

HIGHWAY RESEARCH RECORD

Number 9

Freeway Design Considerations

2 Reports

Presented at the
42nd ANNUAL MEETING
January 7-11, 1963

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Evaluation of Frontage Roads as an Urban Freeway Design Element

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This paper discusses the design and operational aspects of frontage roads and diamond interchanges and develops criteria for their use. Continuous frontage roads have been used extensively on urban freeways in Texas and much operational experience on freeways using these elements has been obtained. Freeway studies have indicated that numerous advantages in system flexibility, capacity, operation, and construction have been obtained with this type of facility and this paper discusses these advantages and presents supporting data obtained from freeway studies in Texas.

A capacity analysis is developed for conventional two-level, split two-level, and three-level diamond interchanges. The results of this analysis indicate the wide range of interchange capacity that can be obtained with these three variations. The capacity analysis also points out various design criteria necessary to obtain maximum operational efficiency and capacity.

•A FRONTAGE ROAD is defined as a local road auxiliary to and located on the side of an arterial highway for service to abutting property and adjacent areas and for control of access (1). Its function is to control access to the traveled way for through traffic where it cannot be controlled otherwise, to provide access to the property adjoining the highway, and to maintain circulation of traffic on the street system on each side of the arterial highway (2). An example of an urban freeway-frontage road system is shown in Figure 1.

From the preceding definition and statement of function, it is easy to see how the frontage road has acquired several commonly used names, such as frontage road, auxiliary road, service road, access road, and collector-distributor. Each of these names is related to a particular function of a frontage road and collectively they indicate the multifunctions of a frontage road. The term "frontage road" is a more common name for the facility discussed in this paper and for that reason, it is used throughout this report.

A frontage road serves a multitude of purposes that are probably not widely recognized. Often, the frontage road is thought of only as a means of providing frontage to abutting property for the control of access. However, the frontage road also adds tremendous flexibility to the operation of a freeway when used as an auxiliary facility.

This paper focuses attention on the frontage road as a design element of an urban highway network and presents data regarding the frontage road, its uses, and its benefits to urban transportation.

ECONOMIC CONSIDERATIONS

The manner in which access to a freeway is regulated is an extremely important factor and affects both the operation of the freeway and development of areas adjacent

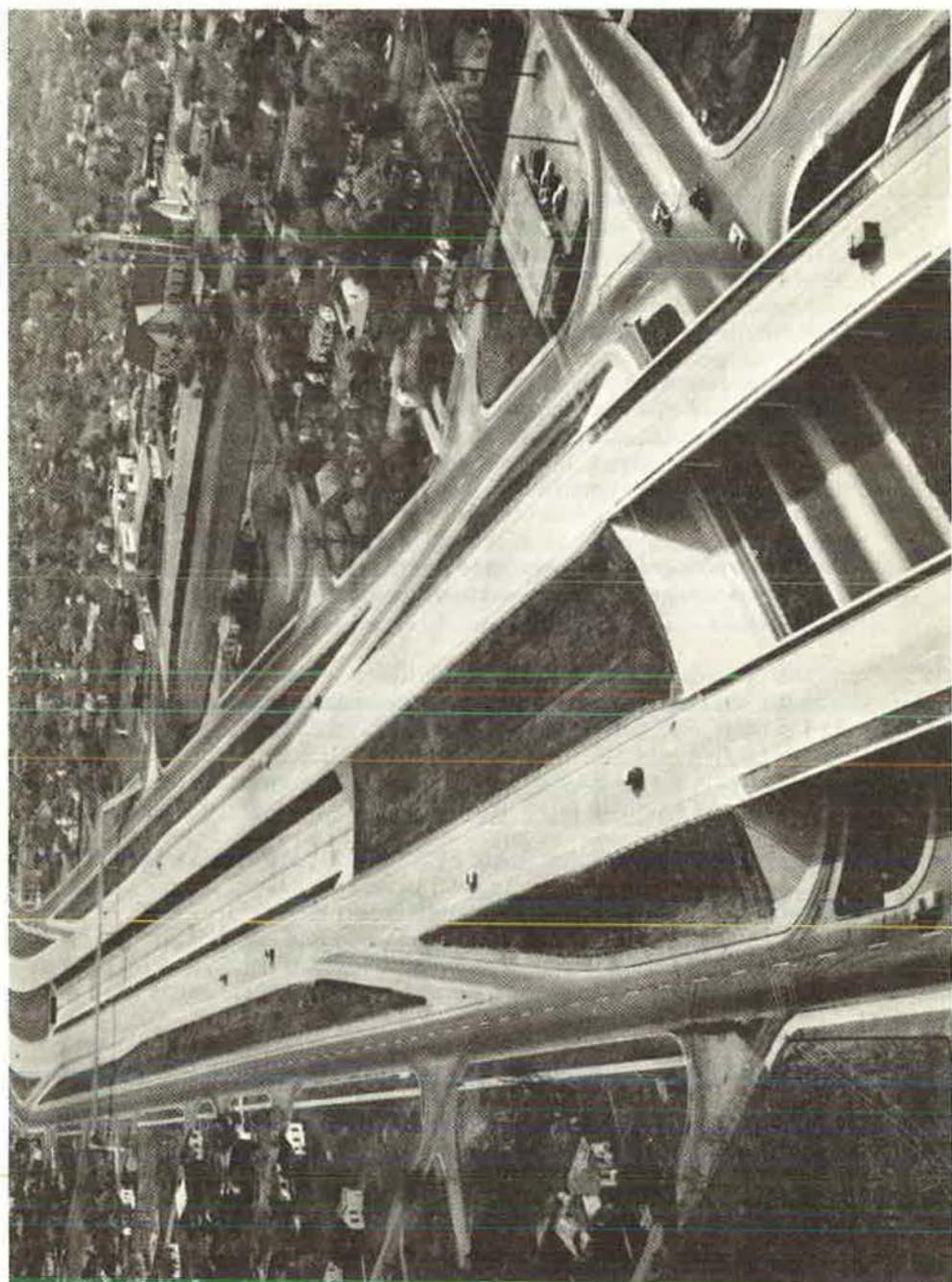


Figure 1. Urban freeway-frontage road system.

to it. The connections to the freeway proper have become fairly well standardized and are therefore not considered in this discussion. The arrangement of the off-freeway end of the ramp has been treated in a number of ways which can generally be categorized as frontage-road and non-frontage-road or closed-corridor designs. The relative economics of these two have been discussed at great length, but possibly some of the significant considerations have not been fully developed.

When the closed-corridor method of controlling access to a freeway is employed, it is necessary to secure either by purchase or by other means any right of access that the owner of any adjacent property may have to the freeway. It may also be necessary to compensate him if his property has been cut into two parts by the freeway. If, before building the freeway, access to his property has been gained by way of a road that is to be replaced by the freeway, it may be necessary to compensate him if he can no longer get to his property or to provide him an alternate access route. These things all cost money, and the value of some of these is often hard to determine accurately.

In theory when a freeway is put on a new location where no road existed previously, the adjacent landowners do not possess any right of access to the freeway. In dealing with these situations, however, the land often costs more than it should and part of this additional compensation is no doubt due to the fact that the landowner is not given access to the freeway.

Damages resulting from the construction of a freeway are extremely hard to determine accurately, and even in the case of a closed corridor, are actually apt to be negative or result in enhancement of value in the long run. Probably the most difficult feature of the entire problem, however, is to treat all property owners fairly.

For example, there is a 3-acre tract of land (Tract 1) located on a crossroad at a closed-corridor freeway interchange (Fig. 2). This property is accessible from the freeway due to the crossroad interchange and is subject to an almost immediate change in land use and to a considerably higher selling price. The fact that the side of this property adjacent to the freeway has an access control fence along it for its entire

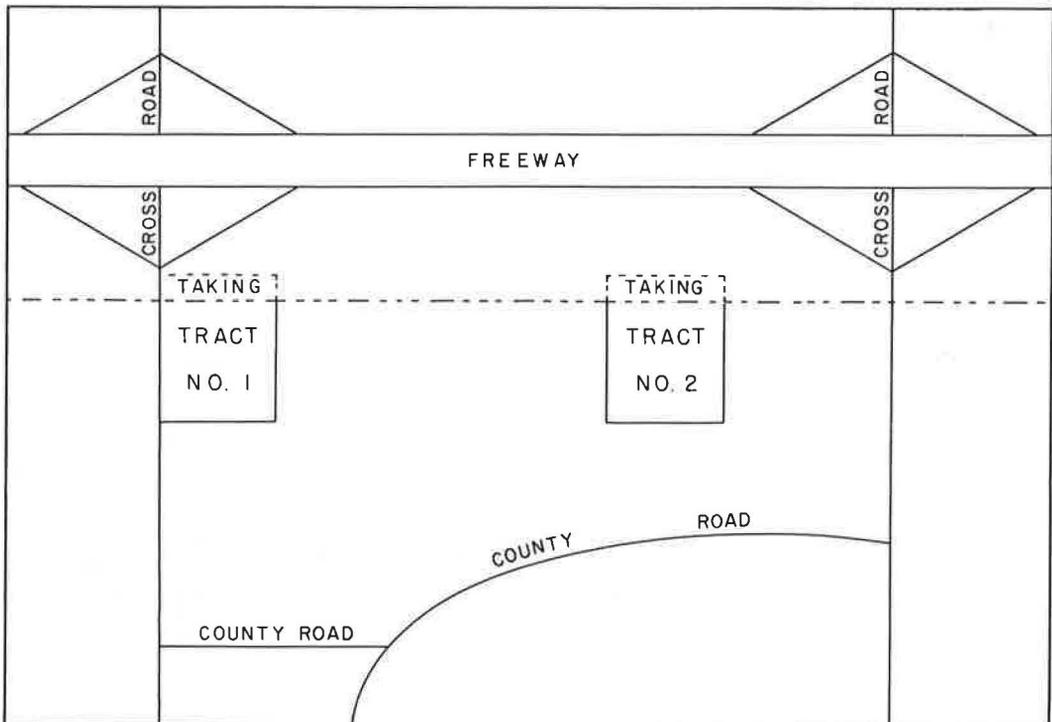


Figure 2.

length is not a major factor because access to the freeway can be gained by way of the crossroad and signs can be erected that are clearly visible on the freeway. The same size tract of land located adjacent to the freeway but midway between interchanges (Tract 2) is in theory similar to Tract 1 in value but is not subject to nearly as great a change in value as that tract because it does not have access to the freeway. Assuming that there are no severance damages to either tract and that each still has the same access to the public road system it had before the freeway, each should theoretically be treated the same as far as right-of-way costs are concerned. This is where the similarity stops, however, because Tract 1 is, as has been pointed out, subject to an immediate enhancement in value by reason of its location at a crossroad, whereas Tract 2 is not. A frontage road along the freeway tends to equalize the effect that the freeway has on the two tracts.

It may also be pointed out that the same comparison might be drawn between two pieces of property, one on the freeway and one a mile away. No immediate value enhancement is realized by the one a mile from the freeway. This is probably true, but no right-of-way dealings are necessary with the property a mile away, whereas it is necessary to secure property from both Tracts 1 and 2, and the inequality is evident to all concerned including a condemnation jury if it is necessary to go to condemnation to secure the needed right-of-way. Experience has shown that these juries are prone to attempt to compensate the owner of Tract 2 for the fact that he is not getting as advantageous an arrangement as the owner of Tract 1. How much additional compensation the owner of Tract 2 should have is something of a guessing game and can vary tremendously.

If a frontage road is provided, Tract 2 has approximately the same access to the freeway as Tract 1 and will enjoy about the same value enhancement as Tract 1. The frontage road also tends to spread the value enhancement over much more property and usually results in less enhancement to a few tracts that happen to be advantageously located while affecting a great many more properties.

Dual Appraisal

In attempting to learn exactly what effect these various treatments will have on the cost of right-of-way, the practice of dual appraisals has been employed on various occasions. Each tract along the freeway route which must be secured as right-of-way is appraised as if the freeway had frontage roads and again as if it had not. The additional cost of securing right-of-way without frontage roads is then compared with the cost of building frontage roads. The cost of frontage roads can be determined with some degree of accuracy and therefore is a reliable figure. The actual cost of right-of-way is not as easy to determine, and reliable appraisals may sometimes be just so much paper when a skilled lawyer finishes tearing them to pieces. Dual appraisals do, however, give a fairly reliable indication of relative costs, and the exact costs can be determined when a final settlement for the property in question is reached. Each case can become final only one way, however, either with frontage roads or without, and it is never possible to know what the final outcome of more than one of the possibilities for a particular piece of property might have been. Similar cases can be compared, however, and yield some interesting results.

In one instance a value of approximately \$106,000 was appraised for negotiation for two parcels of land. No frontage roads were to be provided. A commissioners' award of \$349,000 was rendered on the grounds that the remaining property was not adequately served. The case was appealed. At the jury trial, the property owner gained additional access across the freeway and was awarded \$193,000 compensation and damages. Frontage roads were still not included, but the owner received 182 percent of the appraised value of the taking and it is very likely that at some future date frontage roads may yet be constructed to serve this and additional property in the area. Frontage roads could have been provided initially for about \$130,000.

In another case, which is still on appeal, the appraised value of \$17,000 went up to a jury award of \$295,000, the lack of frontage roads being the prime issue. This is obviously a somewhat ridiculous situation and will undoubtedly be adjusted in the courts, but this right-of-way will cost more than it should and the entire problem could probably have been avoided if frontage roads had been provided in the original design.

TABLE 1
RIGHT-OF-WAY COST DATA ON A SMALL CITY RELOCATION

Parcel	Offer (\$)	Jury Award (\$)	Difference (\$)	Cost of Frontage Roads (\$)
2	3,575	10,230	6,655	2,216
5	13,150	17,960	4,810	186
9	5,095	9,500	4,405	5,980
11	1,217	3,673	2,456	2,240
15	1,190	2,500	1,310	1,976
19	1,600	7,000	5,400	Whole taking
23	330	2,383	2,053	308
25	4,900	19,260	14,360	4,949
28	1,535	3,237	1,702	1,242
31	4,005	16,500	12,495	3,847
34	<u>3,583</u>	<u>9,690</u>	<u>6,107</u>	<u>2,194</u>
Total	40,180	101,933	61,753	25,138

Another example is given in Table 1. The properties here were secured with the understanding that frontage roads would not be constructed. This is a relocation route around a town of 6,500 people and will probably be subject to a limited amount of development. Although this is not exactly a case in point, the evidence considered by the court indicated that the fairly considerable difference between the offer and the jury award was due to the lack of access. The construction of low-type frontage roads would have resulted in some saving in the overall cost of the facility.

These are not necessarily typical examples of the relative cost of providing frontage roads vs securing access rights because a review of numerous cases indicates there is no such thing as a typical case. These examples do indicate some of the problems involved, and numerous additional cases that have been reviewed disclose that the situation can be summarized as follows: Where the freeway is located in an area subject to any change in land use, which possibly can be used by the landowner as a point in right-of-way dealings, then construction of frontage roads will usually result in an immediate saving in overall cost of the freeway development. It is not necessary that this possibility of land-use change be too apparent or immediate.

The greater the possibility of development and the greater the potential of the area to support development, the greater the likelihood that frontage roads will prove to be economical. All of this is exclusive of value enhancement, which is enormous in some cases.

Property Value Enhancement

Numerous studies of the influence of freeways on land values have been conducted. The fact of enhancement is well established in most cases. The amount of enhancement and the countless interacting influences among such factors as specific location, type of land use, area influence, the passing of time, and traffic generation are the subject of much study and more is certainly needed.

In specific cases, land value enhancements in Dallas and Houston have ranged up to more than 600 percent in a few instances, with increases of 300 to 400 percent not uncommon. These freeways have frontage roads, however, so there can be no comparison with what the enhancement would have been without frontage roads.

In San Antonio a study by Adkins and Tieken (3) completed in 1958 drew comparisons between sections where property had access to frontage roads and where property lay adjacent to the freeway right-of-way but did not have access to it. Result of this comparison indicated the value of the property with frontage increased approximately 40

percent more than that without during the study period. Property used for nonresidential purposes where access to a frontage road was provided increased approximately 150 percent more than property that lay adjacent to the freeway but did not have access to a frontage road. This is a relatively limited amount of data and certainly could not be considered conclusive, but when all the circumstances are considered, there is every reason to believe that this condition would prevail on most freeways. Most businesses are dependent on street access to some degree and it is logical that those that are dependent on a large number of people getting to the business would pay a premium for a location on a freeway frontage road.

Several other factors may or may not show up in a cost analysis, but are very important in the development of a freeway. One of these is the attitude of the people who operate businesses or own property in the vicinity of the freeway. By and large, these people are convinced that the freeway with frontage roads will be of considerable benefit to them, whereas without the frontage roads the opposite effect will result. Because these are the people who must be dealt with in securing right-of-way for the freeway, this attitude on their part is very important. In several instances where large developments were concerned, the right-of-way for the freeway has actually been donated with the understanding that frontage roads would be constructed. This attitude may also be the difference between securing right-of-way by negotiation rather than condemnation.

OPERATIONAL CONSIDERATIONS

An urban freeway represents a tremendous investment of public funds and it is, therefore, extremely vital to obtain maximum utilization and operational efficiency from such facilities. A good frontage road system auxiliary to the main freeway lanes can greatly aid in obtaining this goal by providing much additional flexibility to the operation of the freeway facility.

Experience with freeways has shown that the development and life of a freeway facility is likely to progress in the following stages:

1. Stage development. —Period during which a freeway may progress from some initial inferior facility to the final design.
2. Freeway construction stage. —Period during which the freeway proper is being constructed.
3. Design life. —Period from completion of the design facility to the time at which design-hour volumes are being exceeded.
4. Saturated operation. —Period during which peak-hour volumes greatly exceed those for which the facility was designated.

Frontage roads are important and useful at all these stages and probably become most vital when stage 4 has been reached. The application and benefits of frontage roads during each of these stages are discussed separately.

Stage Development

In locations around the fringe of heavily developed areas, it is often economical to build the relatively inexpensive frontage roads initially to serve fairly light existing traffic volumes. The more expensive main lanes and grade separations can be added as the need develops. This makes it possible to prorate available money and to get the most use from it. Such a situation is shown in Figures 3 and 4. Figure 3 shows the completed freeway extending into the heavily developed urban area. The freeway transitions into the more economical frontage-road-only design at the edge of the developed area. As time passes and the development moves out, the freeway can be extended. Figure 4 shows more details of the transition. This arrangement allows extreme flexibility because all right-of-way is acquired at the time of the first development and frontage to all adjacent property is provided initially in such a manner that revision should not be required when the freeway is developed. This arrangement may require a heavier pavement design on the frontage road than would otherwise be required but the cost of this is usually justified, particularly if construction of the freeway lanes can be deferred for a substantial period of time.

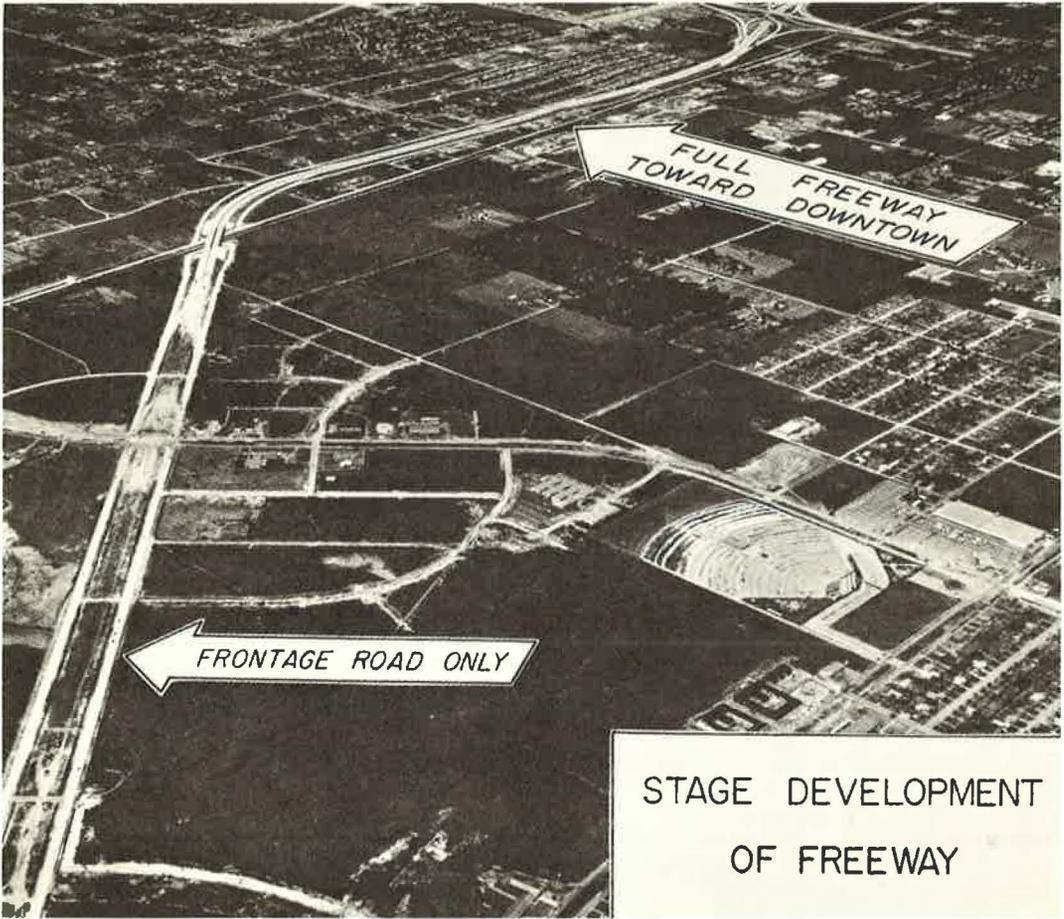


Figure 3. Completed freeway extending into heavily developed urban area.

