SIMULATION OF TRAFFIC FLOW ON THE I-291 AND ROUTE 15 THREE LEVEL DIAMOND INTERCHANGE

Final Report

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I. INTRODUCTION

One of the more challenging problems in the design of controlled access highways is that of providing for the interchange of traffic on two or more intersecting thruways. The diamond interchange is now being used for this purpose because it requires less area than the traditional cloverleaf. A major disadvantage of the diamond interchange, however, is that vehicles desiring to make a left turn will in general be required to stop. It is therefore important that suitable signalling methods be developed to optimize interchange performance under conditions of varying traffic volumes.

The objective of the research reported here has been to develop a simulation of the I-291 and Route 15 diamond interchange in Newington, Connecticut, to evaluate several methods for controlling left turning traffic, and to make recommendations regarding implementation and further study. This interchange, shown to scale in Fig. 1, has three levels - one level for each thruway and a third to accomplish the interchange of traffic. Simulation has been limited to the interchange level only, and more particularly to that portion of traffic undergoing a left turn.

Figures 2 and 3 provide estimates of 1990 peak AM and PM volumes of through traffic, right turning traffic, and left turning traffic. Of principal interest are the left turning volumes, which in Figure 3 are 500, 300, 900, and 400 vehicles per hour, respectively, for vehicles arriving from the North, East, South, and West. (The AM peak left turn volumes of Figure 2 are substantially smaller). Figure 4 shows, schematically, the simulated portion of the interchange and the lane designations to be used. Each lane is approximately 440-ft. in length including the "input lanes" denoted North 01, North 02, West 01, West 02, etc. The progress of each vehicle is simulated from "birth time" until it exits at the final intersection. Thus, a vehicle arriving from the South, for example, is assigned to the shorter of lanes South 01, South 02, and proceeds successively through lanes South 01, (South 02), South 2, (South 3), East 1, (East 2) until it exits at the North intersection.

Not shown in Figure 4 are lanes North 11, North 12, etc., which are 300-ft subsections of North 01, North 02, etc. Queue lengths in these 300-ft lanes are measured for use in signal control algorithms.

Procedures for simulation of traffic flow and signal control are outlined in the next section. Signal control strategies are presented in Section III. Most of these are simple variations of a method proposed by the Traffic Engineering Division of the Connecticut DOT. Section IV summarizes the simulation results and contains recommendations for improvement of the control algorithms.

In short, the simulation studies indicate that the interchange can efficiently service traffic volumes equal to or greater than those projected for 1990. It appears that there may be some advantage to use of a signal control strategy which would require a small special purpose digital computer to implement.

II. SIMULATION METHODS

2.1 Definition of Terms

For convenience in discussing succeeding material, several definitions will be introduced:

Interior or Circulating Lanes - those lanes denoted 1, 2, 3 in Figure 4 and used by vehicles to accomplish a left turn.

Input Lanes - All lanes denoted 01, 02 used by vehicles to enter the interior lanes.

Occupancy Lanes - Lanes denoted 11, 12 (not shown in Figure 4). These lanes are a portion of input lanes 01, 02 extending about 300 ft from the intersection. Occupancy lane queues are used in the signalling schemes to determine the duration and starting time of the next green interval.

Phase A - the light sequence experienced by vehicles on the circulating lanes.

Phase B - the light sequence experienced by vehicles entering on the input lanes.

Intersections North, East, South, West - intersections in Figure 4 at which vehicles enter respectively on input lanes designated North, East, South, and West.

Transit Time for a Designated Lane - the total time spent in the lane, including travel time, waiting time at a red light, if any, and time required to cross the subsequent intersection. Except for lanes designated 11 or 22, all lanes are 440 - ft in length. The basic travel time of each 440 - ft lane is a random number for each vehicle, uniformly distributed over 12.5 - 15.5 seconds with a mean of 14 seconds. The minimum time through an intersection is 2 seconds. Thus the minimum transit time is 14.5 seconds for all regular lanes.

Total Transit Time for vehicles entering North, South, East or West - total time spent in the interchange, starting 440 ft from the first intersection, terminating when vehicle discharges at the last intersection. Thus the transit time for a vehicle entering North is time spent in North 01 (North 02) plus time spent in North 2 (North 3) plus time spent in West 1 (West 2). The absolute minimum possible total transit time if a vehicle makes no stops, is 37.5 seconds (using a minimum transit time of 12.5 second/lane as given above.) Actual minimum total transit times observed during simulation are between 40 and 50 seconds.

Dead Time - the amount of time for which an intersection is not carrying traffic in either direction.

Differential - the time interval by which phase B green at one intersection precedes phase B green at the diagonally opposite intersection.

Maximum Contents of a Designated Lane - the maximum number of vehicles encountered in the given lane during the course of the simulation.

Maximum Time to Leave System - the longest total transit time encountered by any vehicle entering from a particular direction.

2.2 Traffic Flow Simulation

Two simulations of traffic flow within the interchange have been developed using the GPSS (General Purpose System Simulation) package. These have been discussed in some detail in Report No. 1 and are only summarized here.

Program A maintains the position and velocity of every vehicle within the interchange area of Figure 4. Each second this information is updated on the basis of a simple car following model in the form of a nonlinear difference equation. Vehicles in program B, on the otherhand, move between intersections in a given or computed travel time. This avoids the necessity of storing and updating position and velocity, and saves thereby substantial storage and computer time. It is more difficult in this case, however, to realistically simulate large variations in traffic volume.

Vehicles moving through an intersection in program A do so according to the car following model. If a vehicle is required to stop, it subsequently starts up at a maximum specified acceleration. The delay in achieving a steady flow is automatically provided by the model. Start up delays must be built into program B, however. This has been accomplished by assigning the time required for each vehicle to pass through an intersection according to Table I.

Table I - Intersection Time

Time from Start of Green Phase	Time Required to "Cross" intersection
0 - 3 seconds	4 seconds
3 - 6 seconds	3 seconds
> 6 seconds	2 seconds

If three or more vehicles are stopped, the first vehicle will cross the intersection 4 seconds after the start of the green phase; the second vehicle will follow 3 seconds later; subsequent vehicles waiting will follow at 2 second intervals. If, however, the first vehicle in the lane arrives at the intersection 3 - 6 seconds after the light turns green, it will require only 3 seconds to cross the intersection, etc.

Each program maintains records and calculates, at the conclusion of the simulation, information such as the average of the total transit times through the interchange, average transit time through each lane, distribution of each transit time, maximum and average contents of each lane, etc. This information is useful for evaluating the performance of different signalling strategies.

Vehicles are generated in both programs by using random numbers to establish the "birth" time of each subsequent vehicle. The inter-generation interval can be given any probability distribution desired. A truncated exponential distribution corresponding to a Poisson process with a one second mean has been used in all simulation studies thus far. An adjustable multiplier is then used for each direction to provide the average inter-arrival time necessary to produce any desired traffic volume.

Program A, because it is less efficient, has been used primarily to check the validity of program B. In general the two programs are in good agreement except when traffic volumes are unusually heavy. In that case, program A gives less encouraging results. Simulation results will be provided for program B only.

2.3 Signal Control

Signals at intersections designated North, West, South, and East are controlled in the GPSS simulation by two transactions which circulate in a loop of instructions. Variations in the signalling strategy are introduced by changing these instructions.

