OPTIMIZATION TECHNIQUES APPLIED TO IMPROVING FREEWAY OPERATIONS

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This paper describes optimization techniques that have been developed and applied for evaluating freeway operational improvements such as redesign or ramp control strategies. First, a deterministic macroscopic simulation model is described that predicts the traffic performance on a directional freeway as a function of freeway design and traffic demand patterns. Then two decision models are presented that automatically work with the simulation model to select optimum redesign or ramp control strategies. Finally, a freeway corridor model is described. Emphasis is given to the structure of the model and to the first step in the development of the freeway corridor model, which is a major arterial street model.

• THIS PAPER reports on an investigation of various operational aspects of urban freeways, including the application of optimization techniques for improving freeway operations through redesign or control strategies. The first phase of the study began in October 1967 and continued until December 1968. It inventoried existing traffic conditions on approximately 140 directional miles of freeway in the San Francisco Bay area. It also identified critically congested sections, ascertained their cause, and estimated their effect on traffic operations. In addition, preliminary investigations were conducted to study means of improving identified critical sections and to prepare preliminary estimates of user benefits. Eight interim reports and a final summary report were published (1-9).

The second phase began in early 1969. It analyzed in detail two selected portions of the freeway system by using a systematic analytical procedure for evaluating design and control improvements. This procedure was computerized, and the program was named FREEQ. The results are covered in three project reports (10-12).

The third phase began in January 1972 and extended through June 1973. It studied four major areas: (a) freeway model refinement, (b) systems analysis, (c) control strategies, and (d) network flow. Six interim reports that cover the initial research work completed as of June 1972 were prepared (13-18). Efforts were continued during 1972 and 1973, and four reports (one in each area of research) were published in June 1973 that contained the most significant findings of the six interim reports plus the results of the extended work (19-22).

This paper attempts to summarize the research work completed and emphasizes the application of optimization techniques for improving freeway operations through redesign or control strategies. During the third phase of the study, research efforts considered two levels of activity: (a) only the freeway and (b) the freeway corridor. The first level research activity involves the development of analytical techniques for evaluating freeway design or ramp control improvement plans on a freeway-only basis. The foreground of Figure 1 shows a schematic representation of this freeway evaluation process.

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Figure 1. Optimization techniques for improving freeway operations.

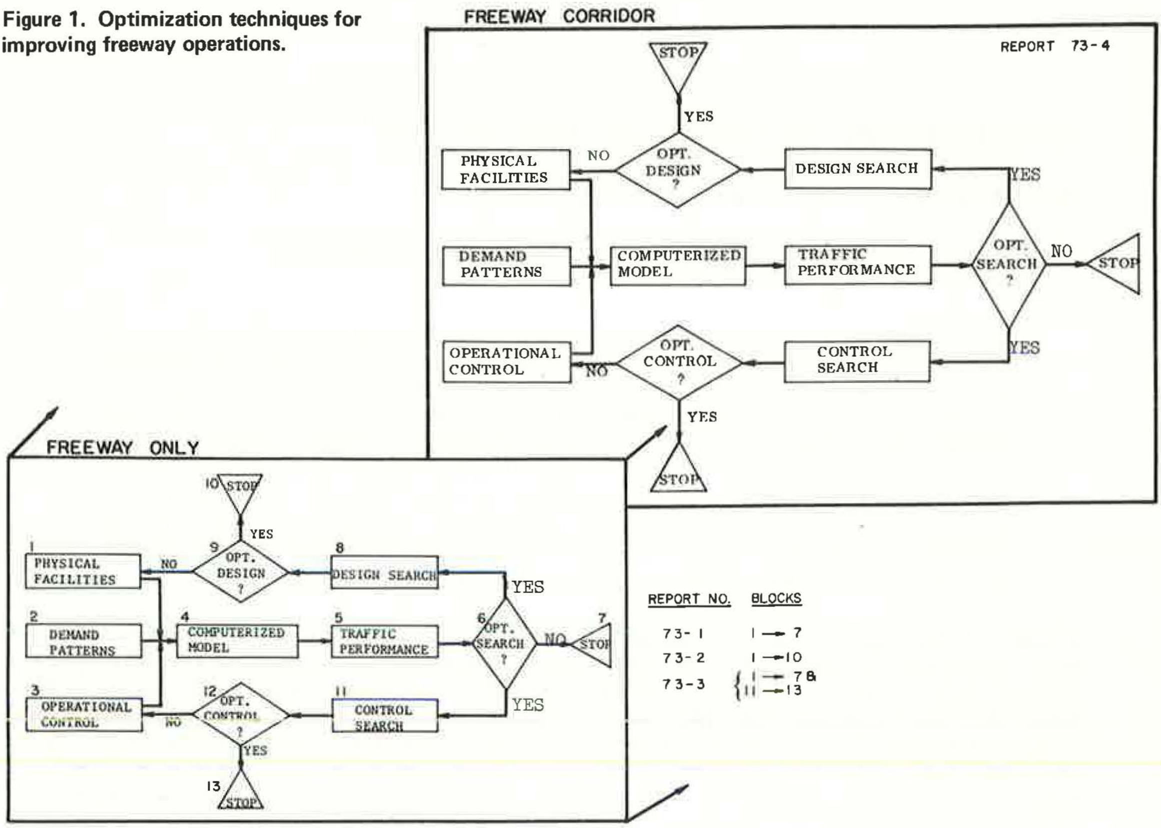
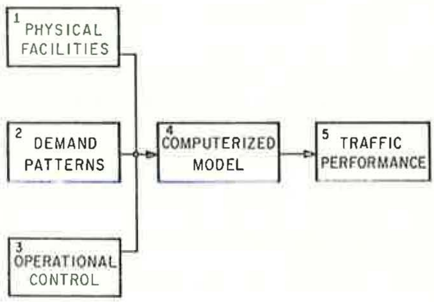