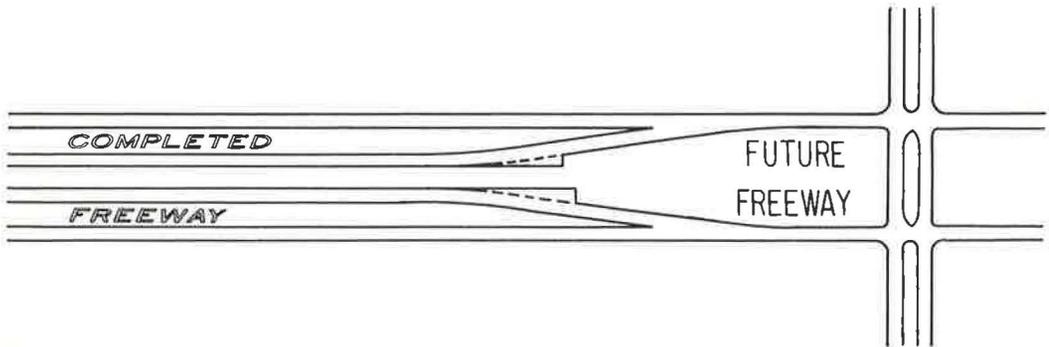
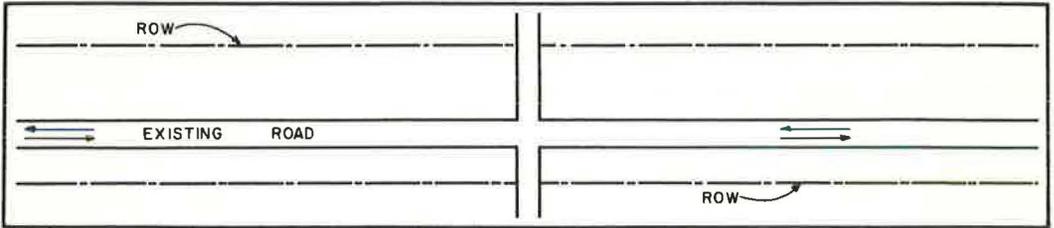


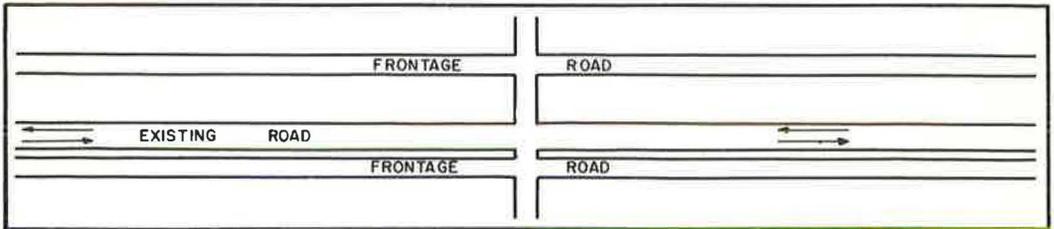
Figure 4. Transition from freeway to frontage road section.

Construction Stage

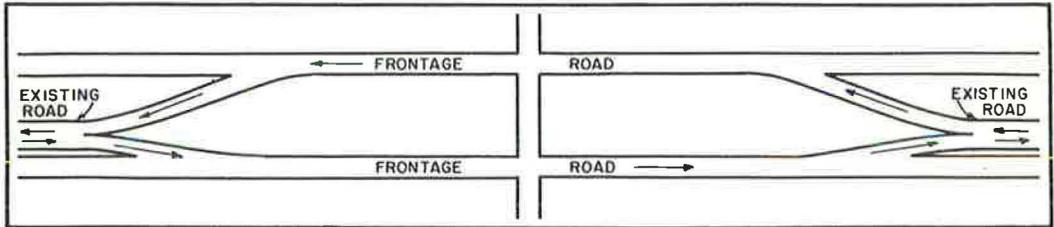
During the time the freeway is being constructed, the frontage road also serves a very important function. A possible sequence of the development is shown in Figure 5. Traffic remains on the existing road while the frontage roads are being constructed. Connections to adjacent property, etc., can be made during this stage. On completion of the frontage roads, the traffic can be rerouted to them and can operate with reasonable efficiency during the period in which the freeway lanes are being constructed. The



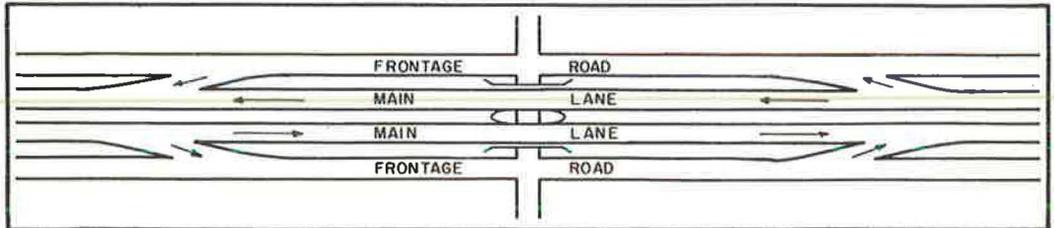
Proposed Freeway Location



Construct Frontage Roads



Re-route Traffic to Frontage Roads



Complete Freeway and Re-route Traffic

Figure 5. Stage development of freeway.

figure is an extremely simplified version of the development sequence that would probably take place in several additional steps but it illustrates the part that the frontage roads play in such a development. This procedure may not be necessary where a good street network is available for use as a detour, but even with some available streets this arrangement will eliminate the many problems arising from trying to use local or residential streets as detours for heavy arterial traffic during the period of freeway construction.

Design Life

During the design life or period of normal or design operation of a freeway, a frontage road system can add much to the overall operational efficiency of the facility. Some of the specific areas in which substantial benefits can be obtained from a freeway-frontage road design are presented.

Land Service.—Although a freeway is designed to accommodate long trip movements, there is no reason why such a facility should not give maximum land service to abutting properties during periods of off-peak flow. This maximum land service helps to realize more gains from the freeway investment and contributes greatly to the transportation needs of the urban area. During periods of peak traffic flow it may become desirable to give greater recognition to the long trip desires on the freeway with a resultant reduction in land service.

A freeway-frontage road system has the flexibility to accommodate both the situations previously enumerated. Figure 6 shows a typical freeway-frontage road system and the accessibility of all abutting properties. This increased land service insures maximum utilization of the freeway. If it is desirable to reduce some of the friction created by numerous entrance ramps during peak flow periods, the ramps may simply be closed and traffic diverted to the frontage road. This could be accomplished automatically with a surveillance system.

Another example of how the frontage road simplifies freeway operation is shown in Figure 7. This situation assumes that a fixed amount of development will exist in the vicinity of the freeway, with frontage either on a frontage road if one exists or on a crossroad. If the frontage develops along the crossroad as symbolized by areas C and D, the round trip path from Point X is shown by the heavy black line to Development C and the diagonal crossed line to Development D. Following these travel paths, it is evident that each requires one right turn, one left turn, and one through movement at a signalized intersection. This is compared with one straight-through movement each at a signalized intersection for vehicles bound to and from Developments A and B located along the frontage roads. Turns at the point of access have been neglected in each case but when considered tend to sway the situation ever farther toward the frontage road. This demonstrates that a given amount of development can actually be served more efficiently if it has access to the frontage road than if it is served by the crossroad.

Interchange Spacing.—In a recent article (4), D. W. Loutzenheiser focused attention on the problem of interchange spacing:

Proper interchange spacing is receiving much attention. Long distances between interchanges enhance the operation on the through lanes but they may accumulate more traffic than the cross facility is able to accommodate. Closely spaced interchanges provide a better distribution of through traffic and increase the level of local service but with consequent interference to through movements.

With continuous frontage roads and properly designed entrance and exit ramps, the interchange spacing problem can be greatly reduced. As previously discussed, maximum land service (to and from the freeway) can readily be accommodated. The interchange problem is then reduced to one of obtaining sufficient distance between interchanges to permit design of adequate ramps while still permitting arterial streets to cross the freeway occasionally to prevent a "barrier effect" to traffic flow on both sides of the freeway.



Figure 6. Typical freeway-frontage road system.

Figure 8 shows a design that might be considered. In this design, the interchange spacing is increased to permit the provision of numerous on and off ramps. This design permits maximum land service and eliminates much of the need for traffic moving to and from properties abutting on the frontage road to pass through a major interchange. During periods of peak flow, many of the entrance ramps could be closed to minimize influence on main lane movement. Thus, in effect, a freeway with variable interchange spacing could be created.

The maximum spacing between interchanges would be governed by the "barrier effect" created by the lack of through streets crossing the freeway. Considerable research is needed to investigate the extent of such a "barrier effect".

Frontage roads can also aid interchange operation by "spreading" the load, instead of concentrating it at a single interchange. It is possible to accommodate a great deal of interchange movement without vehicles passing through a freeway-major arterial interchange.

On-ramp operation is also benefited by continuous frontage roads as traffic desiring to enter the freeway can be "spread" over more than one entrance ramp. This smooths out the effect of "slugs" of traffic arriving from signalized intersections and permits traffic to keep flowing even though one entrance ramp may be loaded.

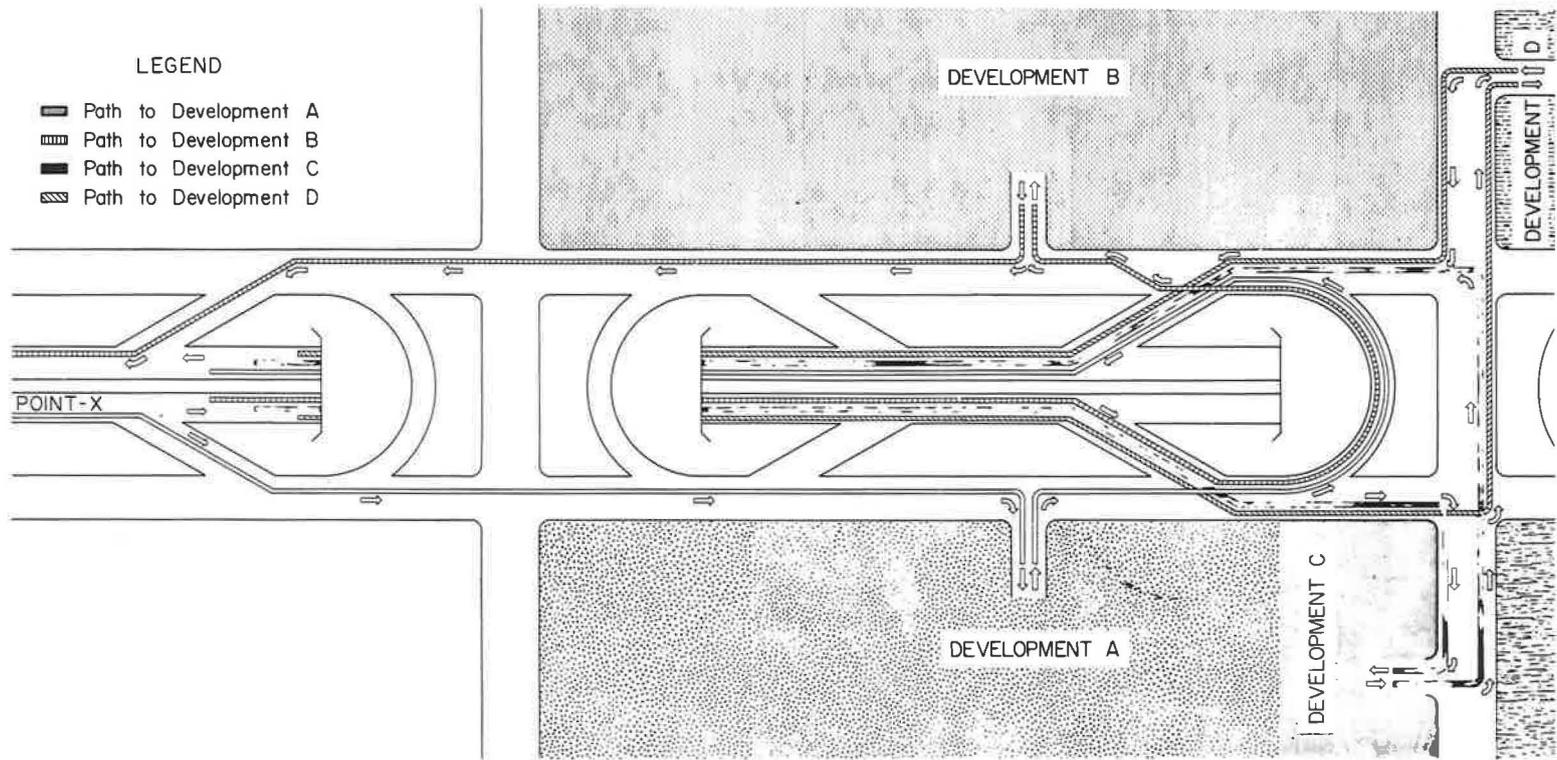


Figure 7. Access paths from Point X to various developments near freeway.

Surface Street Continuity.—One serious disadvantage of an urban freeway without frontage roads is the lack of continuity it creates with regard to the surface street system. This discontinuity is shown in Figure 9 where the "barrier effect" of a freeway without frontage roads is very evident.

With continuous frontage roads as shown in Figure 10, good continuity of the surface street system can be maintained. In fact, the frontage road becomes an extension of the surface system. This continuity is vital to a good traffic flow network. It also adds flexibility to the operation of the freeway as temporary ramp closures are possible without seriously affecting the flow of the system as diverted traffic may utilize the frontage roads.

Special Situations.—During the course of normal operation of a freeway, situations requiring special attention will develop. The following are examples of such situations:

1. When major maintenance is required on the main freeway lanes.
2. When a serious accident occurs on the main freeway lanes.
3. When oversize vehicles such as army transport trucks with high clearance requirements attempt to use the freeway.

In all three cases, a frontage road system provides the needed flexibility to cope with the situation. Figure 11 shows how traffic could be diverted to the frontage road to bypass a maintenance operation or a blocked freeway due to a serious accident. Also in the case of an accident, it is often vital to be able to reach the accident with emergency vehicles as soon as possible. If no frontage roads are available, this may be impossible due to the traffic jam that will probably occur on the main lanes of the freeway. Frontage roads, however, provide a means of quick and direct access to the accident scene. In the case of oversized vehicles with high clearance requirements, re-routing to avoid underpass structures may be required.

Saturated Operation

Many freeway facilities are becoming saturated much earlier than originally planned.

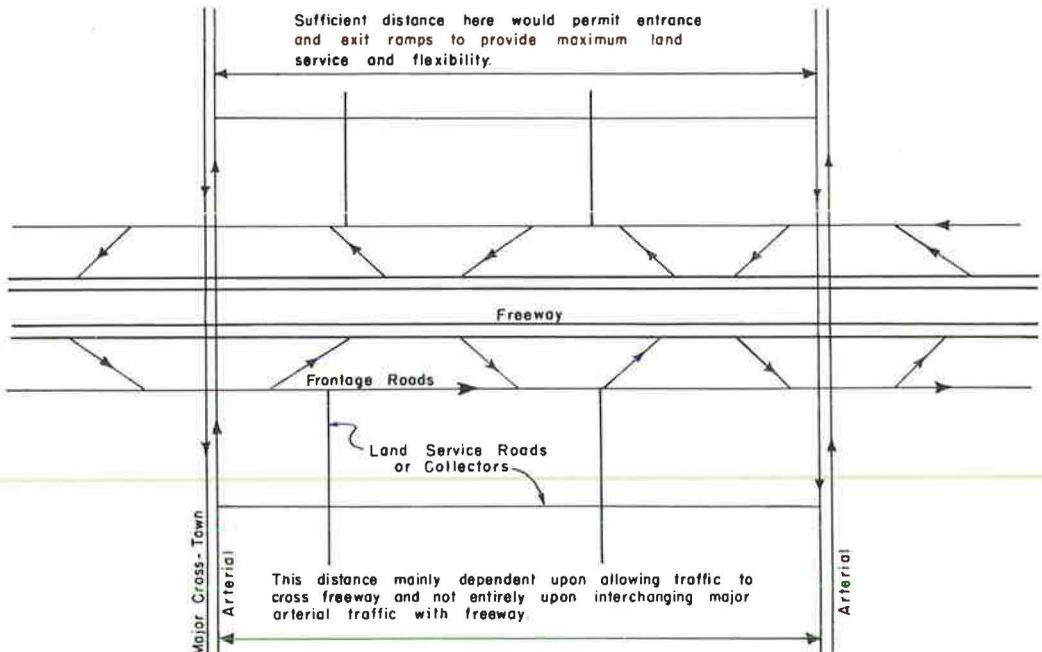


Figure 8. Interchange spacing.

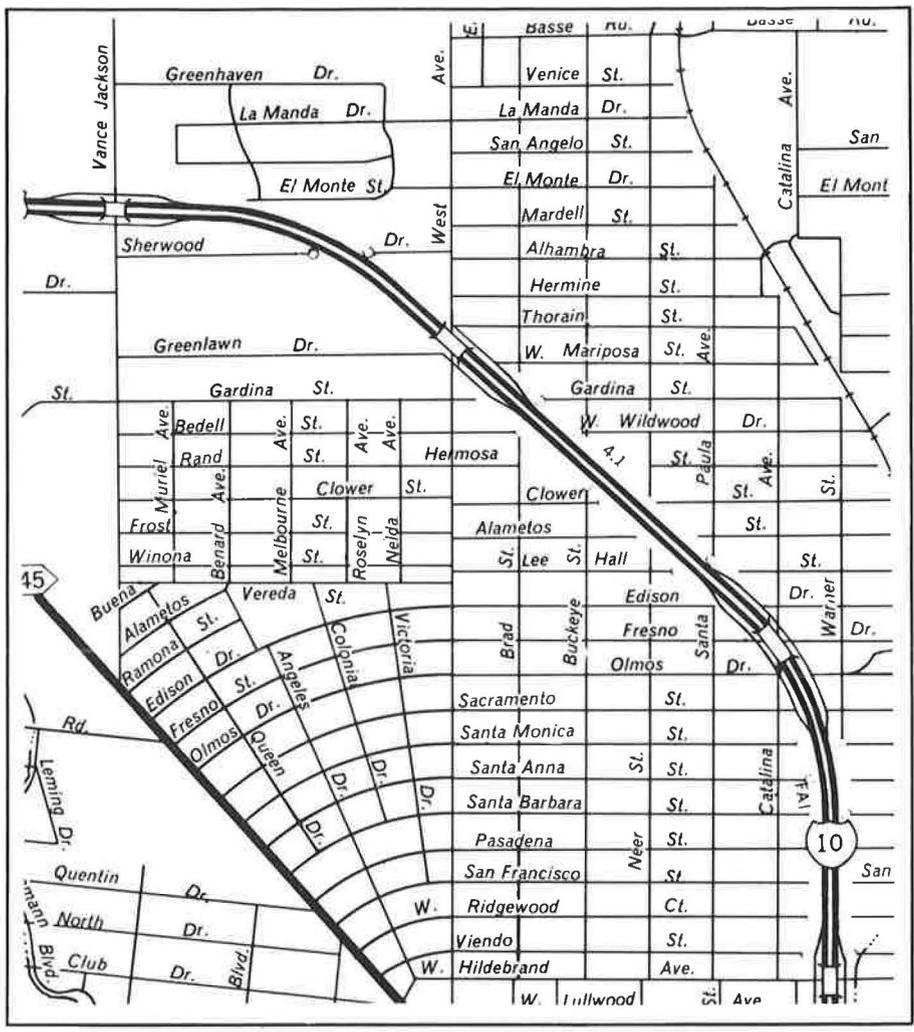


Figure 9. Freeway showing how street system is disrupted by lack of frontage roads.

This has been due mainly to incomplete freeway systems and to rapid increases in trip generation. The saturated flow conditions produce troublesome operation during peak periods of flow and result in facilities operating far below the level of efficiency for which they were designed.

The problem of peak-hour congestion has focused much attention on freeway surveillance and a great deal of worthwhile research is now being devoted to this subject. A question that must be considered, however, is what will be done to improve flow once a surveillance system indicates trouble developing on the freeway. A freeway-frontage road system offers a great deal of operational flexibility and lends itself well to a system of surveillance and control.

First of all, the frontage road provides major street continuity and permits the flexibility of closing off entrance ramps without completely blocking the desired movement of the city's at-grade arterial traffic. Second and probably most important, the frontage road provides additional capacity which can be utilized by the peak-hour freeway traffic. The inherent design of a freeway-frontage road system allows much flexibility of traffic control as various entrance and exit ramps can be opened or closed as the need occurs without seriously affecting the continuity of traffic flow. This condition was rec-

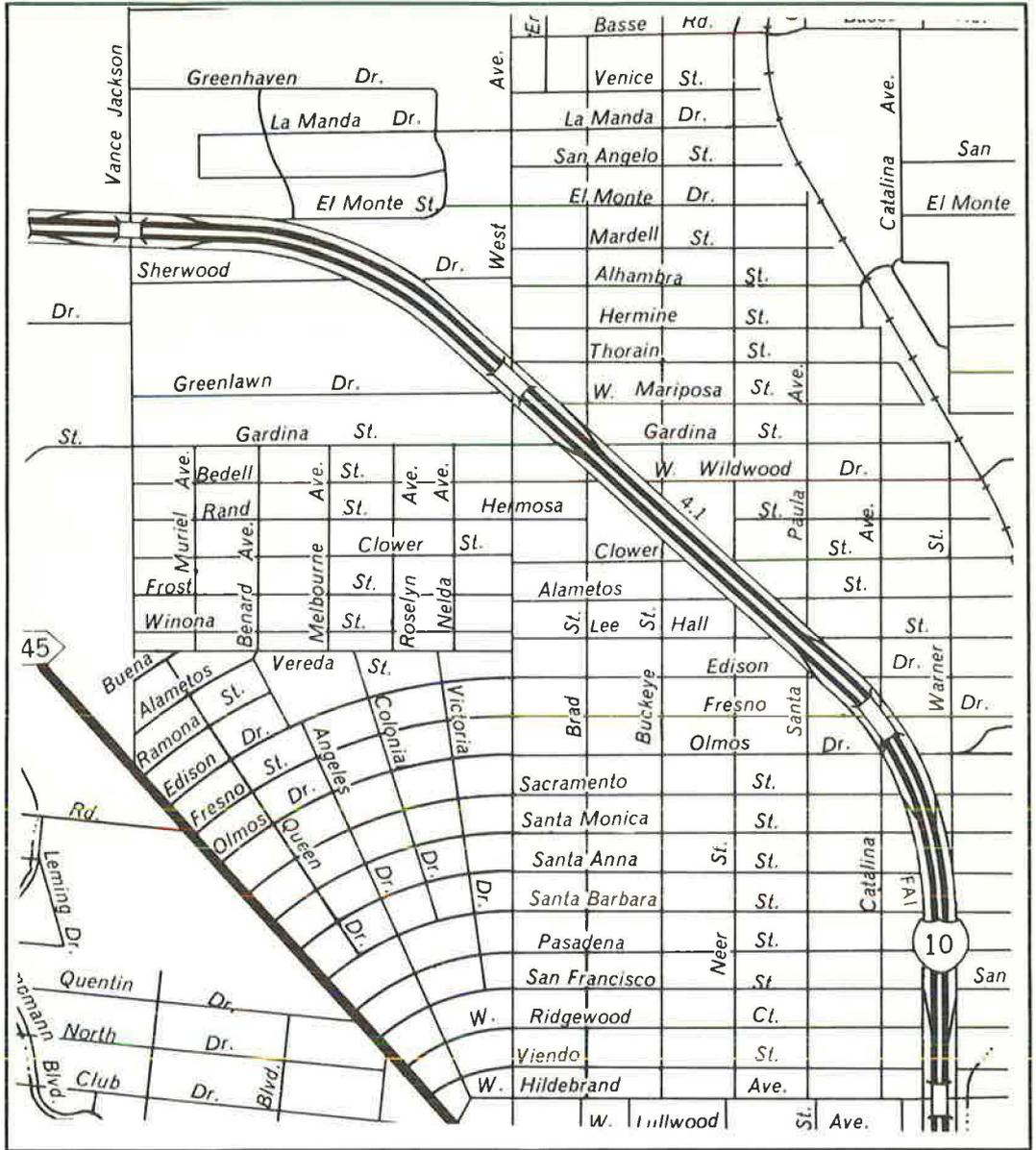


Figure 10. Freeway showing how street system continuity is maintained by frontage roads.