In general, all signal control strategies attempt to allow each vehicle, once it has entered the circulating lanes of Figure 4, to proceed without stopping. Assuming a 15 second travel time between each intersection, this means, for example, that the East phase A green must commence within 15 seconds after the start of South phase B green. Similarly, North phase A green must begin within 30 seconds of the start of South phase B green (and vice-versa). These requirements place rather stringent constraints on the signal sequence. (Stopping vehicles within the circulating lanes is unlikely to produce any benefit except in cases of very short surges in volume.)

Figure 5 is a simplified flow diagram indicating the manner in which traffic signals are controlled in the simulation. At the start of the program, signals are all set at phase A green. Fifteen seconds later, lights at North and South are set at phase B green for 9 seconds. Following the start of phase A green, North and South, there is a 15 second pause, during which platoons entering from North and move past the West and East intersections, respectively. Then a decision is made concerning the starting times and durations of the phase B green at East and West. Fifteen seconds after the termination of phase B green East or West, whichever is first to occur, the starting times and durations of the North and South phase B green are calculated. The process then continues in this manner.

Signal strategies ranging from a fixed cycle to a flexible traffic responsive strategy can be incorporated by changing the procedure with which starting times and durations of the phase B green intervals are calculated. Although Figure 5 is a simulation flow diagram, it also portrays the manner in which a special purpose digital controller might function when supplied with information about queue lengths and short term average traffic volumes measured at the input to each intersection.

Omitted from Figure 5, for simplicity, is the task of setting the intersection crossing times per Table I, as required in program B. (This is required in the simulation but not in actual implementation of the signal strategy.)

III. SIGNAL STRATEGIES

Four signalling strategies have been evaluated. Each of these is designed to virtually eliminate stopping within the interior lanes. Method A is basically that provided by the Traffic Engineering Division of the Connecticut DOT.

Method C - This is a simple strategy in which signals at North and South are synchronous, signals at East and West are synchronous. Phase B green starts simultaneously at North and South and terminates when occupancy lanes North 11 and South 11 are both empty, or after 26 seconds, whichever, occurs first. Allowing for a 4 second amber period, phase A green will commence a minimum of 30 seconds after the start of Phase B green. Fifteen seconds after the start of North-South phase A green, phase B green starts at East - West. A typical phase B sequence is shown in Figure 6 where a fifteen second nominal transit time over interior lanes is assumed.

Method B - Method B adds flexibility to Method C above by permitting North and South (East and West) to use different phase B green intervals. Fifteen seconds after phase A is green at both East and West, the number of vehicles in occupancy lanes North 11, South 11 are observed. If both lanes are full or neither is full, phase B North and South act synchronously as in Method C. If one lane is full (13 vehicles) and the other is not, then a "differential", Δ, is calculated according to

$$\Delta = \begin{cases} 16 - 15\ N/10, & N < 10 \\ 1, & N > 10 \end{cases} \qquad (1)$$

where N is the number of vehicles in the smaller queue. Formula (1) for calculating differential, Δ, conforms relatively well to the assignment of Δ suggested by the Traffic Engineering Division of the Connecticut DOT. Phase B green then starts Δ seconds early for the longer input queue. In Figure 7, for example, South 11 is full, North 11 is not and Δ is calculated as 7 seconds. Phase B green South starts $\Delta = 7$ seconds before phase B green North, and has a duration of $25+\Delta$ seconds. Phase B green North lasts for $25 - \Delta$ seconds. This insures that phase A green North will begin 29 seconds after the start of phase B green South and vice-versa.

Fifteen seconds after both North and South have switched to Phase A green, a similiar calculation is made for the East and West intersections to determine a differential Δ. If $\Delta \neq 0$, then East or West phase B green is delayed Δ seconds, etc.

Note that if the number of vehicles, N., in the smaller queue is two or more and the time required to dissipate an N car queue is 2N + 3 seconds, then it is desirable that

$$\frac{\text{Green time, full queue}}{\text{Green time, shorter queue}} = \frac{25 + \Delta}{25 - \Delta} = \frac{2(13) + 3}{2N + 3}$$

or

$$\Delta = \frac{(26 - 2N)}{32 + 2N} \times 25, \ N > 2 \qquad (2)$$

This is somewhat different from expression (1), but simulation studies indicate no great sensitivity to the choice of Δ.

<u>Method A</u>- An "early start" feature may be added to Method B to provide improvement in some situations. Suppose, for example, that the queue in lane West 11 is depleted E seconds before East 11, and that phase B green West is terminated E seconds before East. 15 seconds after termination of Phase B green West, the North - South differential is calculated. If $\Delta >$ E and favors South, then phase B green South

starts immediately. If $\Delta < E$ and favors South, then phase B green South starts $(E - \Delta)$ seconds later. If $\Delta = o$ or favors North then no early start can be used. Thus, when differential $\Delta = o$, the direction favored may be able to start earlier than in Method B and thereby make more efficient use of the interchange. If $\Delta = o$, no early start is employed.

<u>Method D</u> - Method D is a simple fixed sequence designed to accomodate the given volumes in a more or less optimal fashion. If one examines Figure 7 where a differential $\Delta = 7$ seconds is used, it will be noted that West phase B green can start at 44 seconds, rather than wait until 54 seconds to coincide with phase B green East. Phase A green East starts at 74 seconds, which is 30 seconds after the earlier start of phase B green West, and would permit vehicles entering West to exit without stopping. This observation has been used in Figure 8 which illustrates a sequence similiar to that of Figure 7 with phase B green West advanced. The phase B green durations are approximately in proportion to volumes (900, 400, 500, 300) vehicles / hour respectively for (South, East, North, West).

Observe that the "dead times" in Figure 8 are (0, 4, 9, 15) seconds, respectively, for (South, East, North, West) over an 80 second interval, while in Figure 7 the corresponding dead times are about (10, 20, 10, 28) over an 89 second interval. Equivalently, total phase B green is about the same in both figures, but Figure 8 achieves this in 80 seconds while Figure 7 requires 89 seconds. Thus the scheme in Figure 8 is about 10 percent more efficient for uniform traffic flow. Since the traffic does not arrive uniformly, even in the simulation, it is, of course, questionable whether the fixed sequence of Figure 8 will outperform the more flexible strategy of method C even when traffic volumes are, on the average, those for which Figure 8 was designed.

IV. SIMULATION RESULTS

Tables II - IV summarize performance for three different volume conditions. All simulations are continued until 1000 vehicles exit from the system. This requires about 30 minutes (real time) for the volumes of Table II, 25 minutes for Tables III, IV. Input volumes in Table II correspond to the estimated PM peak volumes of Figure 3. Volumes of Tables III, IV are increased over those of Table II in directions East and/or West.

Tables III and IV, particularly, show the benefits of differential (Method B) over purely synchronous signalling (Method C). Dramatic reductions in average transit time and maximum time to leave system result for the high volume directions at small expense in performance of the low volume directions. Some improvement in the longer transit times also results from the early start feature (Method A). The improvement afforded by the early start feature is somewhat marginal, however, particularly in Table II.

Methods A and D of Table II provide an interesting comparison. The fixed cycle of Method D is clearly superior to the Method A which uses differential and early start. In fact the average transit time for all vehicles is about 84 seconds with Method A, 76 seconds with Method D. Maximum time to leave the system is also superior for the fixed cycle. The implication of this comparison is not that a fixed cycle is best, since the cycle features must be modified in some way as the input volumes vary. Rather, these results indicate that:

(1) An early start feature can be helpful (as in Figure 8) even when volumes are too low to require differential.