Figure 2. The FREQ3 freeway model.



FREQ3 FREEWAY MODEL

The FREQ3 freeway model (19) is a deterministic simulation model that predicts traffic performance on a directional freeway as a function of freeway design and traffic demand patterns. This is a third-generation version of the FREEQ model and is in essentially final form pending its ultimate incorporation into a freeway corridor model.

A schematic representation of the FREQ3 process is shown in Figure 2. The model consists of three main parts: (a) input (freeway design and traffic demand patterns), (b) FREQ3, and (c) output (freeway traffic performance).

Model Input

Input to FREQ3 consists of freeway design and traffic demand patterns, and Figures 3 and 4 show sample listings of such input data.

The directional freeway section is divided into subsections. A subsection is a portion of the freeway section over which the capacity and demand are essentially constant. In other words, a boundary exists between subsections at each on-ramp and off-ramp because of demand changes and at each location where the freeway capacity changes, such as at lane drops, lane additions, or significant grade changes. Pertinent information that is needed for each subsection includes subsection number, number of lanes, length, "straight-pipe" capacity, truck factor, inflow and outflow station locations, special ramp indicator, and location description. A listing of this input data is shown in Figure 3, and represents a directional freeway that is approximately 10 miles (16 km) long and has been divided into 30 subsections.

The study period is divided into equal time slices. A traffic demand pattern is specified for each time slice in the form of an origin and destination table. It is assumed that the arrival demands during the time slice are uniform. The length of the time slice can be selected by the user; time slices of 15 minutes appear to be reasonable. The number of origin and destination input tables is equal to the number of time slices. These tables may be based on results of origin and destination studies or can be estimated from ramp counts. Figure 4 shows the traffic demand pattern for one time slice that serves as input into the model.

Model Description

FREQ3 is described in two parts: the model structure and its computerized program.

The structure of the FREQ3 model can best be described by considering a distance-time diagram (Fig. 5). The horizontal scale is distance, with traffic flowing from left to right. The plan view of the directional freeway section is shown below the horizontal scale and has been divided into subsections. The vertical scale is time of day and is divided into equal time slices. A cell is defined as having a length of one subsection and a time interval of one time slice.

The design features of the various subsections are translated into capacities, and these capacity values C_{ij} can be thought of as flowing upward through the distance-time diagram. The origin and destination tables (one for each time slice) are translated into subsection demands, and these demand values D_{ij} can be thought of as flowing to the right through the distance-time diagram. Initially it is assumed that there are no bottlenecks, but later this is checked and, if necessary, adjustments are made.

The procedural analysis of FREQ3 begins with the cell in the lower left-hand corner of Figure 5 that represents the first subsection during the first time slice. Then the cells immediately to the right are analyzed until all cells in the first time slice have been analyzed. Then the cell that represents the first subsection during the second time slice is analyzed, followed by the cells immediately to the right. This procedure is continued from time slice to time slice, until all cells are analyzed.