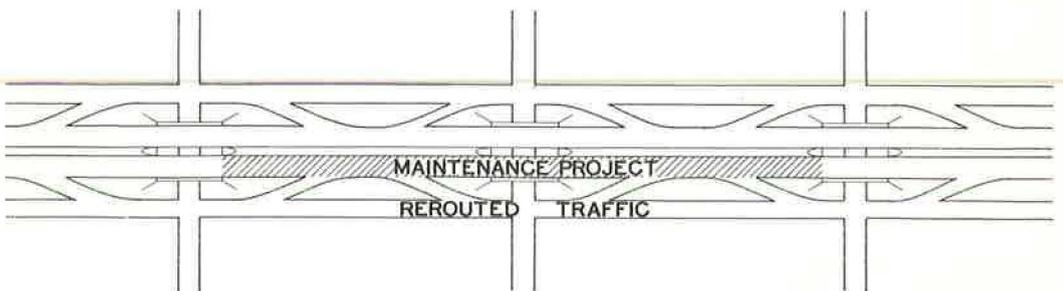


Figure 11. Traffic routed over frontage road during freeway maintenance.

TABLE 2
 FREEWAY AND FRONTAGE ROAD VOLUMES, GULF FREEWAY
 DIVERSION STUDY
 7:10 TO 8:10 AM

Study	Ramp	Inside	Middle	Outside	Total	Frontage Road	Grand Total
Houston II	Open	1,752	1,723	1,564	5,039	652	5,691
Houston III A	Closed	1,761	1,779	1,320	4,860	964	5,824
Houston III B	Closed	1,770	1,786	1,279	4,835	937	5,772

ognized by the AASHO Special Freeway Study and Analysis Committee in their report to the Executive Committee (5).

The feasibility of diverting peak-hour traffic to a frontage road by closing entrance ramps was studied by the Texas Transportation Institute on the Gulf Freeway in Houston, Texas (6). Figure 12 shows a section of the Gulf Freeway to which the inbound entrance ramps were closed during a morning peak period of traffic flow. Traffic which normally used these ramps was diverted to the frontage road.

Before- and-after studies were conducted at Point A in Figure 12 through the use of a 16-mm motion picture camera. Table 2 gives results of the volume studies. The diversion resulted in a slight decrease in freeway traffic but an overall gain in traffic moved.

Of more significance, the operating conditions on the freeway were greatly improved. The average 5-min speed during the before study was 23 mph as compared to 36 mph during the after study. Figure 13 shows speed volume relationship comparisons. Travel time studies on both routes (freeway vs frontage road) indicated that the diverted traffic was not subjected to any significant increase in travel time.

The diversion study previously discussed, provided data to indicate that a frontage road could be used during periods of saturated flow to provide added capacity and to improve overall operating condition. Thus, the continuous frontage road is a vital element in the development of a surveillance and control system and provides the required flexibility and added capacity to permit control of freeway and major arterial traffic.

INTERCHANGE CONSIDERATIONS

One major criticism of the use of continuous frontage roads has been that they tend to create problems at points of interchange between the freeway and major at-

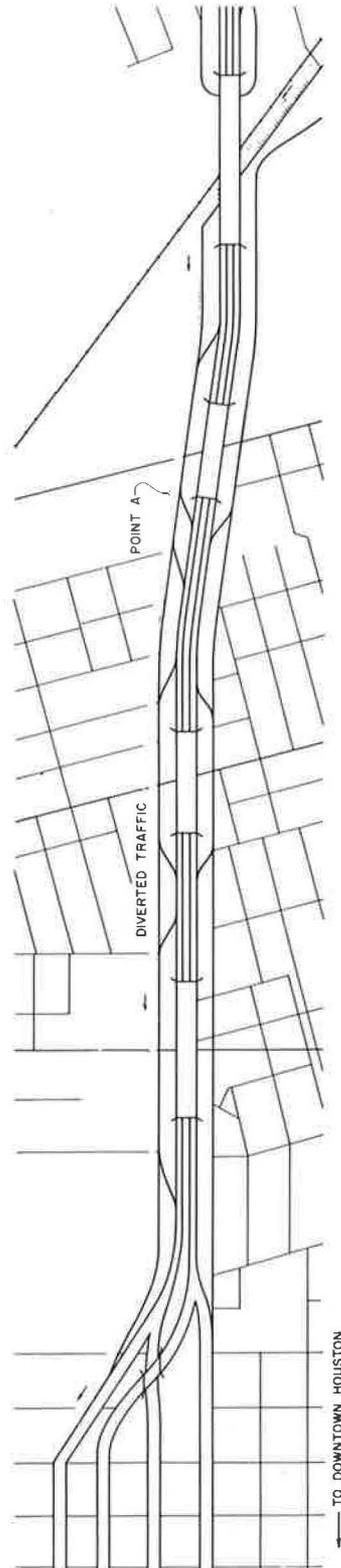


Figure 12. Gulf Freeway traffic diversion.

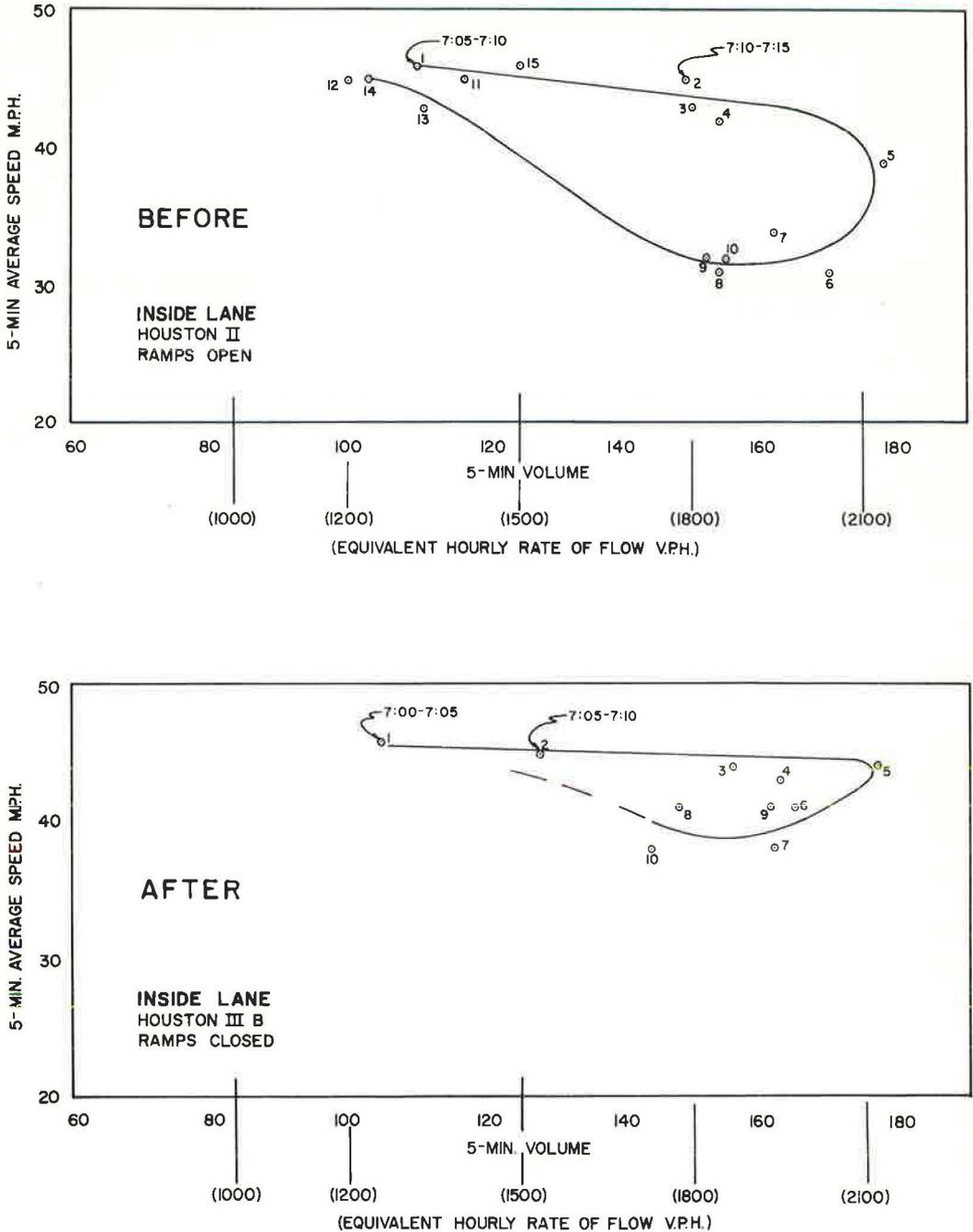


Figure 13. Speed-volume relationships, closed ramp study, Gulf Freeway.

grade arterials. The interchange usually used in conjunction with frontage roads is the diamond interchange and until recently it was felt that this type of interchange had an inadequate capacity to accommodate the movements required.

Recent studies have shown, however, that the diamond interchange when properly designed and signalized has a tremendous potential for the movement of traffic. The two separate intersections created by the intersection of the one-way frontage roads and



Figure 14. Conventional two-level diamond interchange.

the two-way major arterial permit an overlap of signal phases when the intersection is signalized by a single controller which actually increases the capacity of the intersection area over that of a single intersection with two-way approaches.

A recent report (7) develops the relationship between intersection capacity and signal phase overlap, and it discusses in more detail the development of capacity equations used in this report. Two significant findings reported regarding intersection signalization and capacity are as follows:

1. Intersection capacity can be significantly increased by the use of signal phase overlaps.
2. If phase overlaps greater than 18 sec per cycle can be obtained, maximum capacity can be obtained with cycle lengths of 50 to 70 sec.

Variations of Diamond Interchange

The diamond interchange permits a wide range of design flexibility in that several variations of a diamond interchange can be used. The three basic variations of a diamond interchange are (a) conventional two-level, (b) split two-level, and (c) three-level. The capacity of the diamond interchange depends on the type selected.

Conventional Two-Level.—The conventional two-level diamond interchange shown in Figure 14 is the most widely used variation of the diamond interchange. The signal phasing shown in Figure 15 permits a signal phase overlap and yields a basic capacity relationship:

$$N_C = 1,714 - \left(\frac{18.4}{2.1}\right) \left(\frac{3,600}{C}\right) + \left(\frac{\phi}{2.1}\right) \left(\frac{3,600}{C}\right) \quad (1)$$

in which

- N_C = basic capacity (capacity considering only one approach lane per phase)
in vehicles per hour;
 C = cycle length in seconds; and
 ϕ = total signal phase overlap time in seconds.

Eq. 1 can be explained by considering each individual term. The first term (1,714) represents the number of vehicles that can be moved from a single approach lane per hour at an average time spacing of one vehicle every 2.1 sec. The second term represents the number of vehicles "lost" per hour due to starting delays and amber time, and the final term represents the number of vehicles "gained" per hour due to signal phase overlap. Thus, an ideal volume (1,714) is adjusted to the actual volume (N_C).

When $C = 50$ sec and $\phi = 20$ sec, the basic capacity of the signalized intersections of a conventional two-level diamond is as follows:

$$\begin{aligned} N_C &= 1,714 - \left(\frac{18.4}{2.1}\right) \left(\frac{3,600}{50}\right) + \left(\frac{20}{2.1}\right) \left(\frac{3,600}{50}\right) \\ &= 1,714 - 630 + 680 \\ &= 1,764 \text{ vph} \end{aligned}$$

For an interchange with three lanes per approach, the total hourly capacity of the intersection area (N_T) is $N_T = 3(1,764) = 5,292$ vph. This total capacity of 5,292 vehicles per hour can be apportioned to the various approaches on the basis of percent green time available for each phase (or approach). A specific example of possible volumes that could be handled is shown in Figure 16.

Split Diamond.—If more capacity than that provided by the conventional diamond is required, a split diamond design can be used. This design as shown in Figure 17 is obtained by dividing (splitting) the major arterial into two one-way streets at its intersection with the freeway-frontage road system. The major arterial could be a one-way pair or could be transitioned back to a two-way street a short distance from the freeway.

The main advantage of this design is the added intersectional capacity that can be

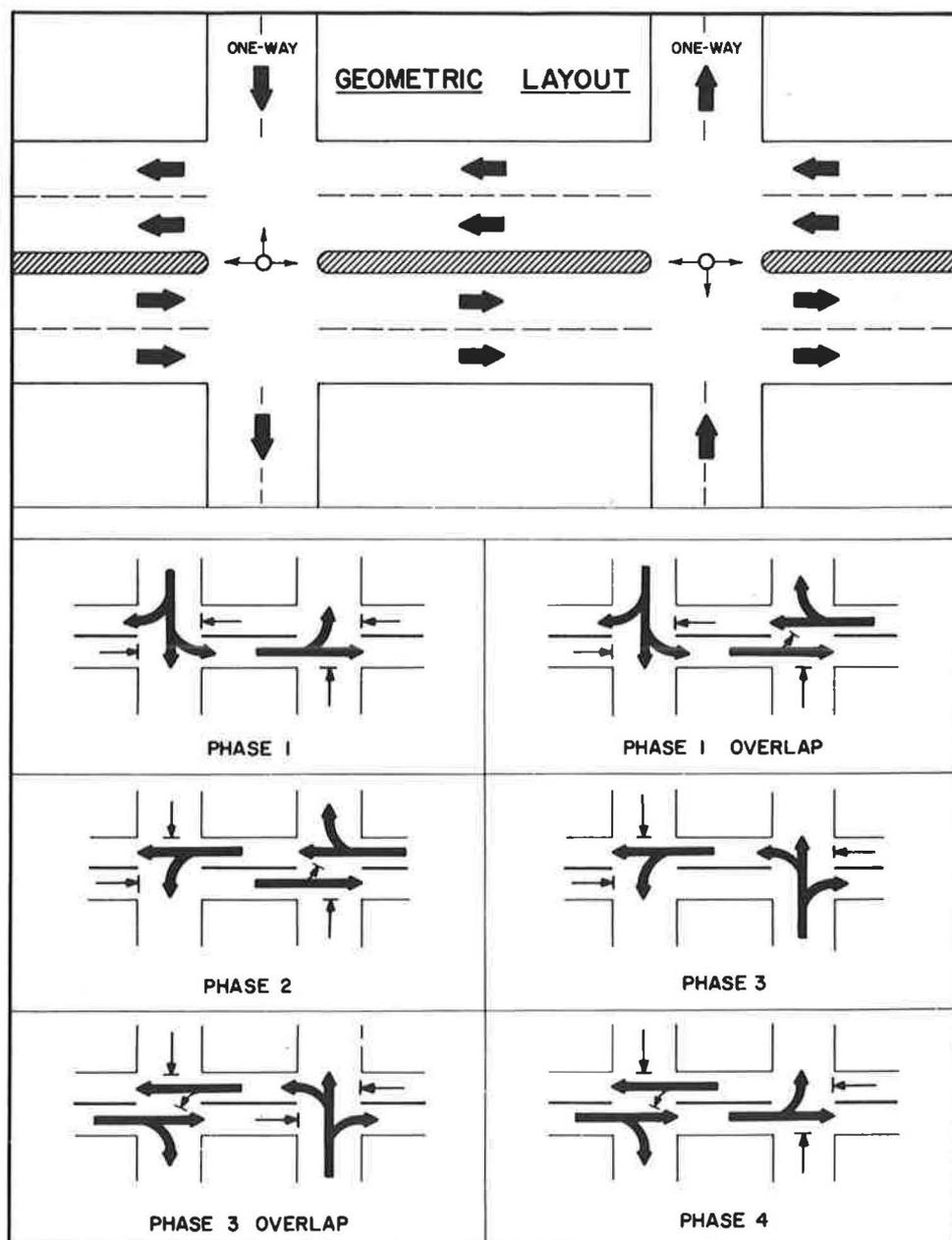
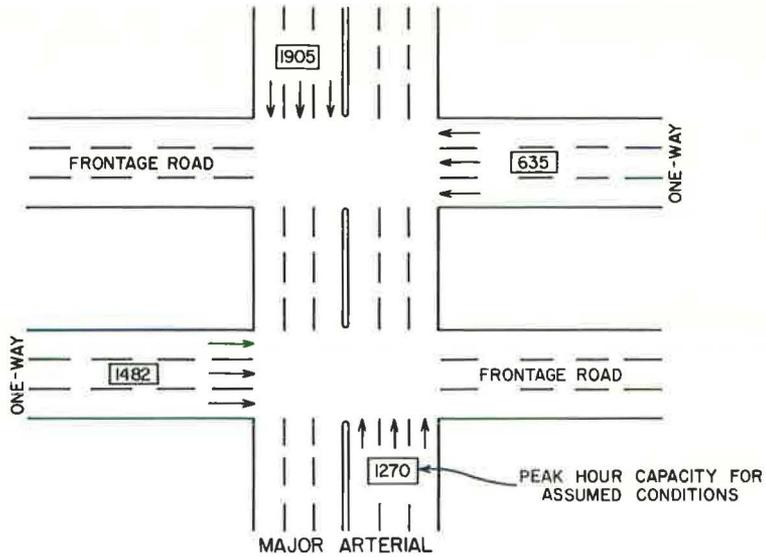


Figure 15. Signal phasing, conventional two-level diamond interchange.

obtained due to the overlap signal phasing which is possible. Figure 18 shows the desired phasing arrangement from which it can be observed that four phase overlaps are now possible compared to only two with the conventional two-level diamond.

The basic capacity equation (Eq. 1) will still hold for this interchange with the exception that a longer cycle length will be required and that ϕ will be increased. The longer cycle length is required to assure ample overlap time on each phase, and ϕ increases due to the increased number of overlaps. If $\phi = 40$ sec (10 sec per phase) and $C = 80$ sec, then $N_c = 1,714 - 390 + 860 = 2,184$ vph.



	PEAK HOUR CAPACITY	POSSIBLE ADT VOLUME
MAJOR ARTERIAL	3175	31,750
FRONTAGE ROAD	2117	21,170
CRITICAL CAPACITY	5292	52,920

ASSUMPTIONS

MAJOR ARTERIAL TRAFFIC IS 60% OF TOTAL INTERCHANGE TRAFFIC AND IS SPLIT 60-40.

FRONTAGE ROAD TRAFFIC IS 40% OF TOTAL INTERCHANGE TRAFFIC AND IS SPLIT 70-30.

PEAK HOUR VOLUME - 10% ADT

Figure 16. Interchange capacity, conventional two-level diamond interchange.

Again considering an interchange with three lanes per approach, the total peak-hour capacity of the split diamond is $N_T = 3(2,184) = 6,552$ vph, which is an increase in capacity of 1,260 vph as compared with the conventional two-level diamond.

Three-Level Diamond. -If neither the conventional two-level nor the split diamond interchange provides the desired capacity, a third variation termed a three-level diamond can be used. This interchange is shown in Figure 19. This interchange carries the major arterial traffic over or under the freeway on a separate (third) level. The interchange movements between the major arterial and the freeway (or vice versa) are accommodated through an intersection area that can be designed and operated exactly like the intersection area of a split diamond (Figure 17).

The intersection capacity of the split diamond (as previously developed) is therefore available to accommodate the interchange movements. The through traffic on the major arterial and freeway travel on separate levels and enjoy uninterrupted flow capacity. Heavy left-turning movements can be handled by the use of dual turn lanes. Heavy right-turn movements can use separate right-turn lanes or loops. Thus, the capacity of a three-level diamond can approach that of a directional interchange.