(2) There appears to be a significant benefit in terms of average transit times in using (short term) average volumes for calculating the features of phase B green.

There are two ways of employing short term average volumes. One method is to employ an algorithm to design a fixed cycle strategy for any given set of volumes. Every 5 minutes or so this cycle would be modified on the basis of a new set of average volumes. A better proposal is to modify Method A so as to incorporate a more elaborate early start procedure with average volume information. For example, suppose Phase B green West terminates before phase B green East. Fifteen seconds after the termination of phase B green West the following calculations and decisions could be made:

(1) Determine if the queue South 01 can be depleted within 30 seconds after the earliest start of phase B green North. (This will require knowledge of queue length and volume of traffic arriving from the South.) <u>If not</u>, proceed to Step (2). <u>If so</u>, start South phase B green immediately, and continue for the <u>calculated</u> time required to deplete the queue. Start phase B green North 15 seconds after start of Phase A green East.

(2) Use current queue lengths and volumes to determine if phase B green South should be longer than North. If so, use early start feature as in Method A. If not, use differential feature of Method B without early start.

This set of calculations can be expanded further. For example, it may be advantageous to start phase B green South immediately, even if the input queue will not be depleted within 30 seconds after the earliest start of phase B North. That is, it may be better in the long run, depending on average volumes, to terminate phase B green South with vehicles still in the input lanes rather than delay the start of phase B green South as may be required in Step (2).

In order to implement an algorithm of this type fairly complex decisions and calculations must be made. Also, short term average volumes must be maintained. Implementation of the algorithm will, therefore, be most readily accomplished using a special purpose digital or hydrid computer.

The behavior of vehicles in the circulating and input lanes was about as expected. The average transit times in all circulating lanes is about 16 - 17 seconds (including delay in crossing intersection at end of the lane). The distribution of transit times in lanes South 1, South 2, South 3 is shown in Table V. Thus 95 percent or more of the vehicles did not stop on these lanes (i.e. time was less than 20 seconds). A small number - 1 in 70 in South 1, 15 in 290 in South 2, and 4 in 219 in South 3 - were required to stop, but only 8 vehicles (in South 2) required longer than 40 seconds. In all other circulating lanes, the transit time was 30 seconds or less. Thus, only the slow driver who just manages to enter a circulating lane on an amber light is likely to be caught at the next intersection.

Tables VI and VII indicate behavior in the input lanes for the volumes of Figure 3 or Table II. Table VI shows the average transit time for signal methods A and D. Comparison with Table II indicates that the main improvement in Method D over Method A in South and East transit times is due primarily to the reduction in transit times in the input lanes. This, of course, is as expected since circulating lane behavior is essentially the same for all methods. Table VII indicates that the maximum time in the input lane was 80 seconds for Method D, while about 16 percent of the vehicles required longer than 80 seconds with Method A.

Table II

Summary of Results

Signal Schemes: A - flexible scheme with differential and early start

B - Same as A without early start feature

C - Synchronous scheme without differential or early start

D - Fixed 80 second cycle (Figure 8)

	Signal Scheme	Input Direction North	South	East	West
Average Input volume (veh/hr)		500	900	400	300
Average Total Transit Time	A	79.9	90.2	81.8	77.0
	B	79.2	93.8	81.7	76.9
	C	76.1	100.8	76.2	74
	D	80.4	73.3	76.4	75.3
Maximum Time to Leave System	A	130	140	140	140
	B	140	150	130	130
	C	120	170	130	120
	D	130	110	120	130
Maximum Contents of Input Lanes	A	10	22	12	6
	B	10	19	10	8
	C	10	19	8	6
	D	11	14	7	6
Percentage Vehicles Leaving in					
110 Seconds	A	89.4	77.8	82.8	87.9
	B	91.8	72.9	87.4	88.8
	C	97.0	59.9	86.4	93.1
	D	92.0	100.0	95.0	93.0
120 Seconds	A	98.4	92.0	97.1	95.4
	B	96.1	83.7	97.9	98.6
	C	100.0	72.7	98.5	100.0
	D	99.0	100.0	100.0	99.0
130 Seconds	A	100.0	96.7	99.5	99.0
	B	99.5	93.4	100.0	99.2
	C	100.0	83.4	100.0	100.0
	D	100.0	100.0	100.0	100.0

Table III

Summary of Results

	Signal Scheme	Input Direction North	South	East	West
Average Volume (vehicles/hr)		500	900	400	720
Average Total Transit Time	A	81.6	93.3	81.9	88.5
	B	79.2	99.8	80.9	96.0
	C	73.0	105.8	76.3	157.1
Maximum Time to Leave System	A	130	160	130	150
	B	140	170	140	160
	C	120	180	120	260
Maximum Contents of Input Lanes	A	11	25	8	14
	B	11	19	9	17
	C	11	19	8	25
Percentage Vehicles Leaving in					
110 Seconds	A	88.9	71.7	86.6	73.8
	B	87.5	58.2	81.0	62.7
	C	95.5	55.2	93.0	12.9
120 Seconds	A	97.6	79.7	98.8	90.2
	B	95.6	72.3	94.3	78.5
	C	100.0	63.5	100.0	18.6
130 Seconds	A	100.0	87.4	100.0	95.5
	B	98.5	82.8	99.3	90.7
	C	100.0	76.5	100.0	22.2
140 Seconds	A	100.0	93.1	100.0	98.8
	B	100.0	92.8	100.0	94.8
	C	100.0	89.7	100.0	27.2

Signal Schemes: A – flexible scheme with differential and early start

B – Same as A without early start feature

C – Synchronous scheme without differential or early start

Table IV

Summary of Results

	Signal Scheme	North	South	East	West
Average Input Volume (veh/hr)		500	900	720	400
Average Total Transit Time	A	83.8	94.4	94.5	81.2
	B	82.7	96.2	97.4	88.4
	C	78.9	130.6	132.6	81.0
Maximum Time to Leave System	A	160	160	160	150
	B	140	160	160	140
	C	120	220	220	130
Maximum Contents of Input Lanes	A	15	18	18	10
	B	10	18	17	9
	C	11	24	23	8
Percentage Vehicles Leaving in					
110 Seconds	A	83.9	66.8	64.2	80.6
	B	85.3	66.9	69.1	69.2
	C	96.8	21.7	20.2	90.6
120 Seconds	A	91.9	82.6	81.6	92.7
	B	94.6	78.0	79.7	88.1
	C	100.0	30.8	31.7	99.2
130 Seconds	A	96.6	91.1	88.7	98.7
	B	99.5	88.8	88.6	97.2
	C	100.0	42.9	39.3	100.0
140 Seconds	A	97.6	95.4	94.2	99.3
	B	100.0	97.4	94.8	100.0
	C	100.0	55.0	46.3	100.0

Signal Schemes: A - Flexible scheme with differential early start

B - Same as A without early start feature

C - Synchronous schemes (without differential or ready start)

Table V Transit Time Distribution for Lanes

South 1, South 2, South 3

Signal Method A, Volumes as in Table II.

