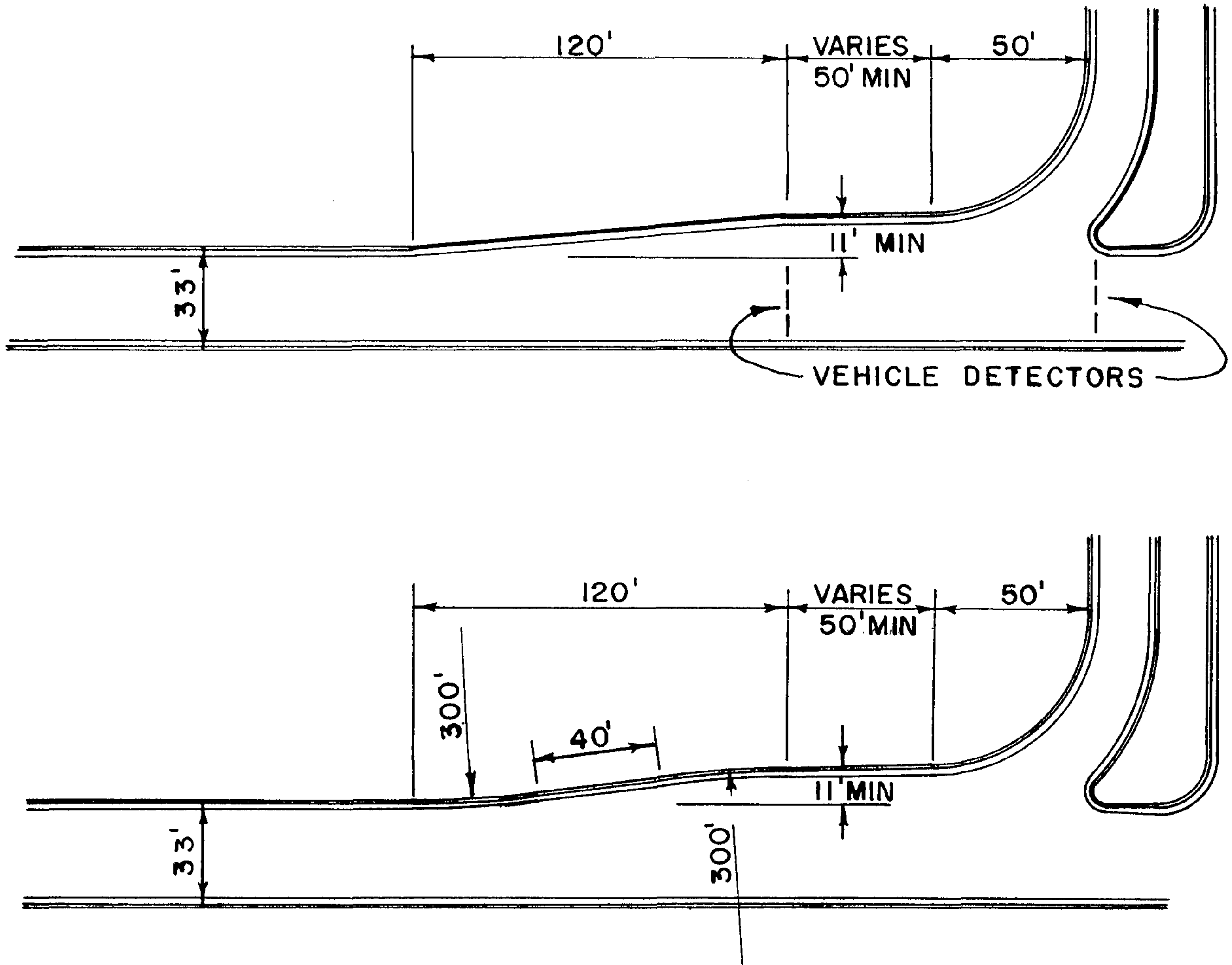
FIGURE 28

storage length of approximately 150 feet is provided for frontage road queues from the stop line to the beginning of the taper. This will handle a 7-car queue in the inside lane before the free entry to the access lane is closed. If the average queue is greater than this length, then the access lane should be lengthened if the unrestricted movement of U-turning vehicles is desired. If actuated signal control is to be used at the intersections, the detector spacing on the frontage road approach should be used as another minimum guide line. Since this phase is usually controlled by a minor movement controller, detector spacing of 75 to 100 feet from the stop line will be adequate in most cases.

A minimum width of 11 feet should be used for this access lane and a minimum taper of 120 feet from the frontage road to access lane. Figure 29 shows two types of minimum design for the access lane to the U-turn lane.

Conclusions

Proper design of U-turn facilities is essential for good operation at diamond interchanges if the delay caused by the U-turning vehicles is to be reduced. Initial consideration of these facilities early in design will not only assure ample space for them at a later date but will also provide a greater flexibility during state construction of freeway interchanges.



DETAIL OF U-TURN ACCESS LANES

FIGURE 29

CONCLUSIONS AND RECOMMENDATIONS

Conclusions

From the investigations of U-turn maneuvers at various interchanges and of the basic design criteria for these interchanges, the following conclusions were drawn:

1. U-turn traffic at interchanges which contain no U-turn facilities is a source of delay to the system. Not only are the U-turn vehicles delayed, but they have the potential of affecting the vehicles on all of the other approaches as well. This delay is not confined to any one time period during the day but is found to occur whenever the U-turning vehicles, heavy or light, are not properly handled through signal control.

2. Improvement in signal efficiency has a larger effect on improving congested interchanges than do some geometric changes. Either actuated or fixed time signal equipment can be utilized at diamond interchanges satisfactorily provided the special four phase overlap signal phasing is provided and continuously obtained.

3. Interchanges should be adequately designed for all features which are considered necessary for efficient movement of high volume traffic at the intersections. Any movement that has a high delay potential should be eliminated from the interchange traffic and by providing the necessary facility to handle this delay causing traffic, better operating conditions will exist at the interchange. Since U-turning traffic is of the type mentioned above, it should be adequately provided for through the design of special U-turn lanes at all diamond type interchanges.

Recommendations for Future Studies

Because the provision of U-turn facilities at all interchanges has not become an accepted design practice and since the results of this study have indicated that such facilities could improve operational efficiency in these areas, it is recommended that additional studies should be undertaken to develop specific warrants for the inclusion of these facilities in basic interchange designs. These studies should include:

1. Cost studies to determine what additional cost would be required for the provision of the U-turn facility initially or at some future date as compared to the standard acceptable design of today.

2. Simulation studies to determine the effect of various percentages of U-turning traffic on the capacity of the diamond interchange.

3. Accident studies to determine if this U-turning traffic is a major contributor to the accident picture in the interchange area or if the design of the interchange itself is the cause.

4. Economic studies to examine the cost of operation of different types of vehicles at interchanges with and without the U-turn facility and to investigate the economic influence on roadside and business development that these facilities may have because of the way they affect the accessibility to these areas.

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