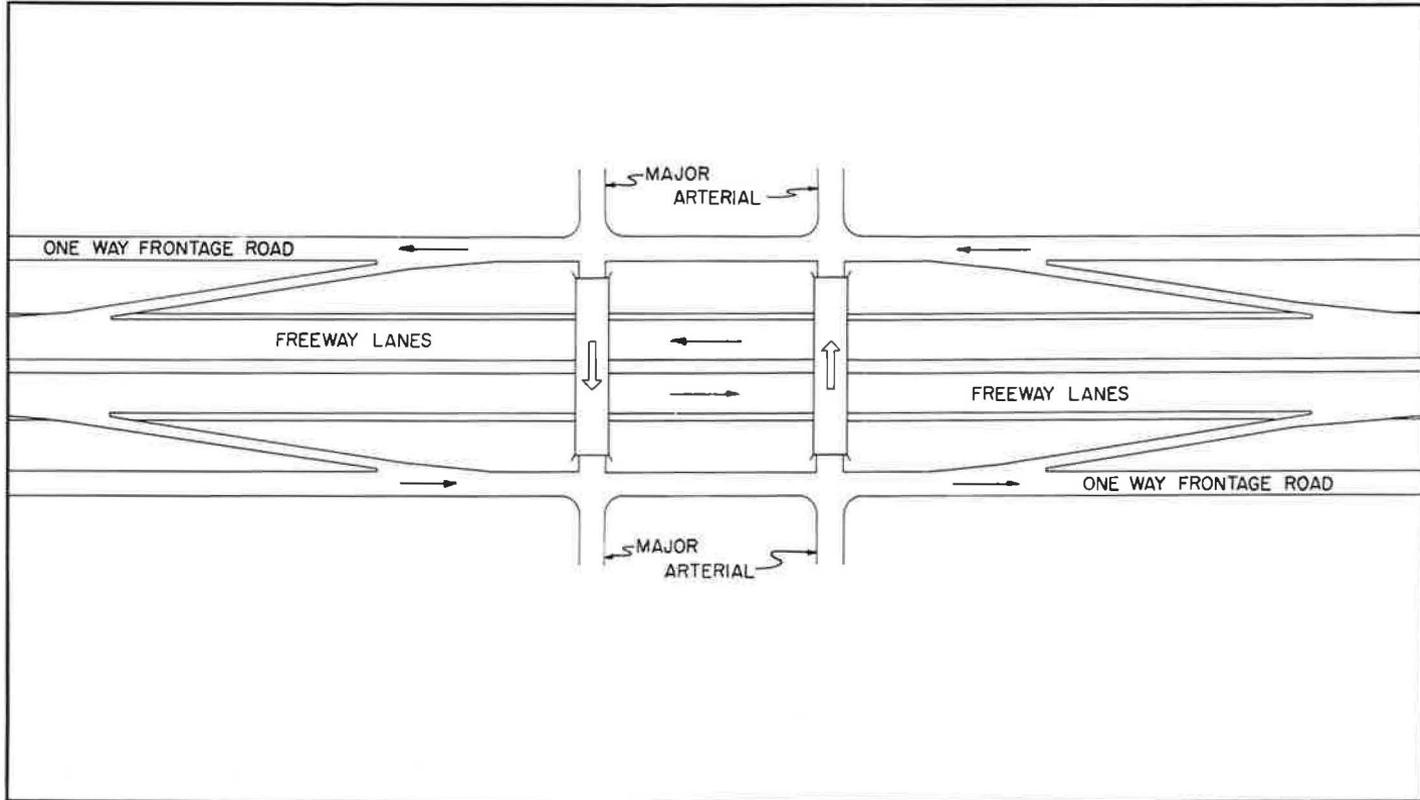


Figure 17. Split diamond interchange.

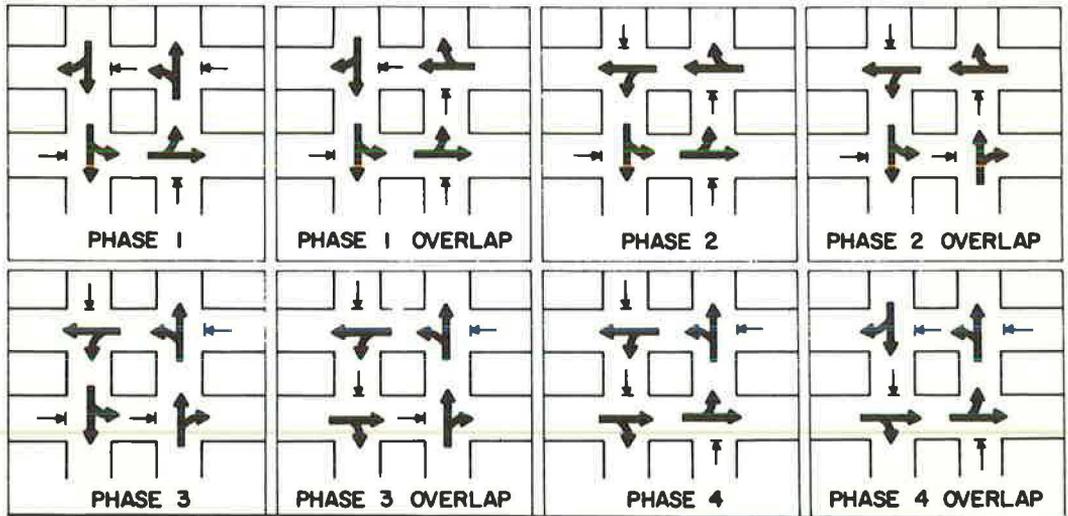
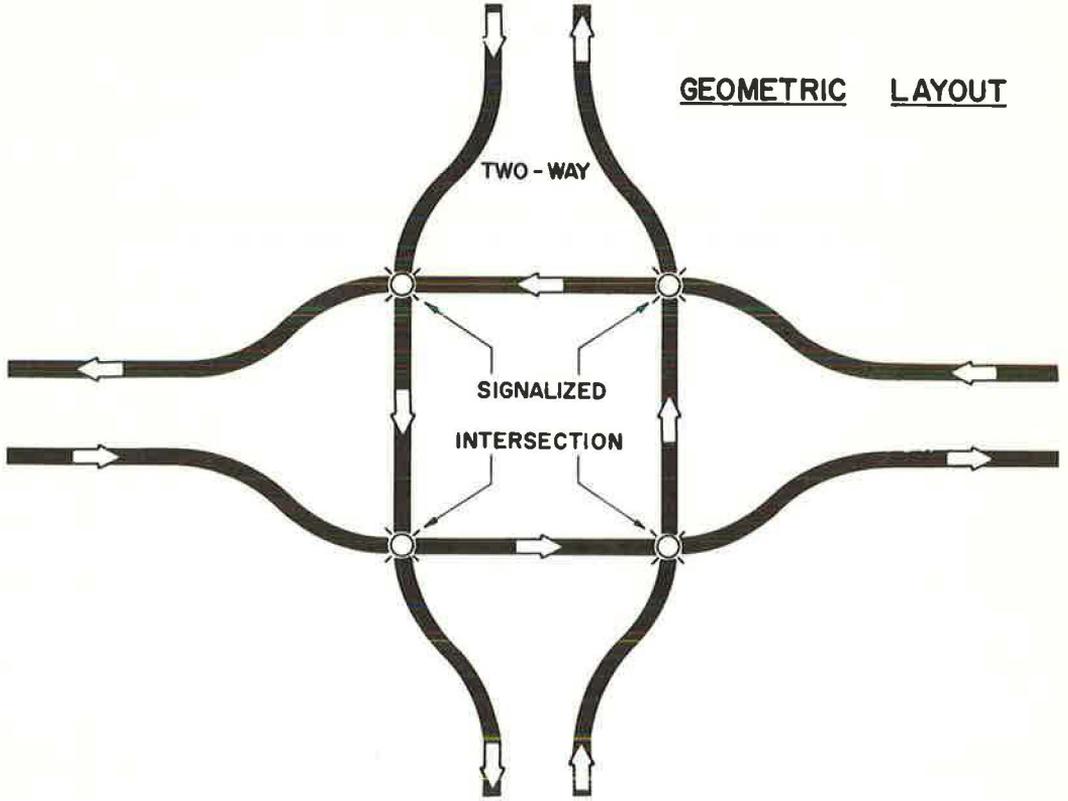


Figure 18. Signal phasing, split or three-level diamond interchange.

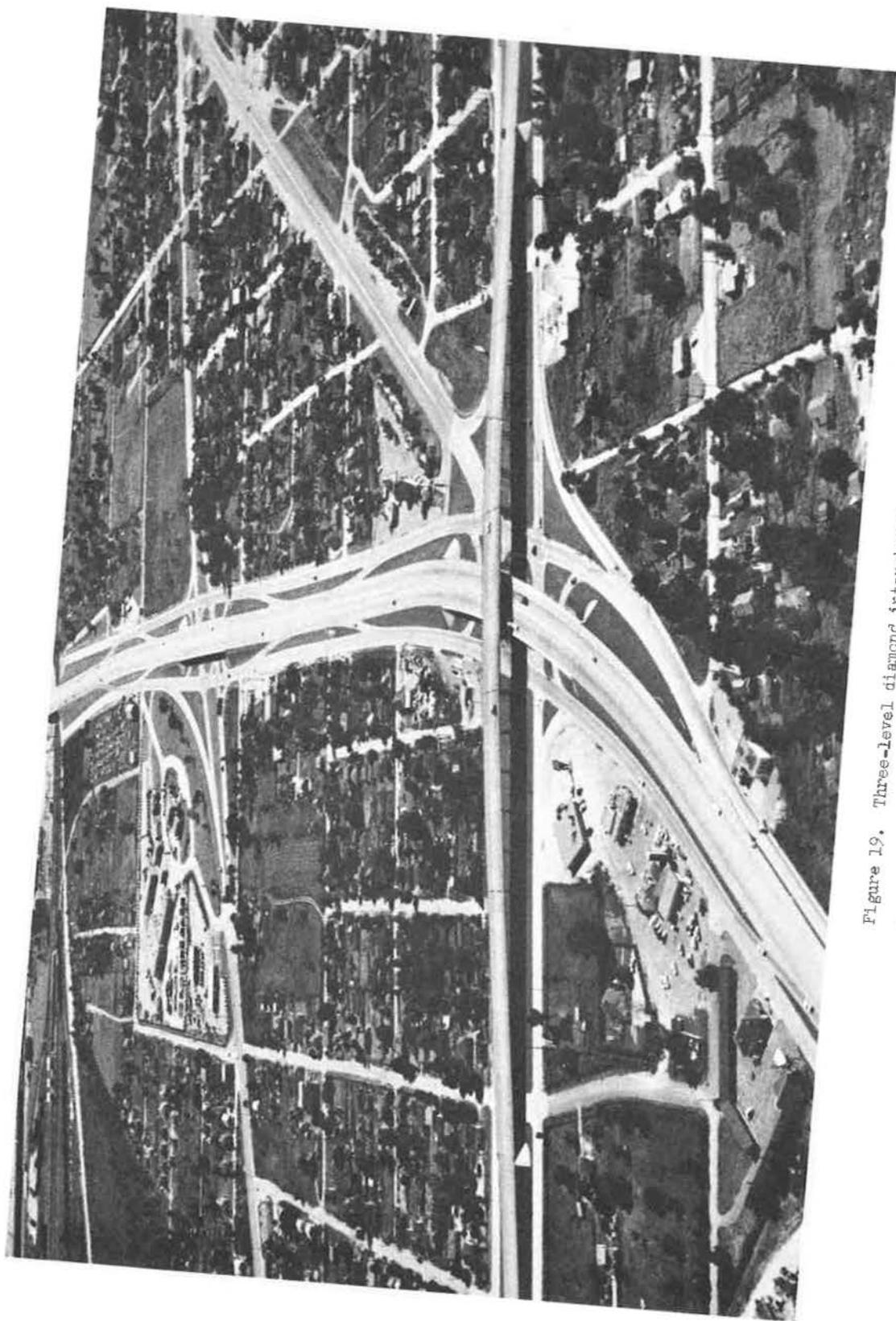


Figure 19. Three-level diamond interchange.

SUMMARY AND CONCLUSIONS

This paper has evaluated the use of frontage roads as an element of urban freeway design. The following conclusions are presented in summary of the evaluation:

1. In view of the effect that the presence or absence of frontage roads has on the cost of right-of-way, the construction of a frontage road system will usually result in a lesser overall cost for the entire facility.

2. The presence of a frontage road adjacent to a freeway tends to equalize the effect of the freeway on adjacent property, to minimize difficulties in right-of-way acquisition, and to distribute the benefits of the freeway more evenly.

3. The frontage road is more than an appurtenance to serve adjacent property. It is a multifunctional, integral part of the overall street system and is beneficial to the operation of the system in many ways.

4. A continuous frontage road system is an asset to a feasible and operationally efficient means of using a stage development program for the construction of urban freeways.

5. A continuous frontage road system provides a means of handling traffic flow during the construction of the main freeway lanes.

6. During the design life of a freeway facility, a continuous frontage road system provides maximum land service to properties abutting the freeway. It greatly increases the flexibility of the interchange system and provides surface street continuity. In addition, such a system provides operational flexibility to handle special traffic situations which may develop.

7. When a freeway reaches a saturated flow condition, a frontage road system can provide the operational flexibility required to operate a system of freeway surveillance and control.

8. The design of a freeway-frontage road system presents no special design problems, and interchange movements can usually be handled with some variation of the relatively inexpensive diamond interchange.

In the past, the frontage road has usually been viewed only as a means of controlling access, and little attention has been focused on the operational advantages such a facility offers. It is hoped that this paper will serve to point up the advantages of a continuous frontage road and to stimulate the freeway designer to give serious consideration to this facility as an essential design element of future urban freeway systems.

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Use and Design of Acceleration and Deceleration Lanes in Indiana

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The speed and lateral placement of vehicles on the various designs of the acceleration and deceleration lanes of the Indiana Toll Road and the Interstate System of the State of Indiana were studied to correlate the acceleration and deceleration lane design with traffic behavior and driver requirements and to determine which acceleration and deceleration lane designs provide the most efficient and safest operation. Data on speeds and lateral placement of traffic using the acceleration and deceleration lanes were obtained by use of a motion picture technique. Spot speeds of through lane traffic were measured using an electromatic radar speed meter.

The study revealed that a large number of the driving public do not know how to use acceleration and deceleration lanes properly. Acceleration or deceleration lane traffic was found to have little effect on the speed of the through lanes before diverging into the deceleration lanes. The long direct taper type of design was found superior for both acceleration and deceleration lanes.

• ON THE BASIS of various recent research studies, it is apparent that highway designers are becoming more and more concerned with the relationship between highway design and traffic behavior. An excellent example of current interest is the design of acceleration and deceleration lanes. To obtain maximum efficiency and safety in the operation of acceleration and deceleration lanes, and to maintain efficiency on the main facility, it is necessary to relate the design of such lanes to traffic behavior, as indicated by the requirements and desires of drivers.

Drivers leaving a highway at an intersection are usually required to reduce speed before turning. On the other hand, drivers entering a highway at an intersection have to accelerate to reach the desired speed of the traffic on the facility. Whenever this deceleration or acceleration by exiting or entering traffic takes place on the main traveled way of the highway, it disrupts the flow of through traffic and is hazardous (1). Thus, to minimize these undesirable aspects on high-speed facilities, maximum use should be made of acceleration and deceleration lanes.

For best operating conditions, acceleration and deceleration lanes should be used for the entire acceleration and deceleration phase by vehicles entering or leaving the through traffic lane. Each such acceleration and deceleration lane, therefore, should be of a design that will enable a driver to maneuver his vehicle onto it without a major change in speed and, once on it, to make the necessary change between the speed of operation on the main facility and the lower speed required for exit or entrance. The optimum condition of operation for an acceleration lane is to have acceleration lane traffic accelerate on the acceleration lane and merge into the through lane traffic at approximately the same direction of travel and at the same speed as the through lane traffic. That for a deceleration lane is to have deceleration lane traffic leave the

through lane at about the same direction of travel as through lane traffic and at the same speed, with all deceleration taking place in the deceleration lane.

Acceleration and deceleration lane design varies very significantly from State to State. Some States follow the standards set forth in the AASHO manual (1); others have developed their own standards using the AASHO manual as a guide.

The Indiana State Highway Commission has used several designs for acceleration and deceleration lanes. The parallel lane type of acceleration lane was initially adopted as a standard. This type, consisting of a full width lane 350 ft long with 400 ft of taper, has been constructed at several locations. Later, the length of the parallel acceleration lane was changed to 250 ft of full width with 250 ft of taper. Later yet, the direct taper type of acceleration lane design was adopted to correct a tendency for entering traffic to move into the through lane too quickly. This latter acceleration lane has a 750-ft taper. More recently, an acceleration lane having 50:1 taper was adopted. This design is similar to one described in "Traffic Behavior on Freeways," Highway Research Board Bulletin 235. This last design, however, had not been incorporated in any construction completed before 1962.

Adopted standards for the State of Indiana for deceleration lanes have also included the parallel lane type and the direct taper type. One design uses a taper lane width ranging from 0 to 12 ft in 250 ft, followed by 50 ft of tangent and then a curve. A second design uses a continuous curve. A third design has a taper followed by a curve, and a fourth design uses a straight short taper. During 1961, a design having about 900 ft of straight taper was adopted.

This paper reports the speed and lateral placement of vehicles on acceleration and deceleration lanes of these various designs as constructed in Indiana. It also correlates acceleration and deceleration lane design with traffic behavior and apparent driver requirements and makes some suggestions relative to acceleration lane and deceleration lane design.

SCOPE

The study was limited to high-type facilities in Indiana. Locations were chosen on the Indiana Toll Road and those sections of the Interstate System that were completed and had been opened to traffic before 1962. The study locations were specifically selected to provide data on speeds and lateral placement for various designs of acceleration and deceleration lanes and for operation under different conditions. Not all the different designs of acceleration and deceleration lanes which have been adopted by the Indiana State Highway Commission could be studied, as some of these designs had not as yet been incorporated in any completed construction.

Locations were chosen, if possible, to include one of three conditions of road geometry: where the acceleration or deceleration lanes met or left the through lanes on (a) a tangent, (b) a right curve, or (c) a left curve. One location was also studied where an acceleration lane joined the through lane on the upgrade portion of a crest vertical curve, and another was included where the junction occurred on the downgrade portion of a sag vertical curve. All these conditions were studied to evaluate what effects each had on traffic behavior and to determine which condition provided for the most efficient and safe use of acceleration and deceleration lanes.

In four cases, more than one location having the same acceleration or deceleration lane design and similar conditions of road geometry were studied. These studies were made at different locations having different traffic, in order to evaluate the repetitive character of the results. It was suspected that other factors such as traffic volumes and types of drivers using the facility would have a significant bearing on the results.

The speeds of through vehicles were also analyzed at each interchange around the point where the acceleration or deceleration lanes joined the through lanes. This was done to compare speeds of acceleration or deceleration lane traffic with through lane traffic at each location. The speeds of through traffic were also obtained before or after the interchanges to evaluate the effect of interchange traffic on the speed of through traffic at the interchange.

The types of vehicles studied were passenger cars and light trucks, which were considered as passenger cars. Data from at least 108 passenger cars were analyzed for

each acceleration or deceleration lane studied. Heavy trucks were not included, as the number of such trucks using acceleration and deceleration lanes on existing Indiana freeways was small and a long and expensive period of data collection to obtain an adequate sample of trucks would have been required. Information on the use of acceleration and deceleration lanes by trucks is certainly desirable and should be obtained. During the course of the study the location where deceleration lane traffic begins to decelerate while on the through lanes and before the beginning of the deceleration lane was found of interest. This information, therefore, was obtained for a few of the study locations for deceleration lanes. Only information for free-moving vehicles was used for all parts of this study.

Vehicles that stopped on the acceleration or deceleration lanes were not included in the speed and lateral placement evaluations but were recorded for possible use as a measure of the efficiency of operation of the various designs. Data on accidents at each site were obtained but the number of accidents was so small because so little time had elapsed since construction that an analysis was not possible.

PROCEDURE

Data on speeds and lateral placement of traffic using acceleration and deceleration lanes were obtained with a 16-mm motion picture camera. The motion picture type of study was selected, after consideration of various other methods, as being the best for detailed study and analysis.

The filming was done from a vantage point, usually an overpass over the main facility at the interchange. The camera used was a 16-mm Bell and Howell movie camera with a built-in turret head.

In the case of acceleration lanes, movies were taken of each free-flowing passenger car as it passed a designated point, usually the nose, beyond which it was able to maneuver and merge into the through lanes, and until the left rear wheel of the vehicle crossed from the acceleration lane onto the through lane. The left rear wheel of the car was chosen as the criterion because acceleration lane traffic traveled away from the camera and therefore was visible, and for all practical purposes the instant the left rear wheel crosses from the acceleration lane onto the through lane, conflict with the through traffic operation becomes serious.

As for the deceleration lane, movies were taken of each vehicle before its left front wheel crossed from the through lane onto the deceleration lane and until it passed a designated point on the deceleration lane, usually the nose, beyond which the car was unable to maneuver back onto the through lane. Here, the left front wheel of the car was chosen because, for deceleration lanes, traffic traveled towards the camera and by that point most of the car had left the through lane and was no longer a conflict on the through lanes.

The film was analyzed by projecting it through a time-motion study projector. A grid was superimposed on the screen where the film was projected to a scale which provided accurate ground measurements. The vehicle speeds at various locations on the acceleration and deceleration lanes were computed using the grid system by determining the distance traveled during a time period. Time was measured by the number of frames a vehicle traveled between two grid lines. The lateral placement of the vehicles on the acceleration or deceleration lanes was also traced by using the grid system. The stations at which the left rear wheel of acceleration lane traffic and the left front wheel of deceleration lane traffic crossed the line between the acceleration or deceleration lane and through lanes were noted.

Speed information was grouped by 100-ft sections according to the location on the acceleration or deceleration lane where each car merged or diverged. For example, the speed data for all vehicles on a study lane which merged between Station 0+00 and Station 1+00 were grouped for analysis. The average speed, 85th percentile speed, standard deviation, and percent of total vehicles leaving or entering between stations were computed for each group. The 85th percentile speed and percent leaving or entering between 100-ft stations are indicated on the sketches of each study site which are included in this report. A single value for the 85th percentile speed for all vehicles entering or leaving the deceleration or acceleration lane is shown on these sketches in parentheses.

A cumulative frequency graph of lateral placements was plotted for each study site. This graph was developed by plotting the cumulative percent of vehicles that merged into or diverged from the through lane against the distance from the nose at which the merger or divergence occurred. It was found that the cumulative curves thus prepared tended to break sharply at approximately 90 percent. Thus, the 90th percentile was taken as an important criterion for determining the length of an acceleration or deceleration lane which would be effectively used.

Spot speeds of through lane traffic were measured using a concealed radar meter. The spot speeds of through traffic were measured at two locations at each study site. The first of these was "within the area of conflict," the area where acceleration or deceleration lanes adjoined the through lane. The second spot speed location was "beyond the area of conflict," a point approximately 1 mi before or after the interchange. Posted speed limits were the same for traffic on all through lanes studied.

Spot speeds of deceleration lane traffic were measured at three deceleration lane sites in an attempt to determine where traffic destined for a deceleration lane begins to decelerate on the through lane. The radar meter was placed so that the cars approaching the deceleration lane intersected the beam at the specified location, but only cars that proceeded to use the deceleration lane were recorded.

RESULTS

Acceleration Lanes

Figure 1 is a map of Indiana showing the location of the acceleration and deceleration lanes studied. Pertinent data for each acceleration lane location are given in Table 1.