TIME (SECOND)	CUMULATIVE PERCENTAGE (Probability Distribution x 100)		
	South 1	South 2	South 3
10	0	0	0
20	98.5	95.1	98.1
30	100	97.2	100
40	100	97.2	100
50	100	100	100

Table VI Average Transit Time in Input Lanes

Volumes as in Table II

LANE	Average Transit Time (Seconds)	
	METHOD A	METHOD D
NORTH 01	48.3	46.6
NORTH 02	46.2	43.7
WEST 01	48.4	42.8
WEST 02	42.4	39.7
SOUTH 01	58.4	39.4
SOUTH 02	56.8	37.2
EAST 01	51.7	42.5
EAST 02	47.2	38.6

Table VII Transit Time Distribution in SOUTH 01

Volumes as in Table II

TIME (SECONDS)	Cumulative Percentage SIGNAL METHOD A	SIGNAL METHOD D
30	8.7	33.4
40	21.3	48.2
50	35.3	66.9
60	48.0	86.0
70	68.9	99.1
80	83.8	100.0
90	92.5	
100	96.5	
110	100.0	
AVERAGE	58.4 Seconds	39.4 Seconds

V. SUMMARY

Simulation of left turning traffic in the Route 15 - I-291 diamond interchange indicates that the interchange can efficiently service the peak PM traffic volumes projected for 1990. Average transit times through the interchange are typically 80 seconds, with about 50 seconds required in travel time, 30 seconds required in waiting at the initial intersection. In most cases, vehicles do not need to stop once the circulating lanes are entered. Even the higher volumes of Tables III and IV do not cause serious delays.

The superior performance of Method D in Table II has suggested further refinements in the signalling scheme purposed by the Traffic Engineering Division. These are outlined in the previous section. Signal control algorithms based on these ideas can be developed, simulated, and tested. However, implementation will likely require a digital or hybrid computer. This, however, is a trend in signal control at major intersections.