TABLE 1
STUDY LOCATION DATA—ACCELERATION LANES

Accel. Lane Identification	Location	Direction of Accel. Lane Traffic	Geometric Condition		Acceleration Lane	
			Description	Type	Description	Type
A1a	Indiana Toll Road—Middlebury	Westbound	Tangent	1	Direct taper 1,200 ft	1
A1b	Gary West	Westbound	Tangent	1	Direct taper 1,200 ft	1
A1c	Michigan City	Westbound	Tangent and down-grade portion of sag vertical curve	4	Direct taper 1,200 ft	1
A1d	Chesterton—Valparaiso	Westbound	Right curve	2	Direct taper 1,200 ft	1
A1e	LaPorte	Westbound	Right curve and up-grade portion of crest vertical curve	5	Direct taper 1,200 ft	1
A1f	Gary East	Eastbound	Left curve	3	Direct taper 1,200 ft	1
A2a	Interstate 65—State Road 39	Northbound	Tangent	1	Parallel— 350-ft str and 400-ft taper	2
A2b	Tri-State—Kennedy Ave.	Eastbound	Tangent	1	Parallel— 350-ft str and 400-ft taper	2
A2c	Interstate 74—Post Road	Eastbound	Tangent	1	Parallel— 350-ft str and 400-ft taper	2
A2d	Interstate 65—State Road 60	Southbound	Right curve	2	Parallel— 350-ft str and 400-ft taper	2
A2e	State Road 39	Southbound	Left curve	3	Parallel— 350-ft str and 400-ft taper	2
A3a	State Road 56	Northbound	Tangent	1	Parallel— 250-ft str and 250-ft taper	3
A3b	State Road 334	Northbound	Left curve	3	Parallel— 250-ft str and 250-ft taper	3
A4a	Tri-State—Calumet Ave.	Eastbound	Tangent	1	Direct taper 300 ft	4
A4b	Calumet Ave.	Westbound	Right curve	2	Direct taper 300 ft	4

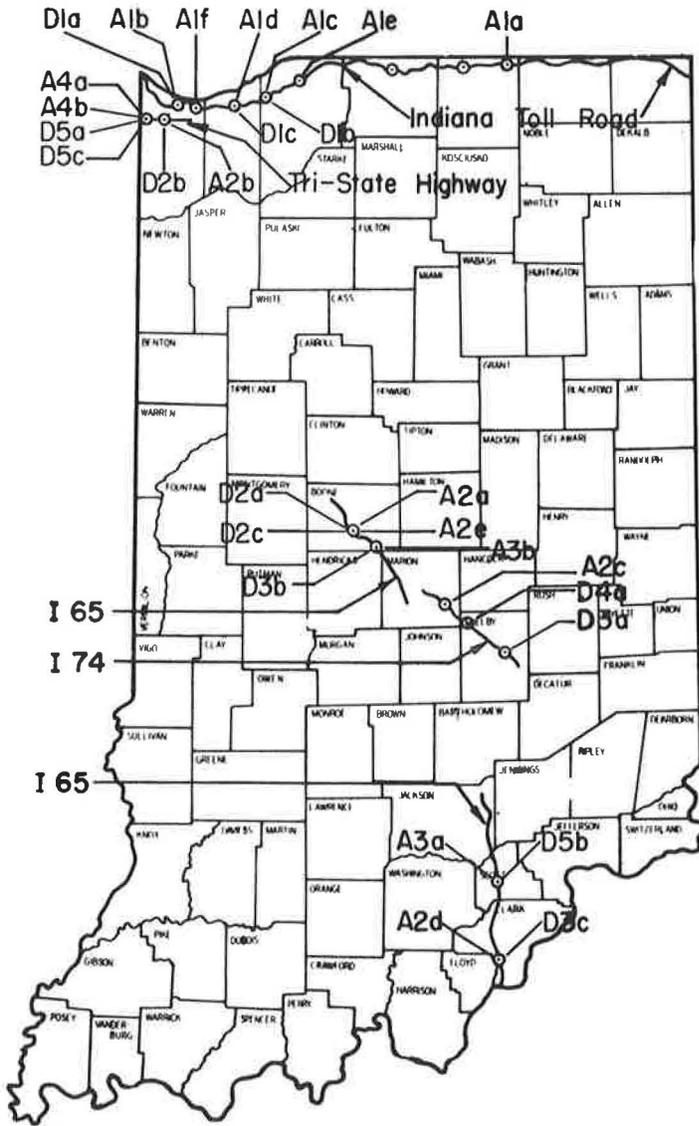


Figure 1. Study locations of acceleration and deceleration lanes.

These locations are coded as follows: A1a, A2b, etc., in which the A stands for acceleration lane; the number, for the type of lane design; and the lower-case letter (a, b, etc.), for the location.

Type 1.—One type of acceleration lane studied is on the Indiana Toll Road and is of the design shown in Figure 2. The length of the acceleration lane is 1,200 ft from the end of the ramp curve to the intersection with the right edge of the through lane. A shoulder, paved but of a different color, separates the acceleration lane from the through lane for a distance of 577.82 ft. For some of this distance, a curb also adjoins the acceleration lane. Beyond the point of separation, the acceleration lane is an extra lane with a 52:1 straight taper from a width of 12 to 0 ft in a distance of 622.03 ft.

Six locations of this type of acceleration lane were studied. At two of these locations, the acceleration lane meets the through lane on a tangent; at a third location, on

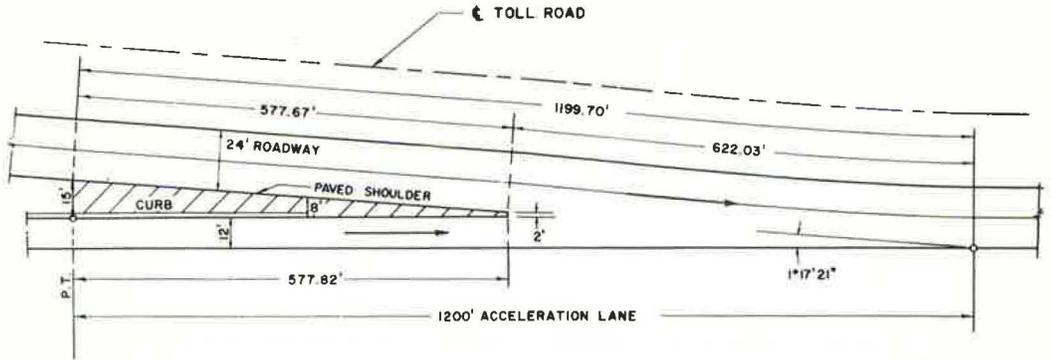


Figure 2. Acceleration lane, type 1.

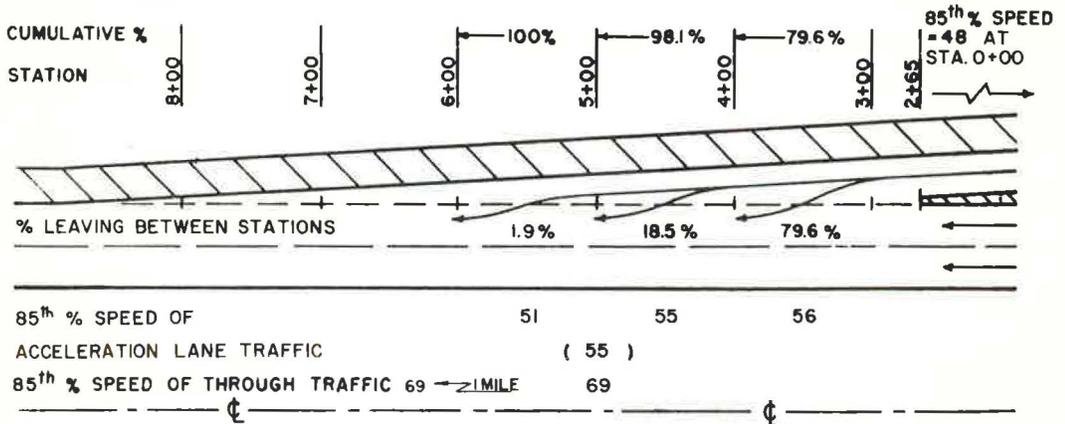


Figure 3. Speeds and lateral placement of passenger cars on type 1 acceleration lane, location A1a—Indiana Toll Road, Middlebury Interchange, westbound traffic.

a tangent and also on the downgrade portion of a sag vertical curve; at a fourth, on a right curve; at a fifth, on a right curve and also on the upgrade portion of a crest vertical curve; and at a sixth, on a left curve.

Station 0+00 was placed as the end of the ramp curb; beyond this point, acceleration lane traffic could merge into the through lanes.

Figures 3 through 8 show the results of the speed and lateral placement studies made at these locations. The results at location A1a are discussed in the following paragraphs as an example of how each of these figures and the data in Table 2 were analyzed. A summary of the important characteristics of use obtained at each location is given in Table 2.

In location A1a, the acceleration lane meets the through lane on a tangent at the interchange. The annual average daily traffic (1960) on the acceleration lane was 620 vehicles per day and on the westbound through lanes was 3,720 vehicles per day.

Figure 3 shows the results of the speed and lateral placement study made at this location. On this figure, as on similar figures for other locations of acceleration and deceleration lanes studied, 85th percentile speeds are shown for acceleration lane traffic and for through lane traffic. For acceleration lane traffic, the 85th percentile speed is given for Station 0+00 and for all vehicles which left the acceleration lane in each 100 ft thereafter at the time of merging. The percentage of the total non-stopping vehicles using the acceleration lanes which left the lane in each 100 ft and the cumulative percent-

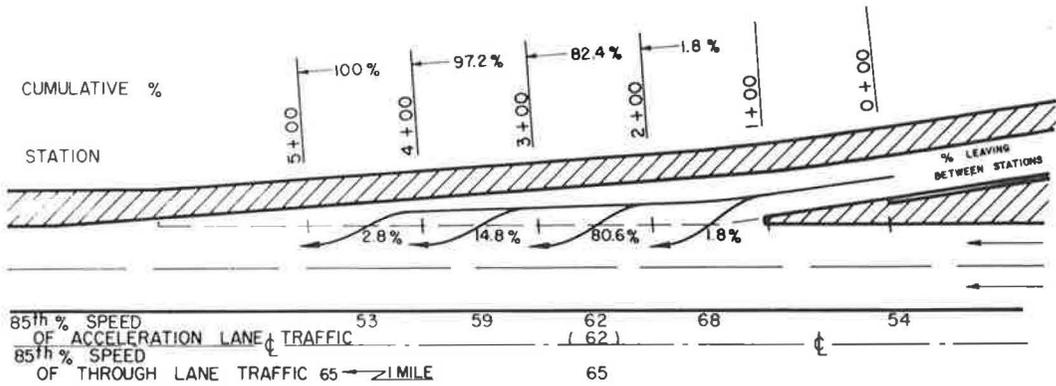


Figure 4. Speeds and lateral placement of passenger cars on type 1 acceleration lane, location Alb—Indiana Toll Road, Gary West Interchange, westbound traffic.

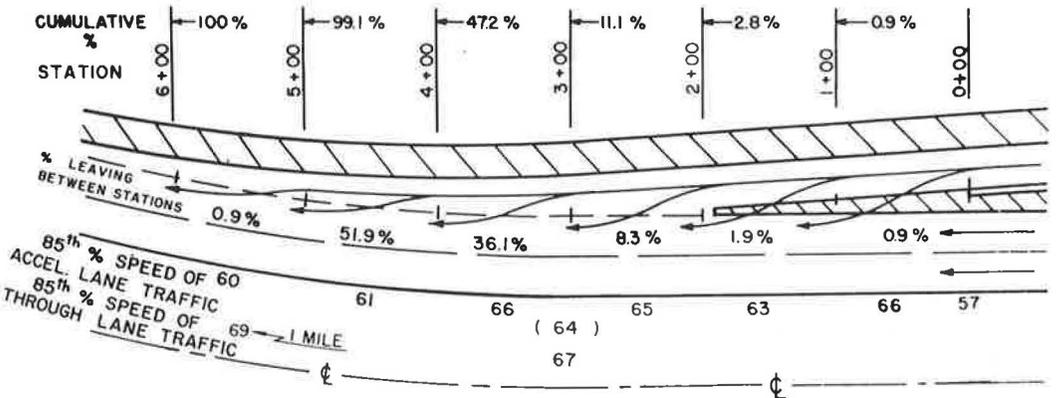
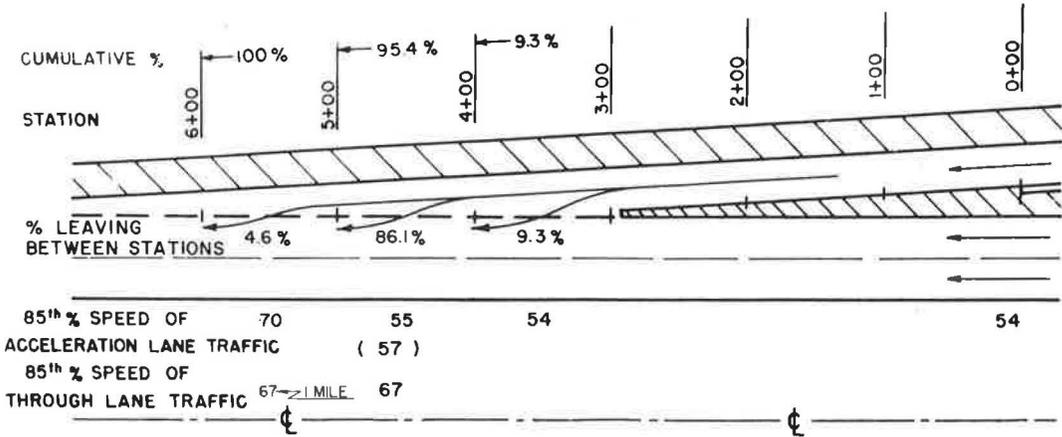


Figure 6. Speeds and lateral placement of passenger cars on type 1 acceleration lane, location Ald—Indiana Toll Road, Valparaiso Interchange, westbound traffic.

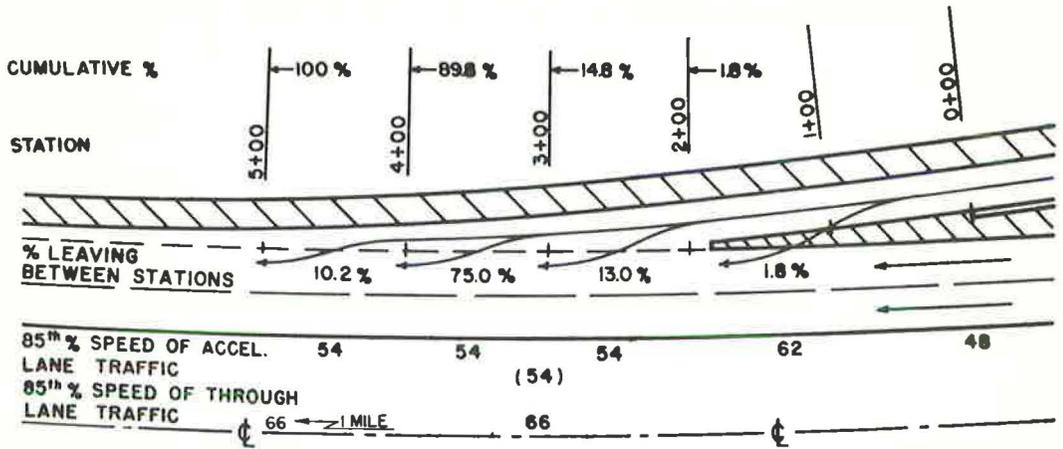


Figure 7. Speeds and lateral placement of passenger cars on type 1 acceleration lane, location Ale—Indiana Toll Road, LaPorte Interchange, westbound traffic.

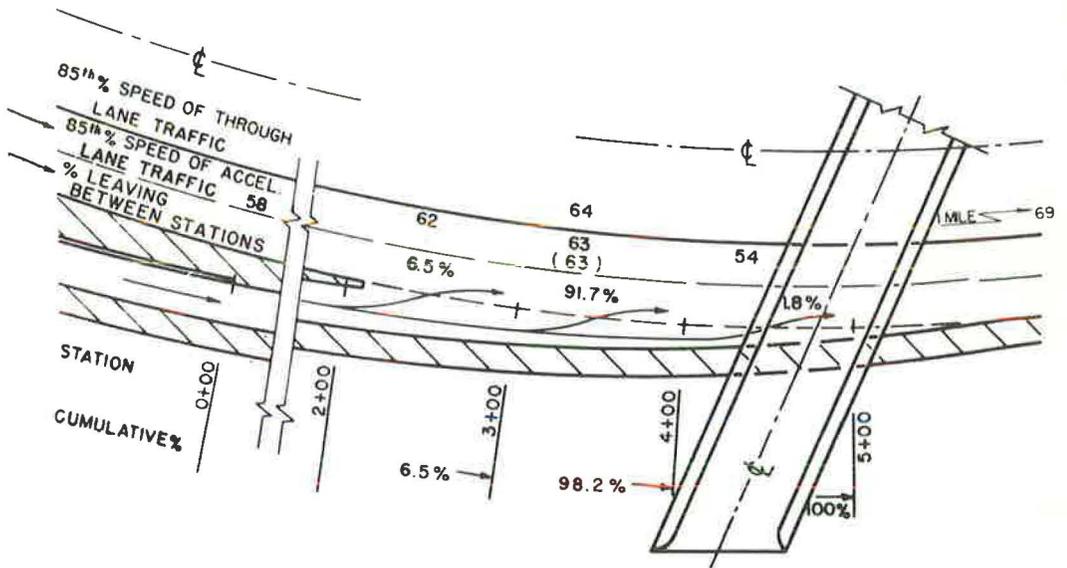


Figure 8. Speeds and lateral placement of passenger cars on type 1 acceleration lane, location Alf—Indiana Toll Road, Gary East Interchange, eastbound traffic.

age leaving at the end of each 100 ft are also given. The average 85th percentile speed of all vehicles using the acceleration lanes and at the time of their merging, except those that stopped, is shown by a number in parentheses; for example, (55). The 85th percentile speed for through lane traffic is given for a point within the area of merging and at a point approximately 1 mi from this area. All percentages of acceleration lane traffic used in this report are on the basis of the total number of passenger cars and light trucks using the lane which did not stop before entering the through lane.

The 85th percentile speed of acceleration lane traffic for location A1a at Station 0+00 was 48 mph (Fig. 3). The difference between the 85th percentile speed of through traffic within the area of conflict and the average 85th percentile speed of acceleration lane traffic as it merged into the through lane was 14 mph (69-55). This difference is

TABLE 2
SUMMARY OF RESULTS—ACCELERATION LANES

Location	Fig. No.	Geometric Condition ¹	Type of Accel. Lane ²	Annual Avg. Daily Traffic			85th Percentile Speed					Distance Beyond Point of Separation Within Which 90% of Accel. Lane Traffic Merged (ft)	Usable Length of Accel. Lane ⁵ (ft)	Length of Accel. Lane Unused by 90% of Through Lane Traffic (ft)	Accel. Lane Traffic That Stopped (%)	Was This Accel. Lane Used Satisfactorily as to	
				Accel. Lane	Through Lane	Station 0+00	Avg. Accel. Lane	Through Lane WAOC ³	Speed Diff. ⁴	Through Lane BAOC ⁵	Speed Diff. ⁴					Speed Placement	
A1a	3	1	1	620	3,720	48	55	69	14*	69	0	185	890	125	1.0	No	No
A1b	4	1	1	1,310	7,860	54	62	65	3	65	0	200	890	110	10.7	Yes	Yes
A1c	5	4	1	870	5,950	54	57	67	10*	67	0	200	890	110	1.8	No	Yes
A1d	6	2	1	650	6,420	57	64	67	3	69	2	260	890	60	3.6	Yes	Yes
A1e	7	5	1	1,230	5,030	48	54	66	12*	66	0	200	890	110	3.7	No	Yes
A1f	8	3	1	590	6,520	58	63	64	1	69	5*	190	890	120	0	Yes	Yes
A2a	10	1	2	260	4,550	42	47	66	19*	68	2	450	650	200	9.3	No	No
A2b	11	1	2	680	17,200	47	51	62	11*	62	0	500	650	150	16.0	No	No
A2c	12	1	2	1,130	3,030	55	55	61	6*	63	2	385	650	265	4.4	No	No
A2d	13	2	2	880	4,530	42	50	64	14*	58	-6*	600	650	50	14.3	No	No
A2e	14	3	2	1,380	4,170	44	46	66	20*	67	1	415	650	235	10.7	No	No
A3a	16	1	3	670	2,800	51	54	61	7*	65	4*	375	475	100	14.3	No	No
A3b	17	3	3	1,450	2,650	50	53	64	11*	63	1	365	475	110	3.6	No	No
A4a	19	1	4	1,960	23,730	44	48	60	12*	66	6*	135	250	115	34.6	No	No
A4b	20	2	4	3,880	21,620	41	45	60	15*	60	0	130	250	120	28.0	No	No

¹Refers to condition given in Table 1.

²Refers to type given in Table 1.

³Within area of conflict.

⁴* = significant.

⁵Beyond area of conflict.

⁶That length from end of ramp curve to where acceleration lane width is 6 ft wide.

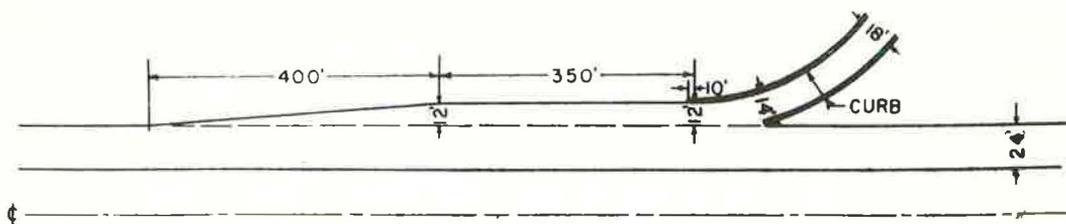


Figure 9. Acceleration lane, type 2.

statistically significant and indicates that much of the acceleration lane traffic did not accelerate to about the same speed as the through lane traffic before merging with it.

The difference between the 85th percentile speed of through lane traffic within the area of conflict and that beyond the area of conflict was zero (69-69).

Ninety percent of the traffic using the acceleration lane merged into the through lane before Station 4+50. This indicates that the majority of drivers merged within a distance of about 185 ft (4+50 - 2+65) beyond the point of separation of the acceleration lane from the through lane by means of the paved shoulder.

For this 52:1 taper design and at this location, the natural straight path of the left wheels of acceleration lane vehicles intersects the edge of the through lanes between Stations 4+00 and 5+00. Only 18.5 percent of acceleration lane traffic, however, merged in this area. The majority (79.6%) merged between Stations 3+00 and 4+00. It thus merged earlier than the natural straight path and at significantly lower speeds than through lane traffic.

A small percentage (1.9%) merged into the through lanes between Stations 5+00 and 6+00 at an 85th percentile speed of 51 mph. This small percentage of vehicles made use of a longer distance of the acceleration lane, and yet they merged at lower speeds than those at the previous indicated locations. The reason for this is either that these cars had to slow down due to conflicts with through traffic or that the drivers hesitated before merging due to inexperience in driving on high-type facilities or to not understanding the proper usage of acceleration lanes.

Of all vehicles using this acceleration lane, 1 percent stopped on the acceleration lane.

These results indicate that, at this location, this type of acceleration lane design was not being properly used by the majority of drivers.

Type 2.—The second type of acceleration lane studied is shown in Figure 9. The acceleration lane is of the parallel type having a full width lane for 350 ft plus 400 ft of taper. Five locations of this type of acceleration lane were studied. At three of the locations, the acceleration lane meets the through lane on a tangent; at the fourth location, on a right curve; and at the fifth location, on a left curve. Station 0+00 for this type of lane was taken to be at the nose. Beyond this point, acceleration lane traffic could merge into the through lane. The results of the speed and lateral placement studies of these five locations are shown in Figures 10 through 14.

Type 3.—The third type of acceleration lane studied is shown in Figure 15. The acceleration lane is of the parallel type having a full width lane of 250 ft plus 250 ft of taper.

Two locations of this type of acceleration lane were studied. At the first location, the acceleration lane meets the through lane on a tangent, and at the second, on a left curve. No suitable location could be found where the acceleration lane met the through lane on a right curve. Figures 16 and 17 show the results of the speed and lateral placement studies at these two locations.

Type 4.—The fourth type of acceleration lane studied is shown in Figure 18. This acceleration lane is of the taper design, but has only 300 ft of taper.

Two locations of this type were studied and both were at the same interchange. At the first location, the acceleration lane meets the through lane on a tangent, and at the second, on a right curve. No suitable location could be found where the acceleration

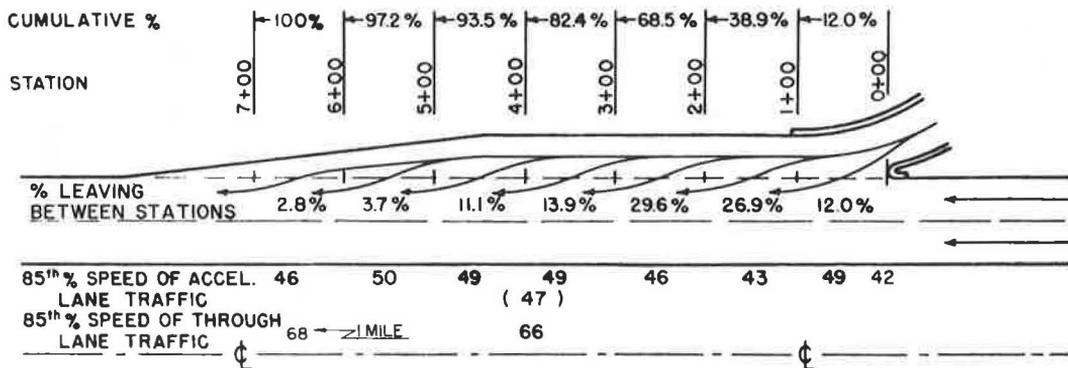


Figure 10. Speeds and lateral placement of passenger cars on type 2 acceleration lane, location A2a—Interstate 65, S. R. 39 Interchange, northbound traffic.

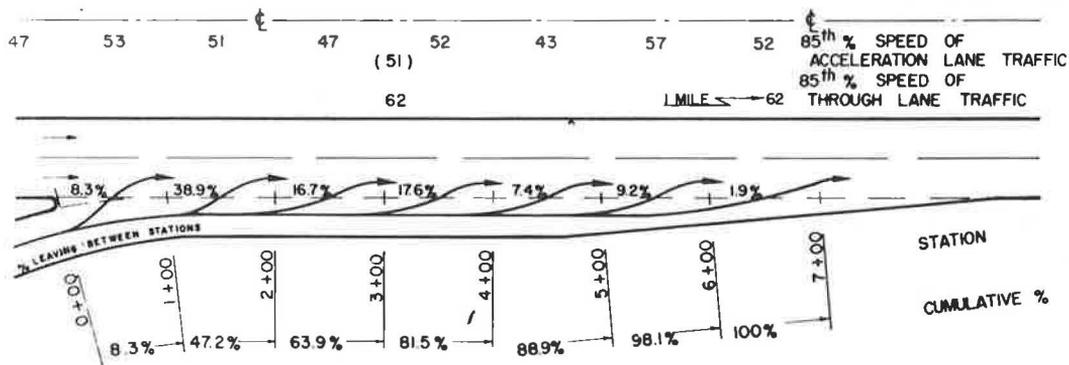


Figure 11. Speeds and lateral placement of passenger cars on type 2 acceleration lane, location A2b—Tri-State Highway, Kennedy Avenue Interchange, eastbound traffic.

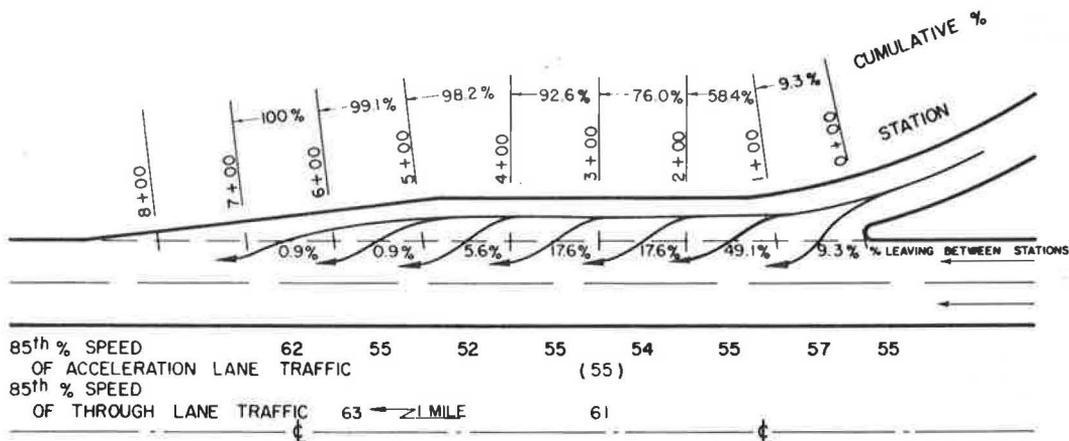


Figure 12. Speeds and lateral placement of passenger cars on type 2 acceleration lane, location A2c—Interstate 74, Post Road Interchange, eastbound traffic.

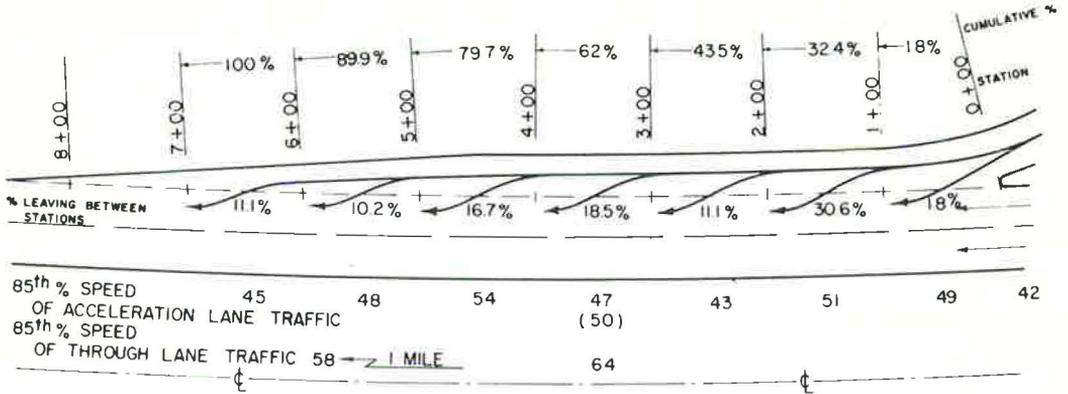


Figure 13. Speeds and lateral placement of passenger cars on type 2 acceleration lane, location A2d—Interstate 65, S. R. 60 Interchange, southbound traffic.

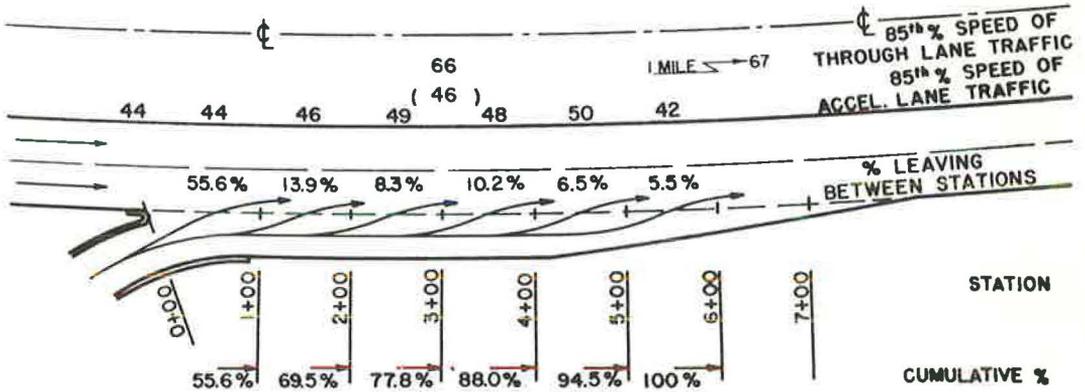


Figure 14. Speeds and lateral placement of passenger cars on type 2 acceleration lane, location A2e—Interstate 65, S. R. 39 Interchange, southbound traffic.

lane met the through lane on a left curve. Figures 19 and 20 show the results of the speed and lateral placement studies at these two locations.

Deceleration Lanes

The locations of the deceleration lanes studied are also shown in Figure 1, and pertinent data for each location are given in Table 3. These locations are referenced like those for acceleration lanes, except that the D stands for deceleration lanes.

Type 1.—The first type of deceleration lane studied was on the Indiana Toll Road (Fig. 21). The deceleration lane is 1,200 ft long from the right edge of the through lane to the beginning of the ramp curve. It consists of 845 ft of direct taper, having an angle of divergence of 1° 30 min with the through lanes. Along this distance, the deceleration lane is not separated from the through lane. It is then connected to a 356-ft direct taper, having an angle of divergence of 4° with the through lane. For this 356 ft, the deceleration lane is separated from the through lanes by a shoulder paved but of a different color than the traveled area. The exit nose is offset 6 ft from the edge of the through lane, and there are 173 ft of recovery.

Three locations were studied of this type of deceleration lane. At two of the locations,

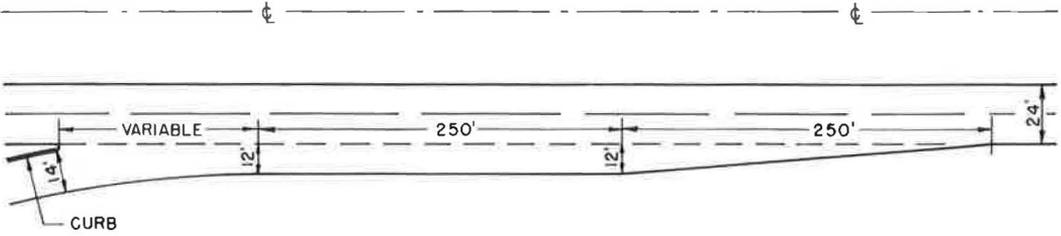


Figure 15. Acceleration lane, type 3.

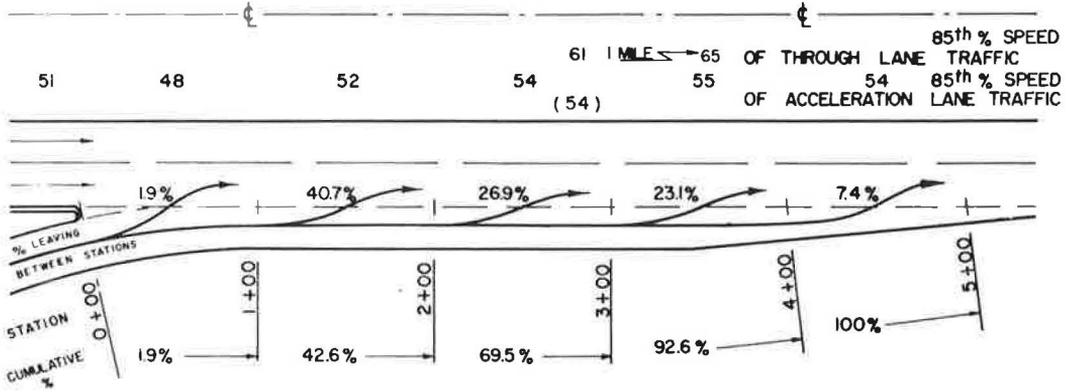


Figure 16. Speeds and lateral placement of passenger cars on type 3 acceleration lane, location A3a—Interstate 65, S. R. 56 Interchange, northbound traffic.

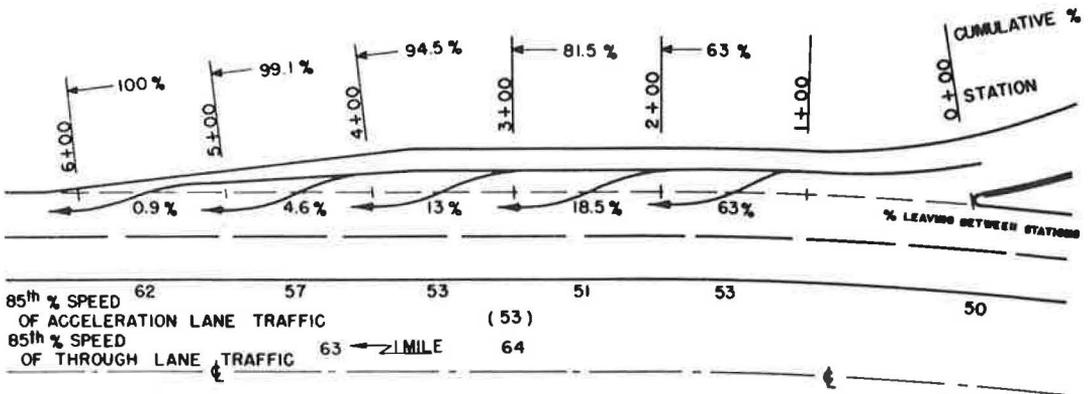


Figure 17. Speeds and lateral placement of passenger cars on type 3 acceleration lane, location A3b—Interstate 65, S. R. 334 Interchange, northbound traffic.

the deceleration lane leaves the through lanes on a tangent, and at the third, on a left curve. No suitable location could be found where the deceleration lane left the through lane on a right curve.

Station 0+00 was taken at the nose, which is the point where the paved shoulder that

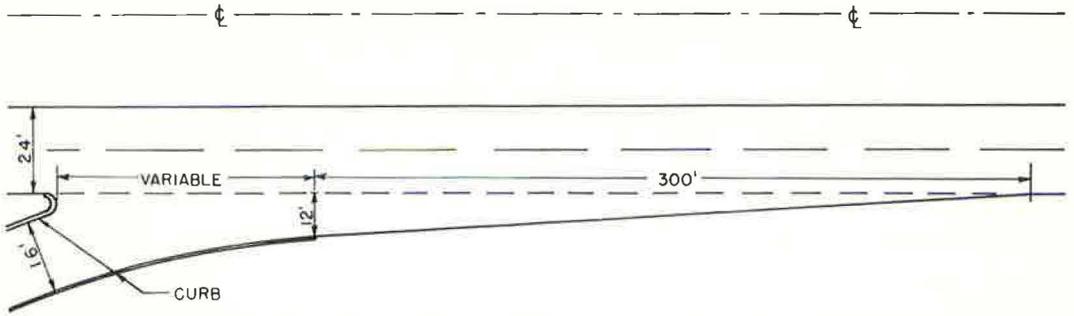


Figure 18. Acceleration lane, type 4.

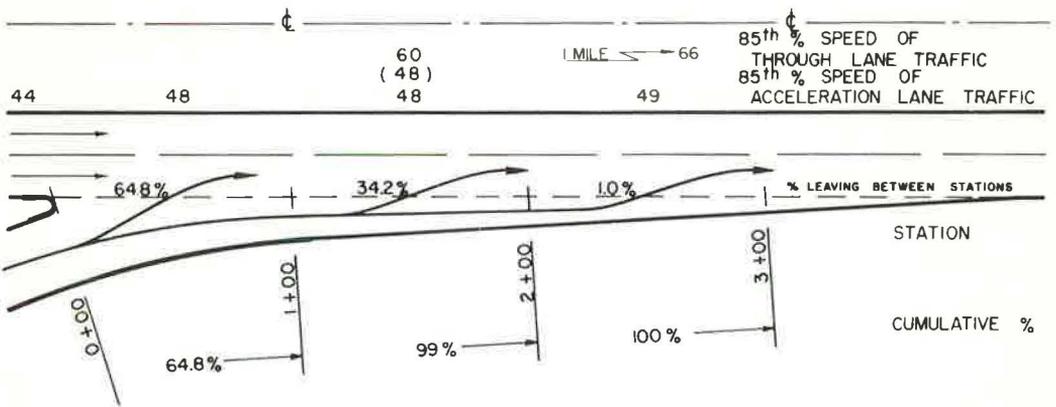


Figure 19. Speeds and lateral placement of passenger cars on type 4 acceleration lane, location A1a—Tri-State Highway, Calumet Ave. Interchange, eastbound traffic.

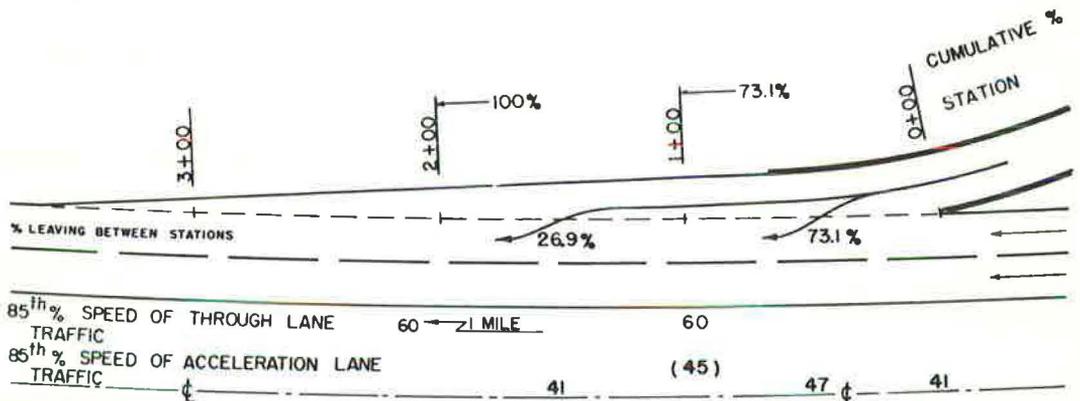


Figure 20. Speeds and lateral placement of passenger cars on type 4 acceleration lane, location A1b—Tri-State Highway, Calumet Ave. Interchange, westbound traffic.

separates the deceleration lane and the through lane begins. At the end of the deceleration lane and the beginning of the exit ramp there is an advisory speed sign, "Ramp Speed 25."

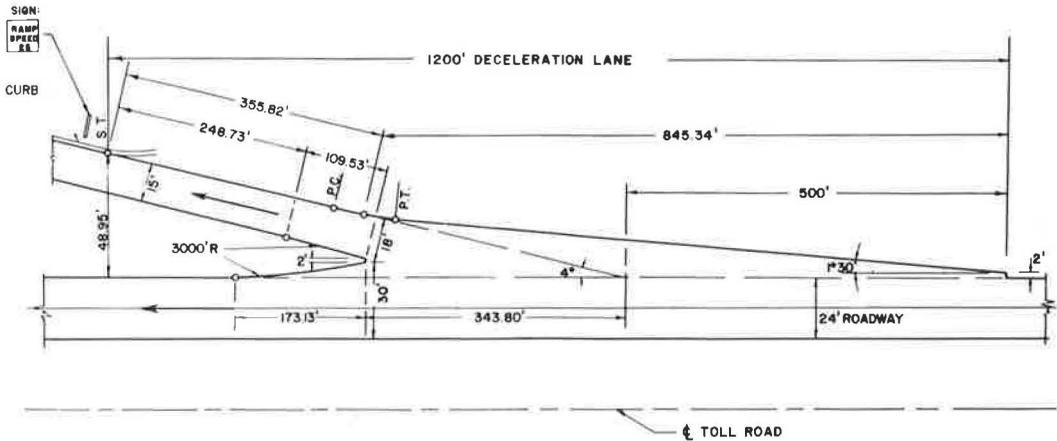


Figure 21. Deceleration lane, type 1.