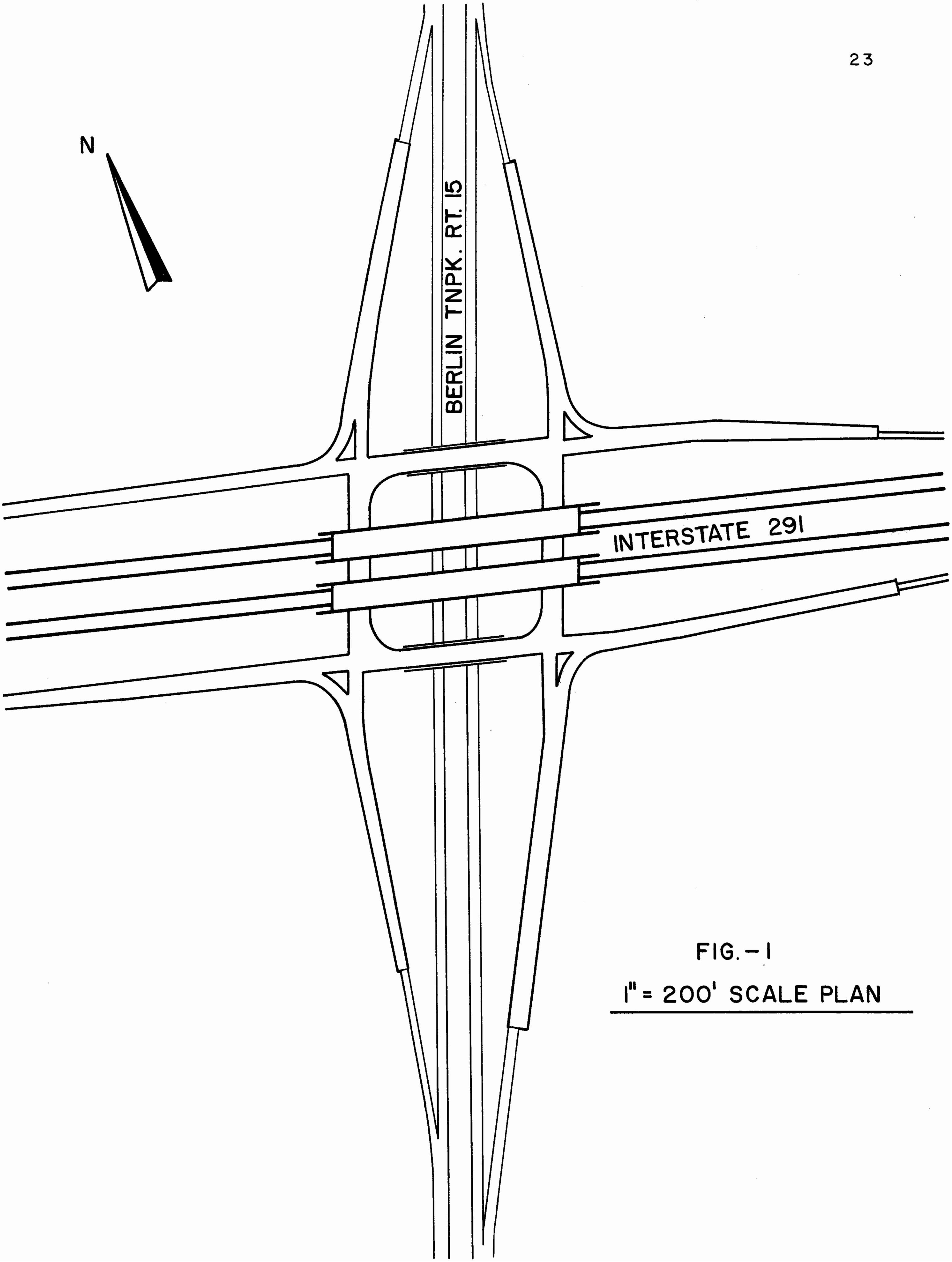


FIG. – 1

1" = 200' SCALE PLAN

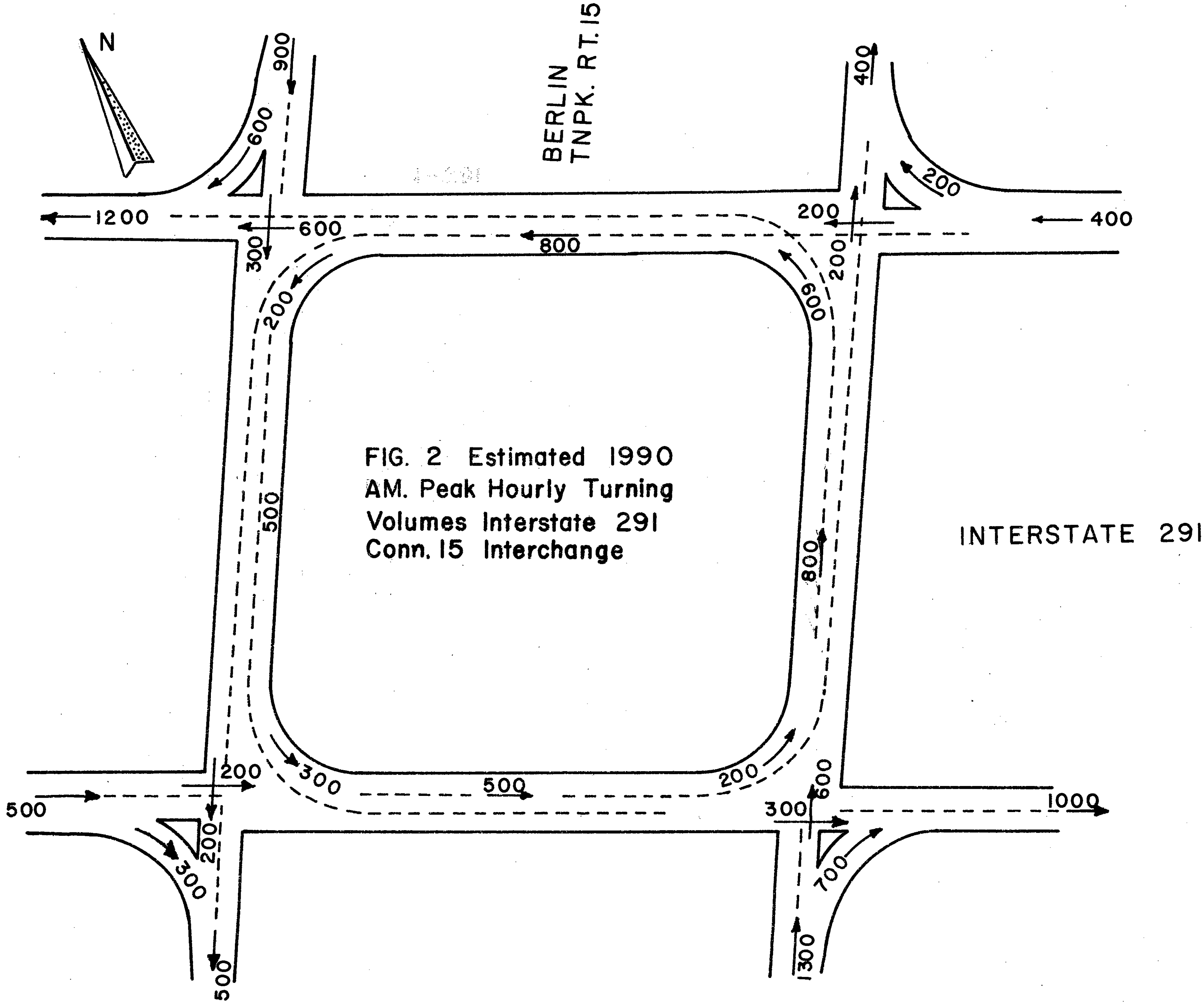


FIG. 2 Estimated 1990 AM. Peak Hourly Turning Volumes