TABLE 3
STUDY LOCATION DATA—DECELERATION LANES

Reference	Location	Direction of Deceleration Lane Traffic	Geometric Condition		Deceleration Lane	
			Description	Type	Description	Type
D1a	Indiana Toll Road—Gary West	Eastbound	Tangent	1	Direct taper 1,200 ft	1
D1b	Michigan City	Westbound	Tangent	1	Direct taper 1,200 ft	1
D1c	Chesterton—Valparaiso	Eastbound	Left curve	3	Direct taper 1,200 ft	1
D2a	Interstate 65 and State Road 39	Southbound	Tangent	1	250-ft taper and 50-ft straight + curve	2
D2b	Tri-State and Kennedy Avenue	Westbound	Tangent	1	250-ft taper and 50-ft straight + curve	2
D2c	Interstate 65 and State Road 39	Northbound	Right curve	2	250-ft taper and 50-ft straight + curve	2
D3a	Interstate 74 and State Road 9	Eastbound	Tangent	1	Curve	3
D3b	Interstate 65 and State Road 334	Southbound	Right curve	2	Curve	3
D3c	Interstate 65 and State Road 60	Northbound	Left curve	3	Curve	3
D4a	Interstate 74 and Pleasant View	Eastbound	Tangent	1	Direct taper 400 ft	4
D5a	Tri-State and Calumet Avenue	Westbound	Tangent	1	Taper 200 ft + curves	5
D5b	Interstate 65 and State Road 56	Northbound	Tangent	1	Taper 250 ft + curves	5
D5c	Tri-State and Calumet Avenue	Eastbound	Left curve	3	Taper 200 ft + curves	5

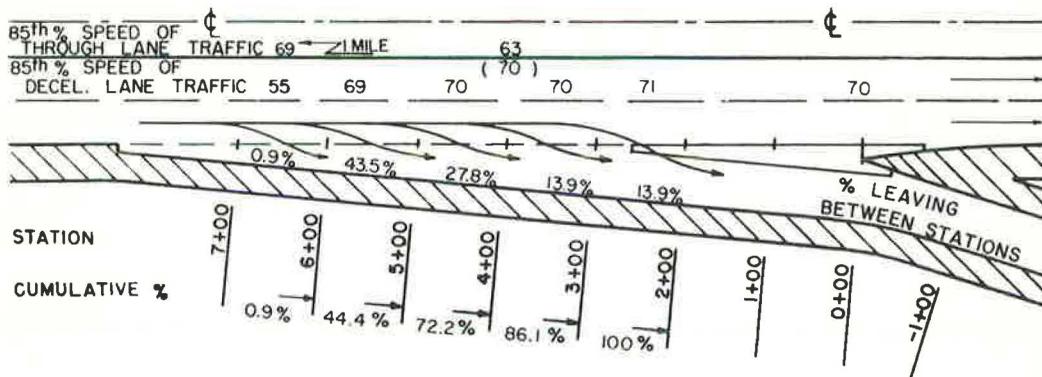


Figure 22. Speeds and lateral placement of passenger cars on type 1 deceleration lane, location D1a—Indiana Toll Road, Gary West Interchange, eastbound traffic.

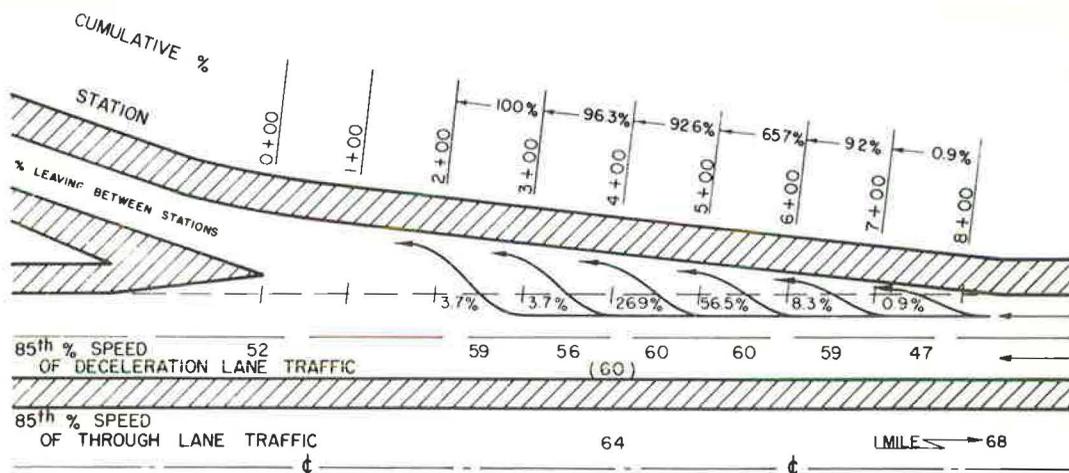


Figure 23. Speeds and lateral placement of passenger cars on type 1 deceleration lane, location D1b—Indiana Toll Road, Michigan City Interchange, westbound traffic.

Figures 22, 23, and 24 show the results of the speed and lateral placement studies at these three locations. The results at location D1c are described next as an example of how the information shown in the figures was analyzed. Also, summaries of the important deceleration lane characteristics and of the results obtained at each location studied are given in Table 4.

The deceleration lane leaves the through lane at location D1c on a left curve. The annual average daily traffic on the deceleration lane was 730 vehicles per day and on the through lanes eastbound was 6,630 vehicles per day.

The difference between the 85th percentile speed of through lane traffic within the area of conflict and the average 85th percentile speed of deceleration lane traffic as it diverged into the deceleration lane was 14 mph (69-55) (Fig. 24). This difference is significant, and indicates that most of this deceleration lane traffic started to decelerate on the through lane before diverging into the deceleration lane.

Because of the significant deceleration on the through lane at this location, a study was made as to where it occurred with respect to the beginning of the deceleration lane. Figure 25 shows for 200-ft intervals prior to the beginning of the deceleration lane the

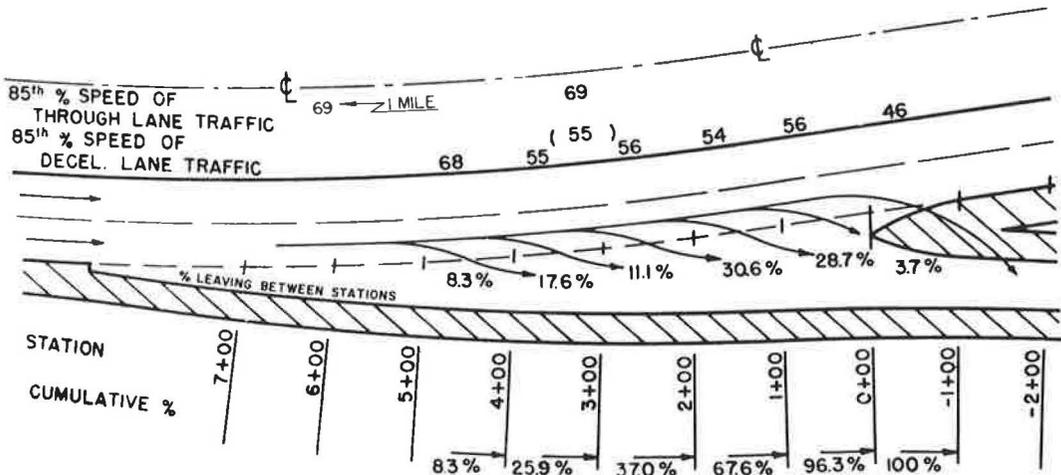


Figure 24. Speeds and lateral placement of passenger cars on type 1 deceleration lane, location Dlc—Indiana Toll Road, Chesterton-Valparaiso Interchange, eastbound traffic.

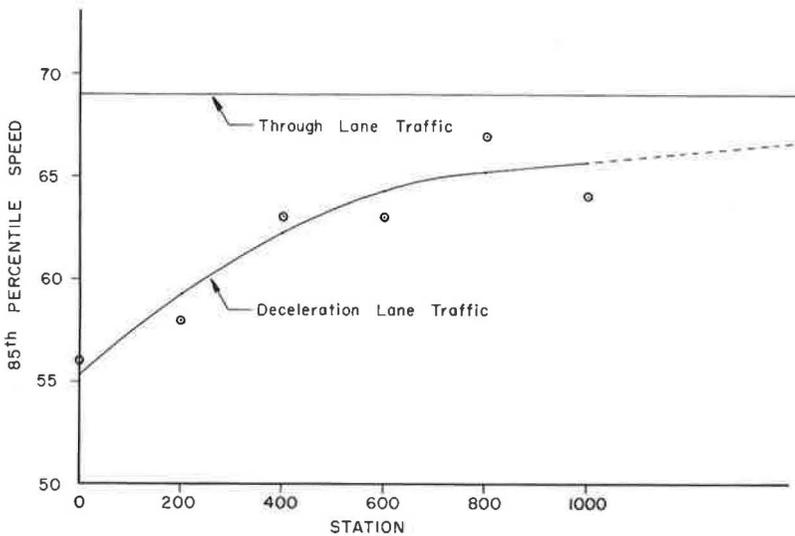


Figure 25. Determination of point where deceleration lane traffic begins to decelerate on through lane, location Dlc.

85th percentile speeds for the traffic that later entered the deceleration lane. Through traffic in this area was traveling at 69 mph. There is a clear indication that some of the traffic planning to use the deceleration lane at this location begins to decelerate well in advance (over 1,000 ft) of the deceleration lane.

The data in Figure 22, as well as that in similar figures for other deceleration lanes, are similar to that given for acceleration lanes, and explained earlier in this report.

Approximately 90 percent of the drivers using the deceleration lane diverged within a distance of approximately 400 ft.

The natural straight path of exit of the left wheels of the vehicles, which was taken as a line parallel to but offset by a distance of 3 ft from that path joining the point of beginning of the deceleration lane with the end point of the deceleration lane at the nose (Station 0+00), intersects the through lane between Station 4+00 and 5+00, where only

TABLE 4
SUMMARY OF RESULTS—DECELERATION LANES

Location	Figure No.	Geometric Condition ¹	Type of Decel. Lane ²	Annual Avg. Daily Traffic		85th Percentile Speed						Distance Within Which 90% of Decel. Lane Traffic Diverged in (ft)	Usable Length of Decel. Lane ⁵ (ft)	Length of Decel. Lane Not Used by 90% of Traffic (ft)	Was This Decel. Lane Used Satisfactorily as to	
				Decel. Lane	Through Lane	Station 0+00	Avg. Decel. Lane	Through Lane WAOC ³	Speed Diff. ⁴	Through Lane BAOC ⁵	Speed Diff. ⁴				Speed	Placement
D1a	22	1	1	1,460	7,680	70	70	63	-7*	69	6*	300	990	50	Yes	Yes
D1b	23	1	1	310	5,910	52	60	64	4*	68	4*	300	990	30	No	Yes
D1c	24	3	1	720	6,630	46	55	69	14*	69	0	400	990	230	No	No
D2a	27	1	2	240	4,170	49	49	67	18*	68	1	200	370	120	No	No
D2b	28	1	2	1,430	16,790	41	46	60	14*	63	3*	200	370	75	No	No
D2c	29	2	2	1,060	4,550	46	50	68	18*	70	2	200	370	90	No	No
D3a	31	1	3	—	—	46	51	62	11*	65	3*	100	150	0	No	Yes
D3b	32	2	3	1,010	3,230	50	54	65	11*	67	2	100	150	0	No	Yes
D3c	33	3	3	930	4,740	54	55	60	5*	60	0	200	300	25	No	Yes
D4a	37	1	4	250	3,780	50	57	63	6*	63	0	200	308	20	No	Yes
D5a	39	1	5	4,100	21,820	32	46	63	17*	62	1	100	240	40	No	Yes
D5b	40	1	5	770	2,800	51	56	64	8*	67	3	100	340	50	No	Yes
D5c	41	3	5	1,830	23,530	35	42	61	19*	60	1	100	240	50	No	Yes

¹Refers to geometric condition given in Table 3.

²Refers to type given in Table 3.

³Within area of conflict.

⁴** = significant.

⁵Beyond area of conflict.

⁶That length from beginning of ramp curve to where deceleration lane width is 6 ft wide.

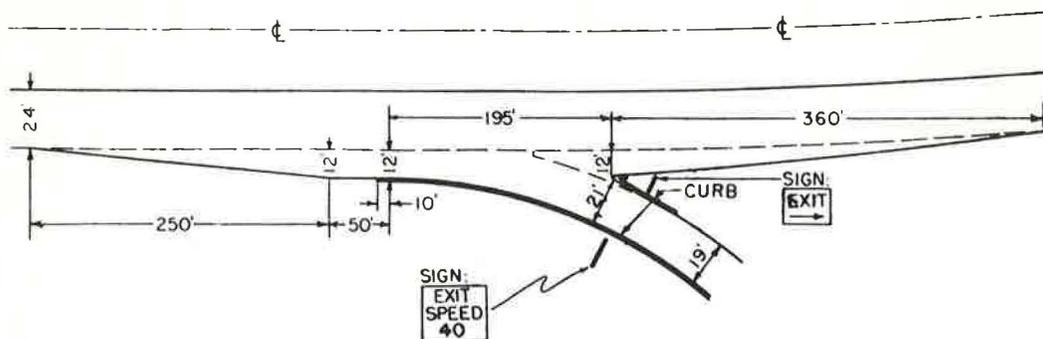


Figure 26. Deceleration lane, type 2.

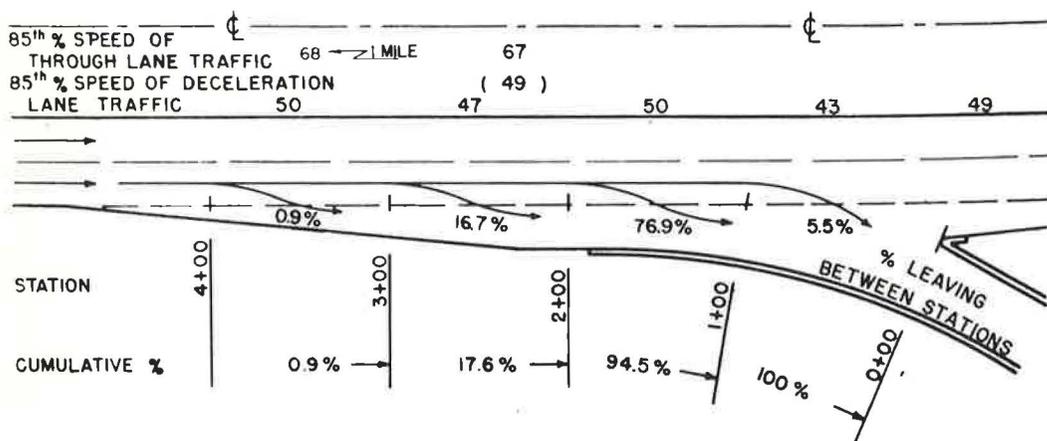


Figure 27. Speeds and lateral placement of passenger cars on type 2 deceleration lane, location D2a—Interstate 65, State Route 39 Interchange, southbound traffic.

a small percentage (8.3%) of deceleration lane traffic diverged at an 85th percentile speed of 68 mph. Most of the remaining drivers diverged later and at lower speeds, thus indicating deceleration on the through lane instead of on the deceleration lane. This could have been aggravated by the effect of the left curve.

A small percentage (3.7%) of deceleration lane traffic crossed the paved shoulder and diverged between Stations 0+00 and 1+00 at an 85th percentile speed of 46 mph. This speed is lower than that of other observed deceleration lane traffic, and this might have been caused by these drivers hesitating on the through lane and making the decision at the last minute to leave at this interchange.

These results indicate that drivers were not properly using the deceleration lane at this location for this roadway geometry and this type of deceleration lane design.

Type 2.—The second type of deceleration lane studied is shown in Figure 26. The deceleration lane has a taper from 0 to 12 ft in a distance of 250 ft, followed by 50 ft of tangent and 195 ft of curve. Near the exit nose there is a regulatory speed sign, "Exit Speed 40." Three locations of this type of deceleration lane were studied. At two of the locations, the deceleration lane leaves the through lane on a tangent, and at the third, on a right horizontal curve. No suitable location could be found where the deceleration lane left the through lane on a left curve. Station 0+00 was taken at the nose of the deceleration lane. The results of the speed and lateral placement studies at these locations are shown in Figures 27, 28, and 29.

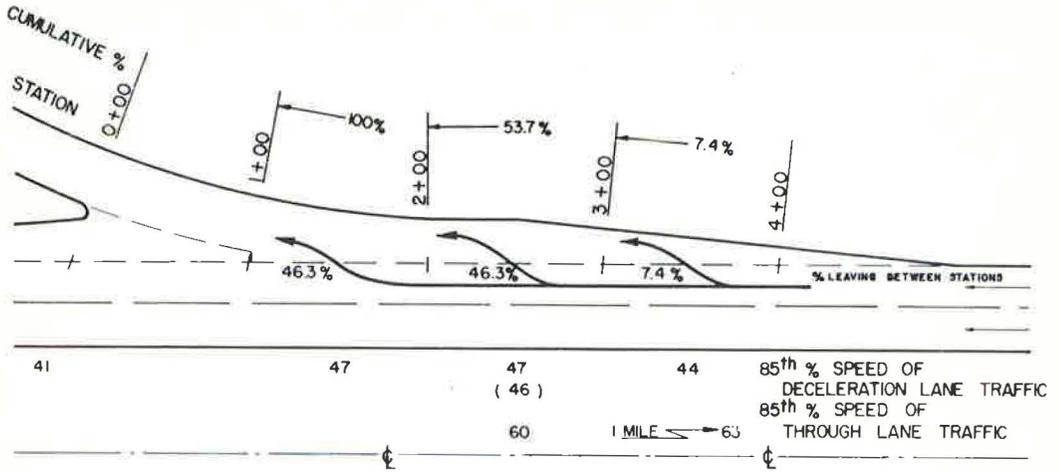


Figure 28. Speeds and lateral placement of passenger cars on type 2 deceleration lane, location D2b—Tri-State Highway and Kennedy Avenue Interchange, westbound traffic.

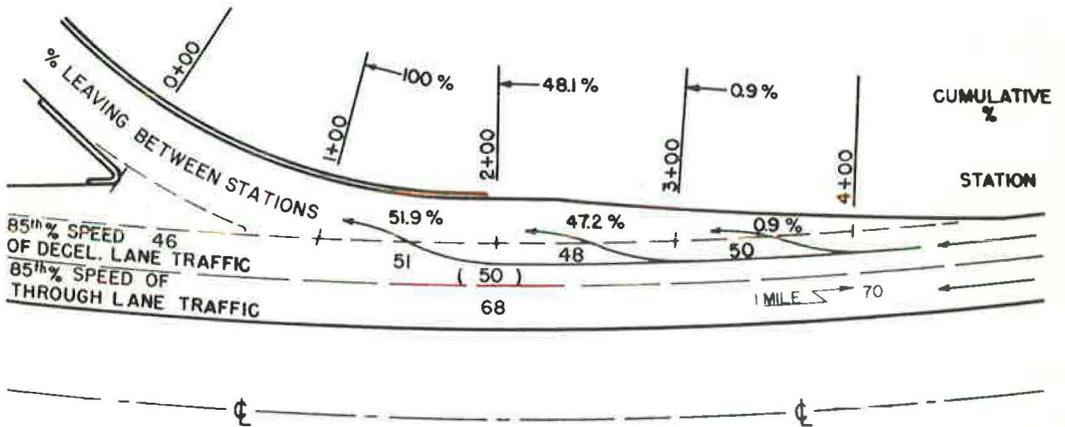


Figure 29. Speeds and lateral placement of passenger cars on type 2 deceleration lane, location D2c—Interstate 65, State Route 39 Interchange, northbound traffic.

Type 3.—The third type of deceleration lane studied is shown in Figure 30. The deceleration lane in this design is a curve of varying degrees of curvature. Near the nose there is a regulatory speed sign, "Exit Speed 40."

Three locations were studied of this type of deceleration lane. At the first location, the deceleration lane leaves the through lane on a tangent; at the second, on a right curve; and at the third, on a left curve. The results of the speed and lateral placement studies at these three locations are shown in Figures 31, 32, and 33. Figures 34 and 35 show the results of a study at two of these locations where the distance prior to the beginning of the deceleration lane and in which the deceleration occurred was determined.

Type 4.—The fourth type of deceleration lane studied is shown in Figure 36. The deceleration lane in this design is a direct taper 400 ft long. Near the exit nose, there is a regulatory speed sign, "Exit Speed 40."

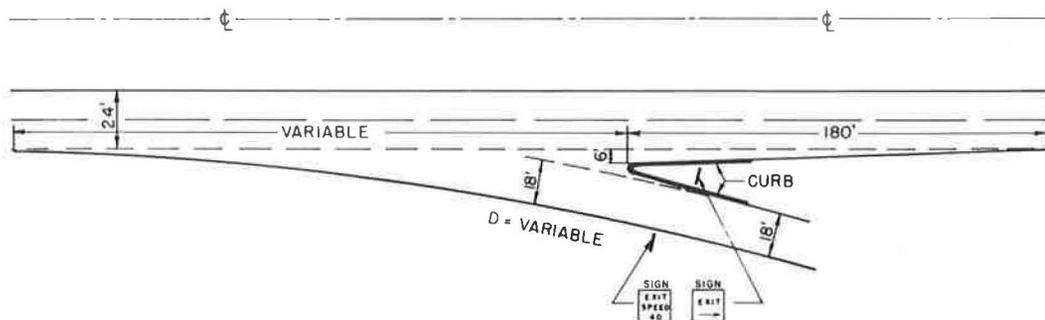


Figure 30. Deceleration lane, type 3.

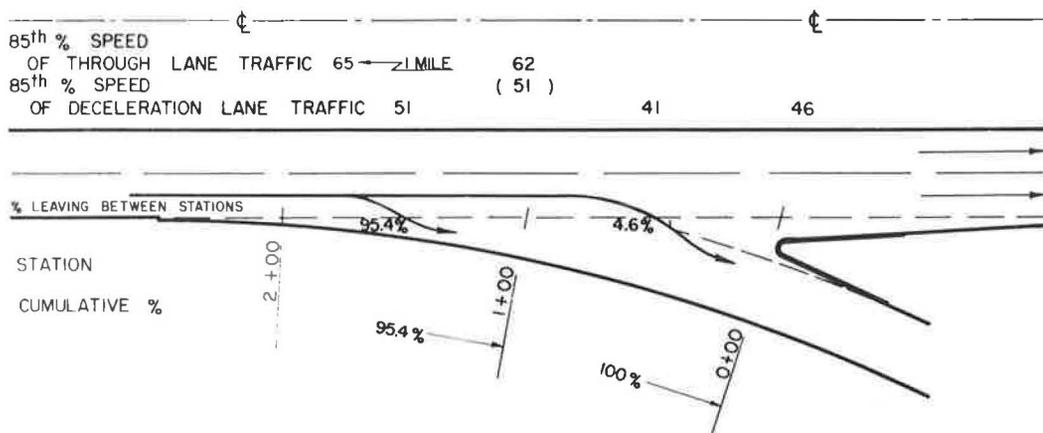


Figure 31. Speeds and lateral placement of passenger cars on type 3 deceleration lane, location D3a—Interstate 74, S. R. 9 Interchange, eastbound traffic.