Interstate 291 Conn. 15 Interchange

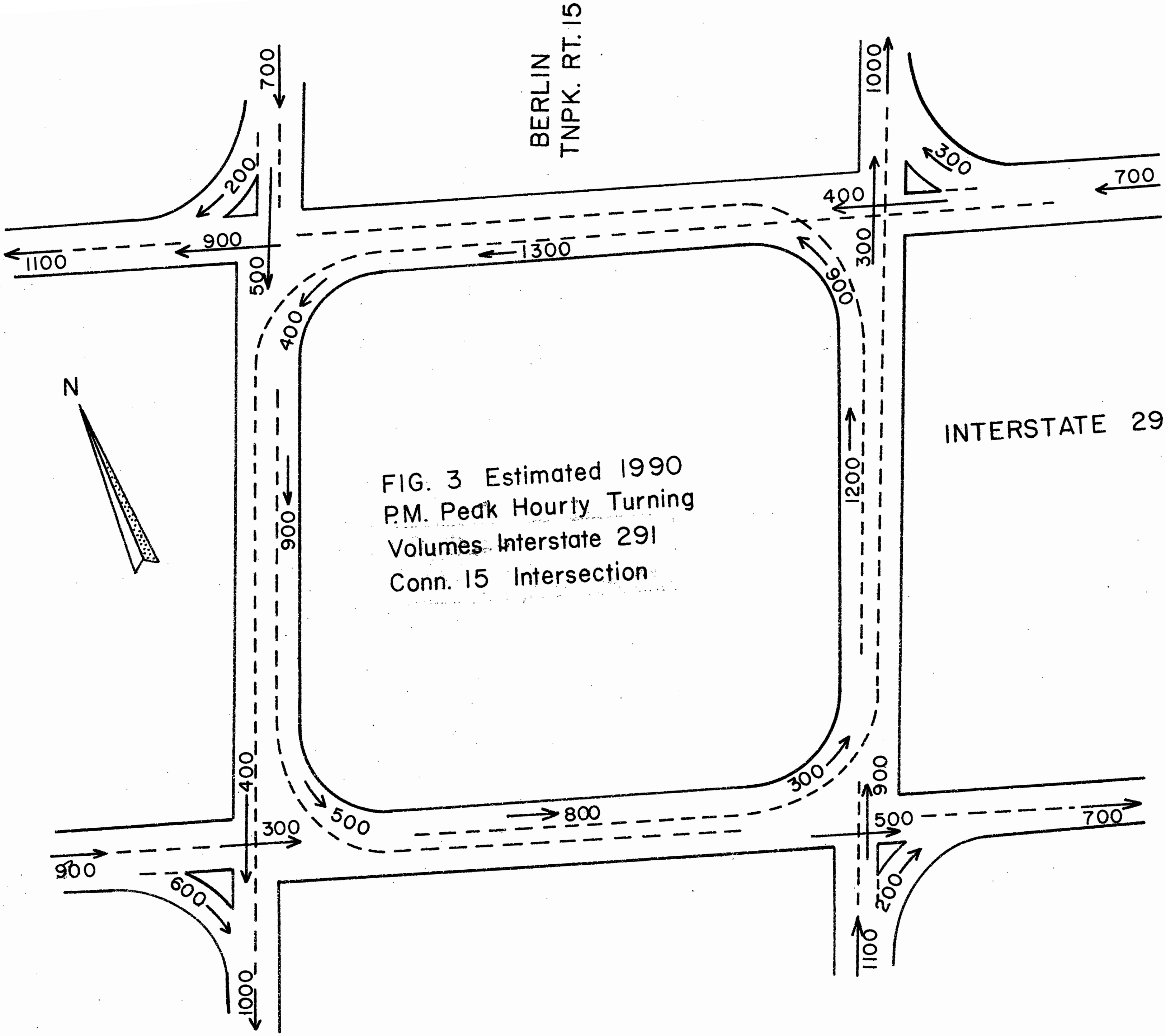


FIG. 3 Estimated 1990
P.M. Peak Hourly Turning
Volumes Interstate 291
Conn. 15 Intersection