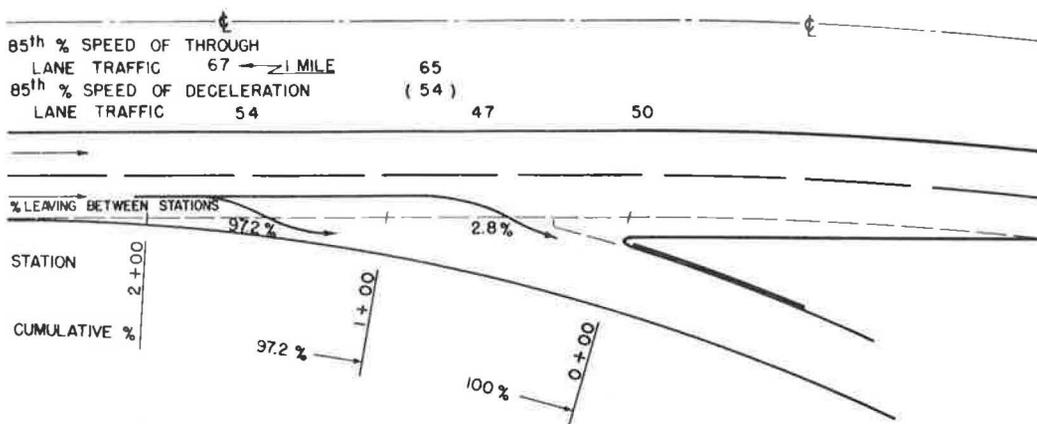


Figure 32. Speeds and lateral placement of passenger cars on type 3 deceleration lane, location D3b—Interstate 65 and S. R. 334 Interchange, southbound traffic.

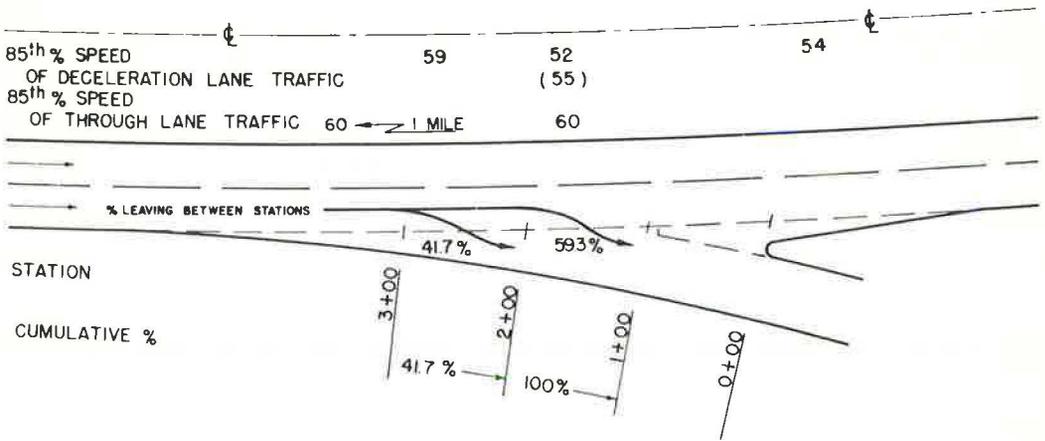


Figure 33. Speeds and lateral placement of passenger cars on type 3 deceleration lane, location D3c—Interstate 65 and S. R. 60 Interchange, northbound traffic.

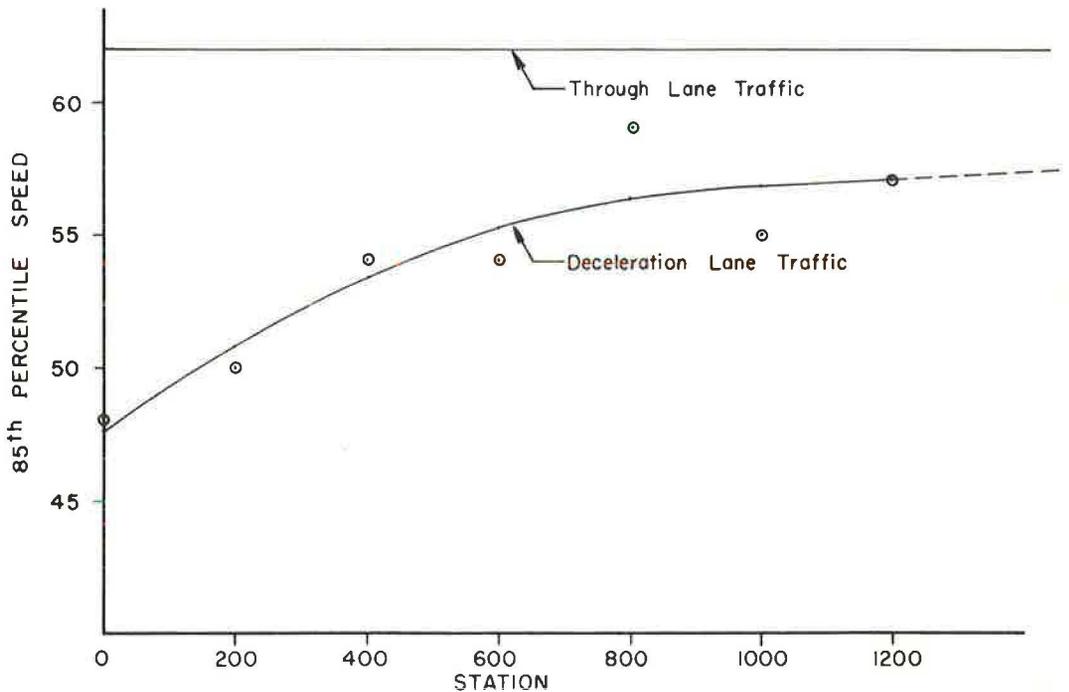


Figure 34. Determination of point where deceleration lane traffic begins to decelerate on through lane, location D3a.

Only one location was studied of this type of deceleration lane as it was the only one completed as of the date of this study. Figure 37 shows the results of the speed and lateral placement study at this location.

Type 5.— The fifth type of deceleration lane studied is shown in Figure 38. The deceleration lane in this design is a variable distance, direct taper followed by a curve. Near the exit nose, there is a regulatory speed sign, "Exit Speed 40."

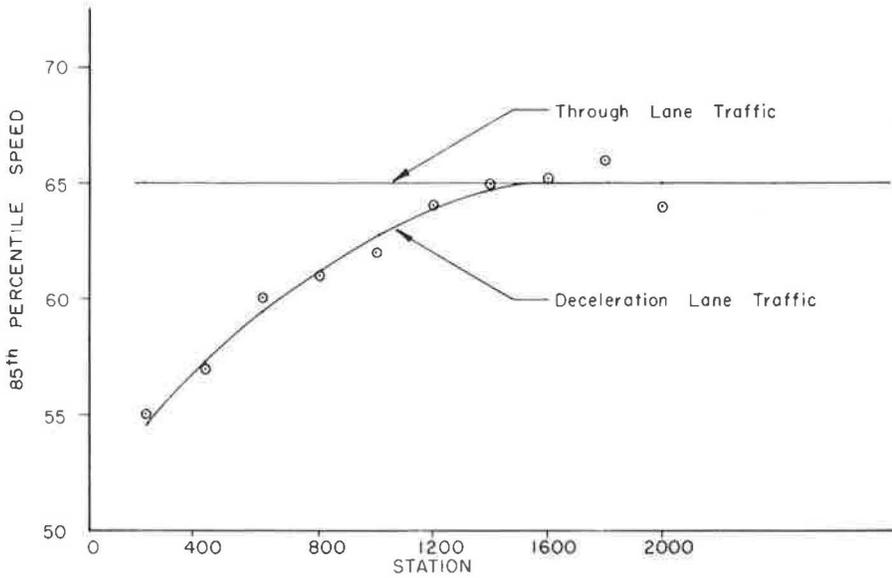


Figure 35. Determination of point where deceleration lane traffic begins to decelerate on through lane, location D3b.

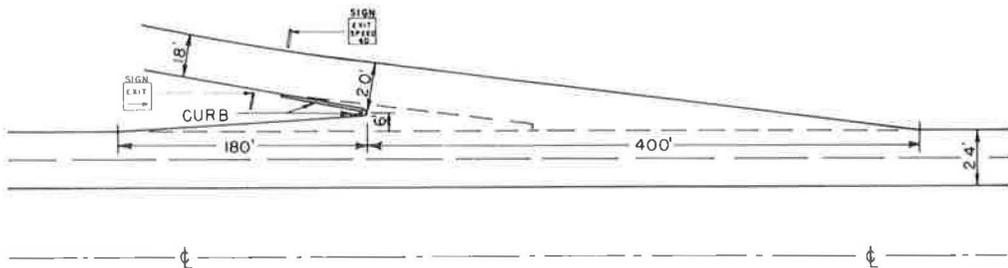


Figure 36. Deceleration lane, type 4.

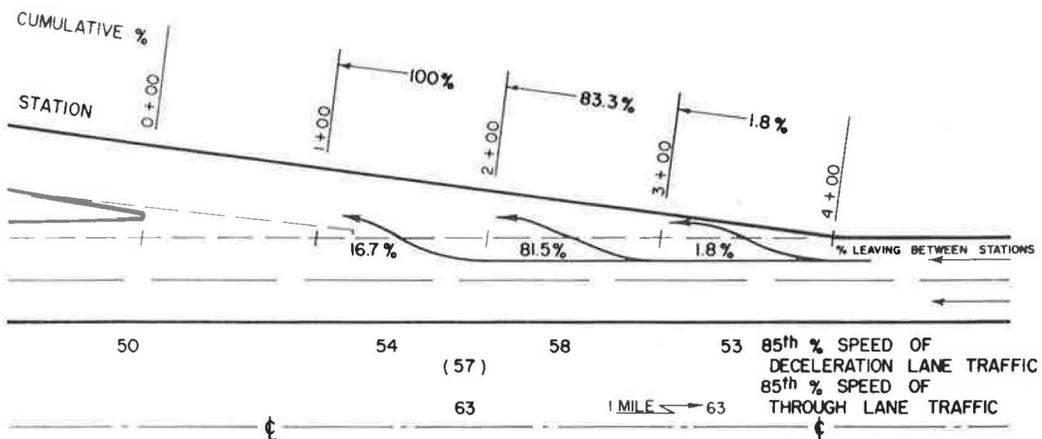


Figure 37. Speeds and lateral placement of passenger cars on type 4 deceleration lane, location D4a—Interstate 74, Pleasant View Interchange, eastbound traffic.

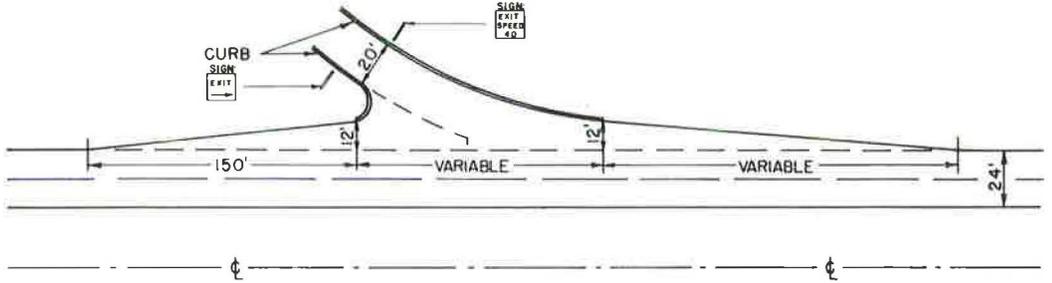


Figure 38. Deceleration lane, type 5.

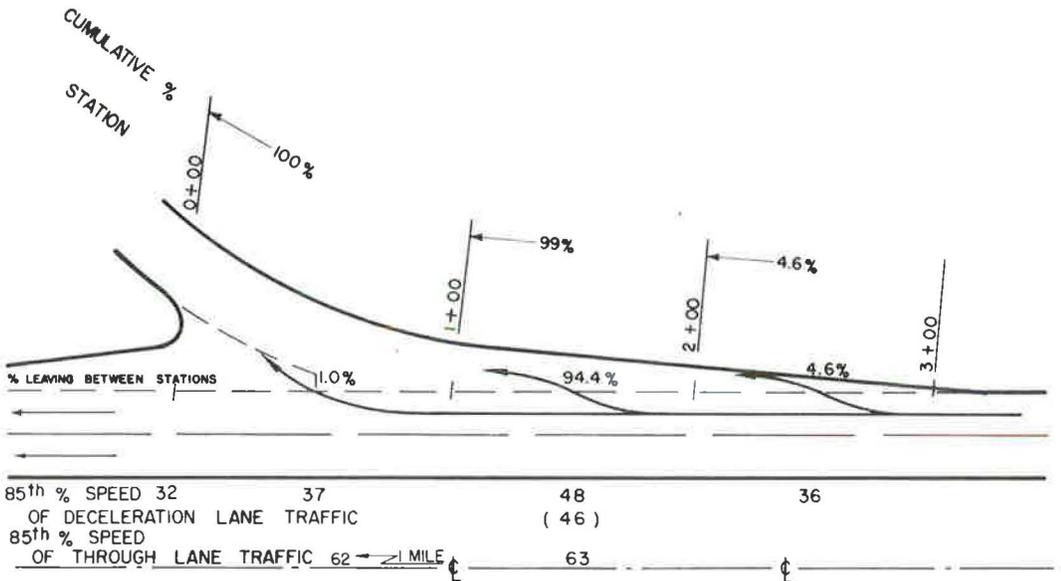


Figure 39. Speeds and lateral placement of passenger cars on type 5 deceleration lane, location D5a—Tri-State Highway and Calumet Avenue Interchange, westbound traffic.

Three locations were studied of this type of deceleration lane. At the first two locations, the deceleration lanes leave the through lanes on a tangent, and at the third, on a left curve. At the first and third locations, the length of the tangent section is 200 ft and the length of the curve portion to the nose is 139 ft, whereas at the second location, these lengths are 250 and 217 ft, respectively. Figures 39, 40, and 41 show the results of the speed and lateral placement studies at these three locations.

SUMMARY AND FINDINGS

Summaries of the important acceleration lane characteristics and of the results obtained at each location studied are given in Table 2. A similar table for the deceleration lanes studied is Table 4. These two tables provide a comparison of the use characteristics found on the various designs and permit a rather rapid comparison of the various designs.

The behavior of vehicles on acceleration and deceleration lanes as found and reported in the figures and tables indicates the following:

General

1. There were large differences in speed between acceleration or deceleration lane traffic at the time of merging or diverging, respectively, and that of the through lanes on many of the highway sections studied. It is obvious that better use of these and similar facilities that will be constructed is desirable.

2. A large number of motorists apparently do not know how to use acceleration and deceleration lanes properly. Thus, for the most efficient and safest operation of traffic, it is imperative that the driving public be better informed on the proper use of acceleration and deceleration lanes. It is recommended that all the interested agencies of the city, State, and Federal governments recognize their responsibilities in this area and formulate at an early date a program of education directed toward the proper use of acceleration and deceleration lanes.

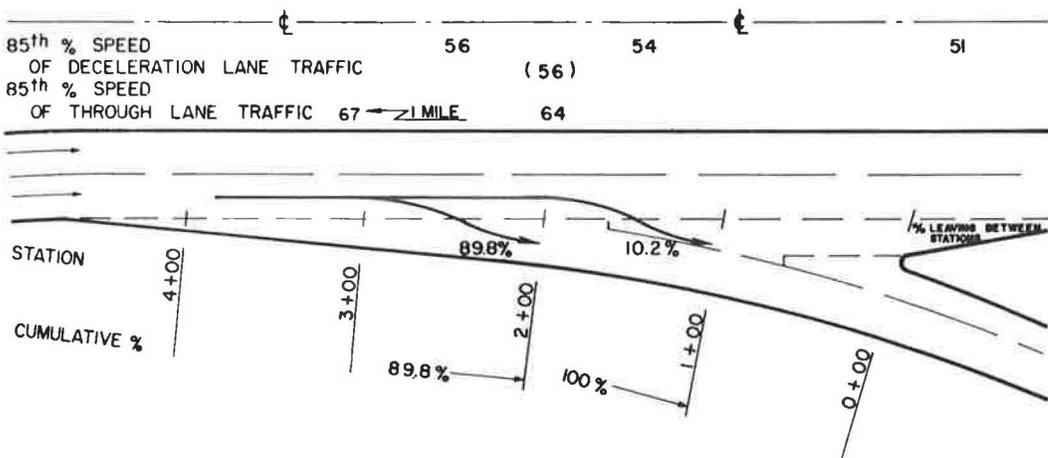


Figure 40. Speeds and lateral placement of passenger cars on type 5 deceleration lane, location D5b—Interstate 65 and S. R. 56 Interchange, northbound traffic.

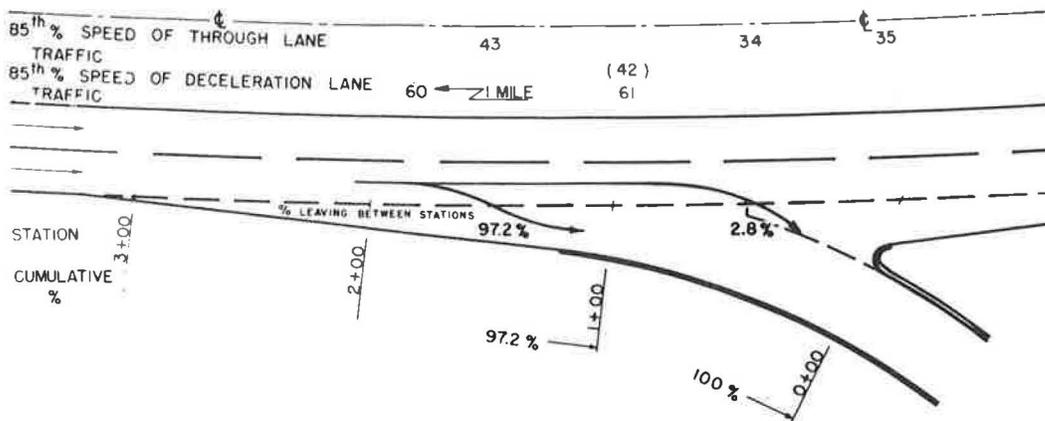


Figure 41. Speeds and lateral placement of passenger cars on type 5 deceleration lane, location D5c—Tri-State Highway and Calumet Avenue Interchange, eastbound traffic.

To facilitate this education of the driving public in the use of such lanes and to eliminate much of the confusion that motorists now exhibit, some standardization of design for acceleration and deceleration lanes should be adopted, at least within the same geographic region. The Indiana State Highway Commission has already constructed three different designs of acceleration lanes and four different designs of deceleration lanes on the Interstate sections that have been opened to traffic. On some of these sections, the design varies from one interchange to the next.

3. The use results were not similar at all locations for acceleration and deceleration lanes having the same design and operating under the same roadway geometry. This indicates that other factors, such as types of driver using the lanes and traffic volume, had a significant bearing on the results.

4. Acceleration or deceleration lane traffic had little effect on the speed of through traffic at interchanges where acceleration and deceleration lanes were studied in this research. At only 8 of the 28 acceleration and deceleration lane locations studied was the difference between the 85th percentile speed of the through lane traffic within the area of conflict and that beyond the area of conflict statistically significant. Drivers slowed at 7 of these locations and traveled faster at the eighth. The change in speed was never greater than 6 mph. At some of the locations where the speed effect was significant, other factors, such as a narrow median or a horizontal curve, probably contributed to the changes in speed.

5. For the best use for both acceleration and deceleration lanes at an interchange location, it is desirable to have the through lanes, at the location of the acceleration or deceleration lanes, on a tangent.

Acceleration Lanes

1. A higher percentage of drivers utilized more length of the acceleration lane when the acceleration lane met the through lane on a right curve, and less length of the acceleration lane when it met the through lane on a left curve, than under the condition where the acceleration lane met the through lane on a tangent.

2. On the four acceleration lane designs studied, the long direct taper type of design with separation from the through lanes for approximately 500 ft was found to be the only type where most drivers tended to approach the optimum condition of operation for acceleration lanes.

3. For the long direct taper design, a high percentage of drivers followed a natural straight path from the beginning of the acceleration lane at the end of the ramp curve until they merged into the through lanes. Some control, such as a curb, appeared to be desirable beyond the end of the ramp curve to align some motorists properly in a straight path, and thus prevent too early a merging at too low a speed into the through lane.

4. For the long direct taper type of design studied, most drivers merged (left rear wheel entered through lane) within a maximum distance of 260 ft beyond the nose.

5. Most drivers tended to merge soon after entering all parallel acceleration lanes studied and at too low a speed. A longer length of the parallel portion of the acceleration lane did not show better usage than a shorter length.

6. Most drivers tended to merge as soon as possible and at too low a speed on the short taper design acceleration lanes which had no separation from the through lanes.

Deceleration Lanes

1. A higher percentage of drivers utilized less length of the deceleration lane and diverged later at lower speeds, thereby decelerating more on the through lane, when the deceleration lane left the through lane on a left curve than when the deceleration lane left the through lane on a right curve or tangent.

2. Most drivers at most locations studied did not obey the regulatory speed signs (Exit Speed 40) placed near the nose of the deceleration lane on the Interstate sections. It is suggested that this regulatory speed sign be changed to an advisory ramp speed sign with the numerical value the safe design speed of the exit ramp and placed so as to permit comfortable deceleration to this safe speed.

3. At all but one of the locations studied, much of the deceleration lane traffic started to decelerate on the through lane before diverging into the deceleration lane. The results further show that many drivers began to decelerate well in advance (more than 1,000 ft) of the beginning of the deceleration lane.

4. Of the five types of design studied, the long direct taper type of deceleration lane with separation of the lane from the through lane for about 300 ft (as on the Indiana Toll Road) was found to be the best. On this design, most drivers tended to approach the optimum condition of operation for deceleration lanes.

5. Most drivers on the deceleration lanes studied desired to follow a natural straight path of exit with a minimum of maneuvering.

6. Ninety percent of the drivers diverging onto a deceleration lane did so within a maximum distance of 300 ft, except for one case where the lane was on a left horizontal curve.

7. The curve type design for deceleration lanes tended to provide good usage throughout the length of the lane, but most drivers decelerated considerably before entering lanes of this design.

8. Most drivers tended to decelerate appreciably on the through lane where a short direct taper design was used.

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