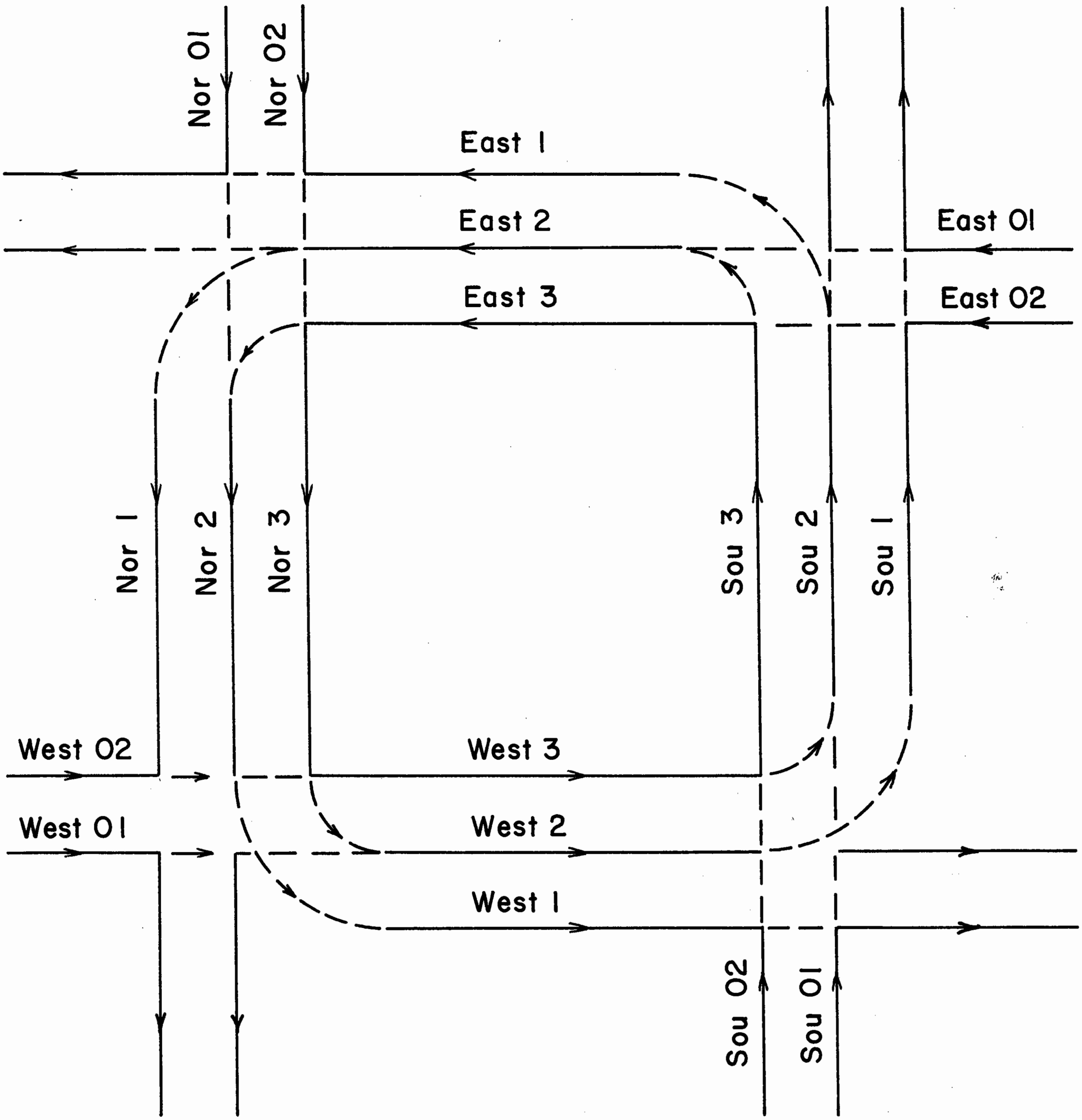


FIG. - 4 Lane Designations

Figure 5

Basic Signal Control Simulation

SGS = Time to start of Phase B green, South

SGE = Time to start of Phase B green, East

GS = Duration of Phase B green, South

GE = Duration of Phase B green, East

Etc.

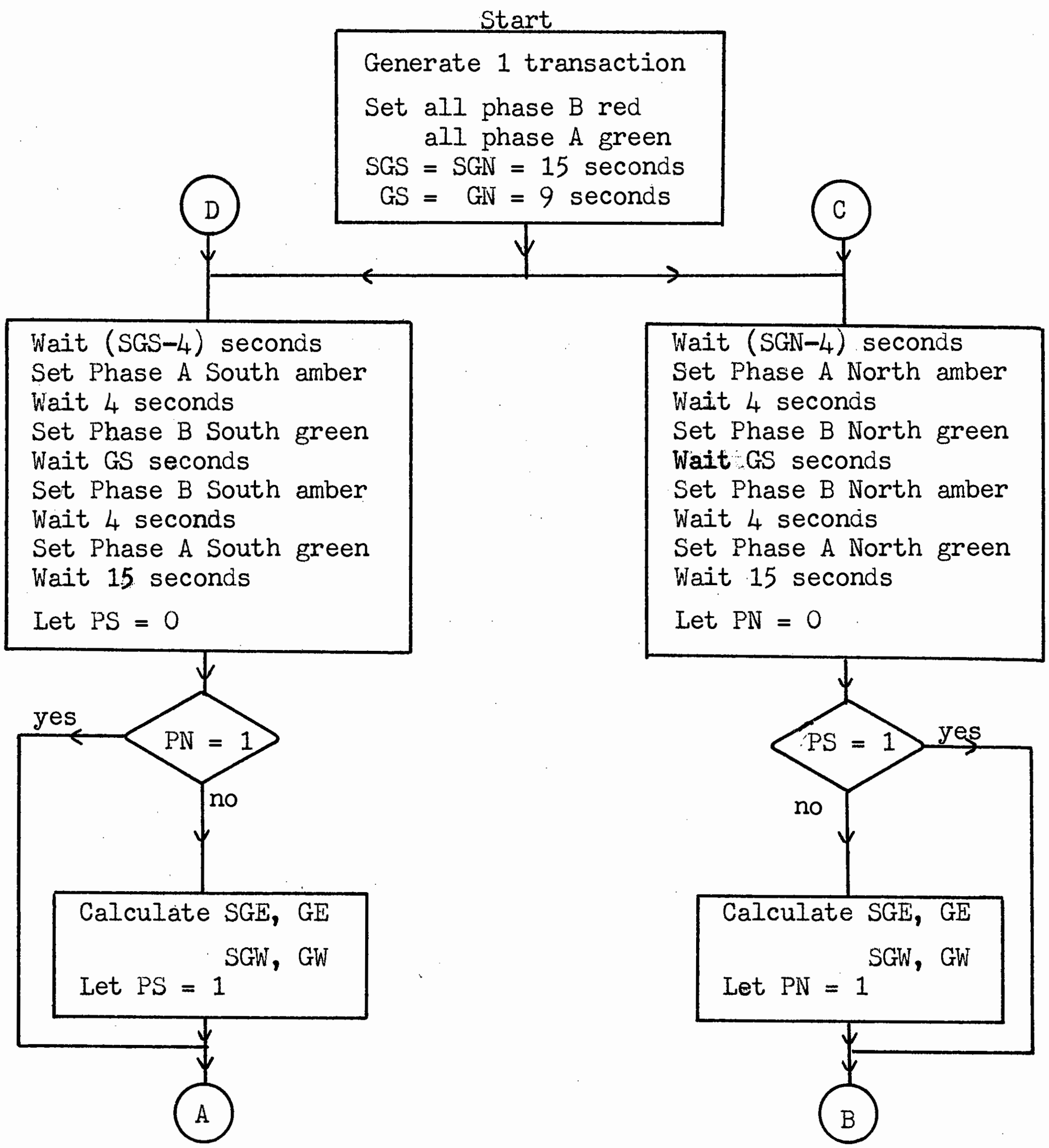
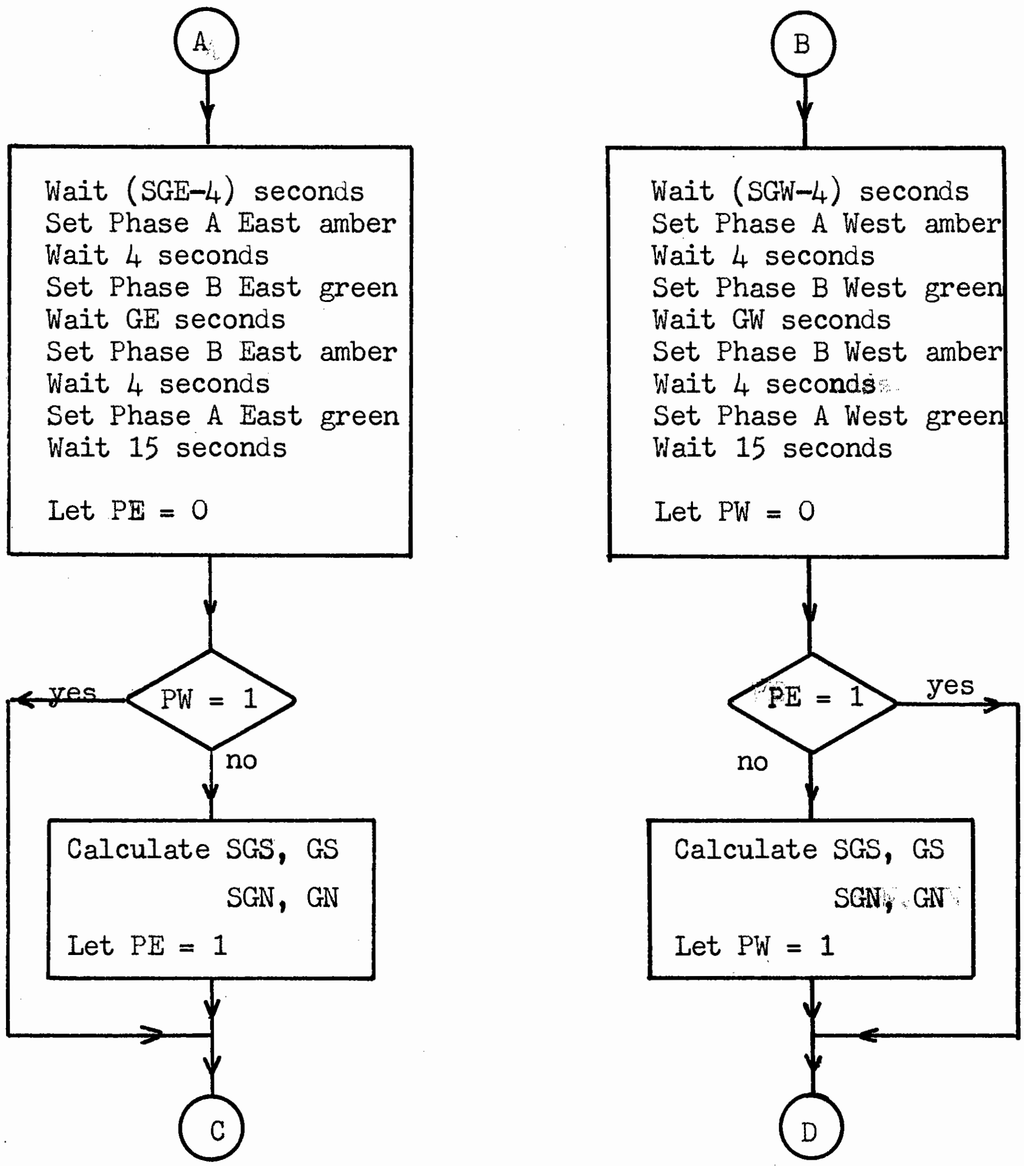


Figure 5 Continued



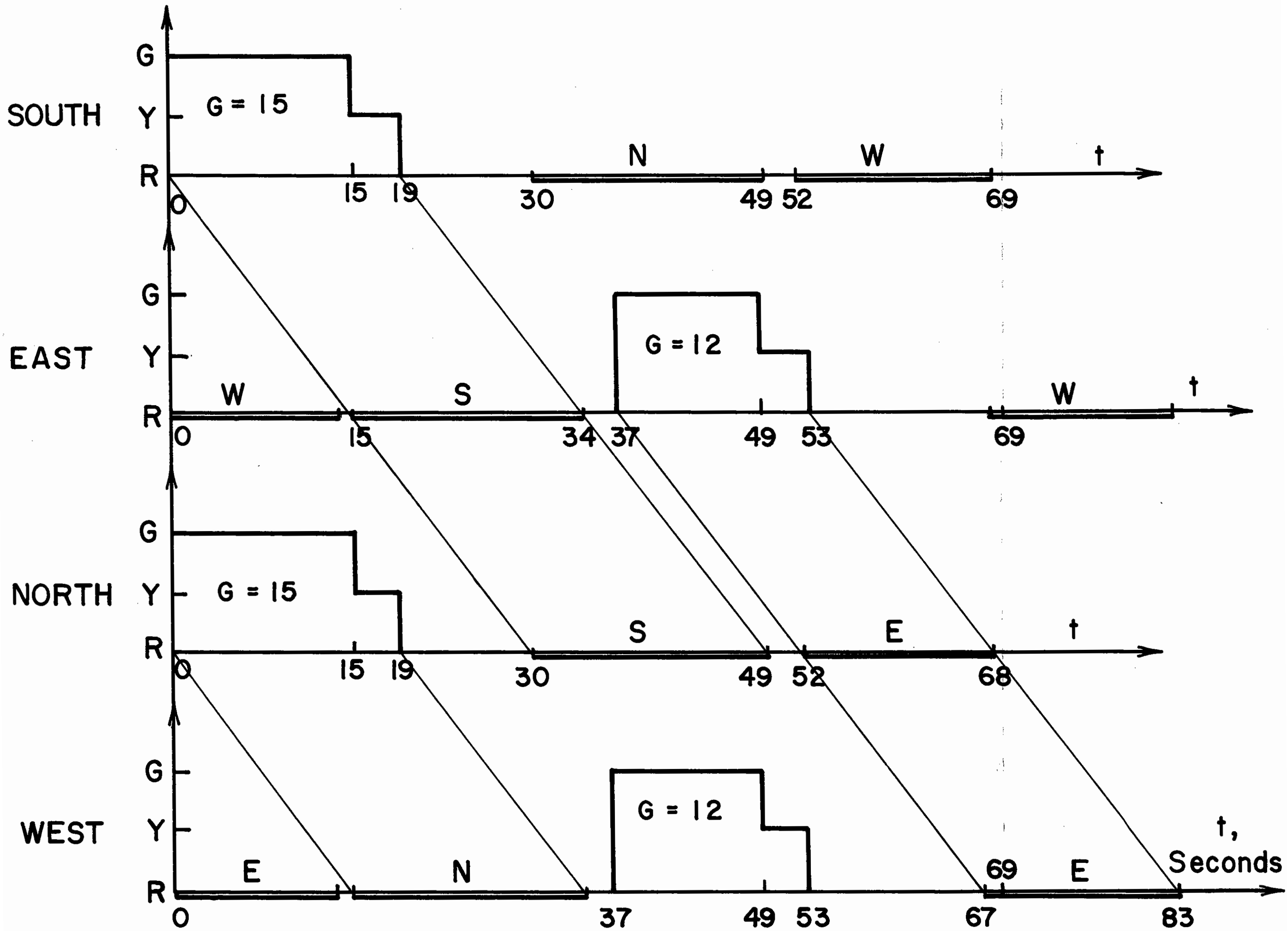


Fig. —

Typical Phase B Signal Sequence Where Differential (Δ) Is Not Required

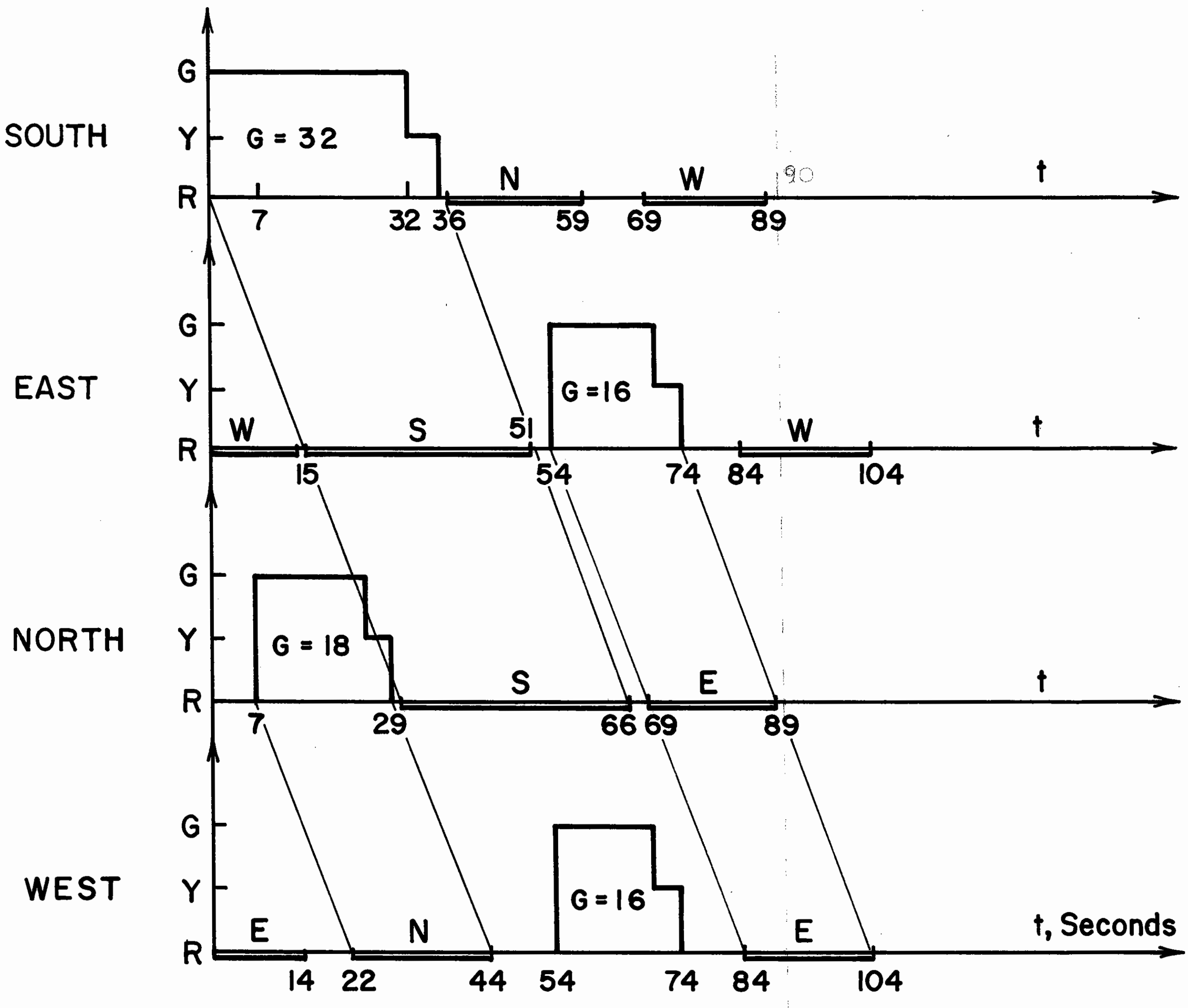


Fig.-

Typical Phase B Signal Sequence Where Differential (Δ) of 7 Seconds Is Required For South

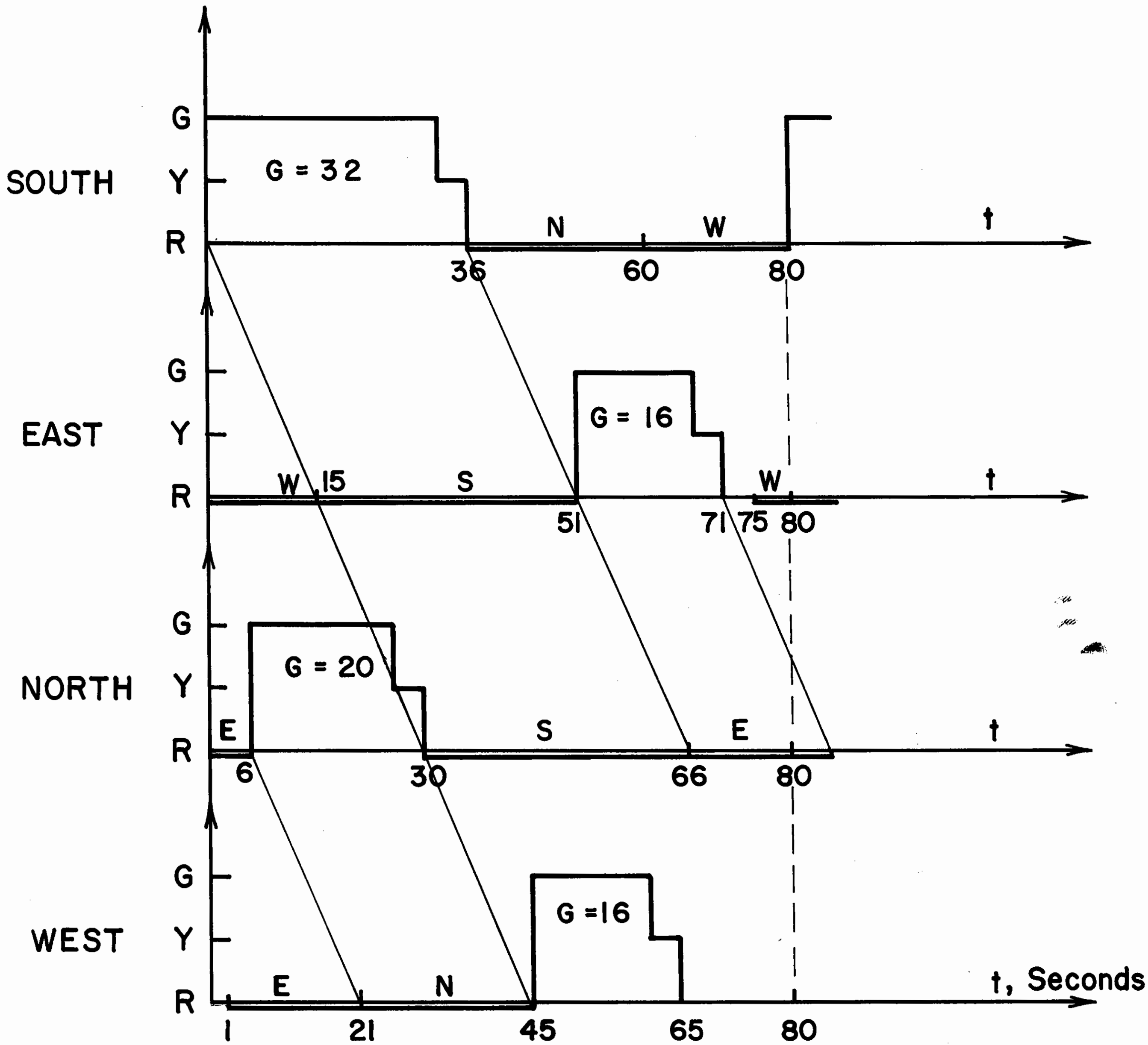


Fig. -

Phase B Signal Sequence For Fixed 80 Second Cycle