The analysis in each cell consists of comparing the C_{ij} value with the D_{ij} value. Two outcomes are possible: either $D_{ij} \le C_{ij}$ or $D_{ij} > C_{ij}$. If $D_{ij} \le C_{ij}$, then this subsection is not a bottleneck, and the flow V_{ij} is equal to D_{ij} is equal to D_{ij}. The flow-capacity ratio can be calculated, and from this ratio, speeds and travel times are estimated. However, if $D_{ij} > C_{ij}$, then this subsection is a bottleneck and a more elaborate analytical procedure is required.

Figure 3. Freeway design input to FREQ3.

SUB SEC NO.	NO. LANE	CAP.	LENG. FT.	TRK. FACTOR			SUBSECTION DESCRIPTION		
1	5	9434	1630	.943	OD		MAINLINE ORIGIN	TO	POWELL OFF
2	5	9453	1960	.961			POWELL OFF	TO	POWELL ON
3	5	9615	1550	.961	OD		POWELL ON	TO	ASHBY OFF
4	4	7619	1960	.952			ASHBY OFF	TO	ASHBY ON
5	4	7550	500	.952	O		ASHBY ON	TO	500 FEET PT.
6	4	7550	4790	.952	D		500 FEET PT.	TO	UNIVERSITY OFF
7	4	7692	3030	.961			UNIVERSITY OFF	TO	UNIVERSITY ON
8	4	7692	2160	.961	OD		UNIVERSITY ON	TO	GILMAN OFF
9	4	7563	2030	.961			GILMAN OFF	TO	GILMAN ON
10	5	9130	1250	.970	OD		GILMAN ON	TO	BUCHANAN OFF
11	4	7619	900	.952	D	1	BUCHANAN OFF	TO	HOFFMAN OFF (LEFT)
12	3	5671	1320	.961	D		HOFFMAN OFF	TO	PIERCE OFF
13	3	5750	720	.961			PIERCE OFF	TO	PIERCE ON
14	3	5806	2610	.970	OD		PIERCE ON	TO	CENTRAL OFF
15	3	5728	1660	.970			CENTRAL OFF	TO	CENTRAL ON
16	3	5806	1890	.970	OD		CENTRAL ON	TO	CARLSON OFF
17	3	5520	2310	.940			CARLSON OFF	TO	CARLSON ON
18	4	7930	1460	.980	OD		CARLSON ON	TO	POTRERO OFF
19	4	7680	3800	.970			POTRERO OFF	TO	CUTTING ON
20	5	9140	1100	.980	O		CUTTING ON	TO	GRADE CHANGE PT.
21	5	9230	660	.980	D		GRADE CHANGE PT.	TO	MACDONALD OFF
22	4	7930	1480	.980	D	0	MACDONALD OFF	TO	SAN PABLO OFF
23	4	7840	1480	.970			SAN PABLO OFF	TO	SAN PABLO ON
24	5	8840	800	.980	OD		SAN PABLO ON	TO	SOLANO OFF
25	4	7604	4690	.970	D		SOLANO OFF	TO	SAN PABLO DAM OFF
26	4	7760	2190	.970			DAM ROAD OFF	TO	DAM ROAD ON
27	4	7680	2320	.970	OD		DAM ROAD ON	TO	ROAD 20 OFF
28	4	6732	830	.850			ROAD 20 OFF	TO	GRADE CHANGE PT.
29	4	6480	1180	.793			GRADE CHANGE PT.	TO	ROAD 20 ON
30	4	6265	2560	.780	OD		ROAD 20 ON	TO	MAINLINE DESTINATION

ALL RAMPS LIMIT = 1500 EXCEPT OFF-RAMP 6 LIMIT = 2200

Figure 4. Traffic demand pattern input to FREQ3.

```
TIME SLICE 1  FB72N
1.38     =     OCCUPANCY FACTOR
 144 198 227  54 108 252  10  88  25  37  15  40  17  80  44 220
           4       5   5           9  22   5   5   2   9   5  24
               7      20       7  12      19   3   3  13   7  36
                      41          11   7  11       4  16  11  48
                   7  31       4   4   8      12   4  17  10  53
                                   2   4   2       1   6   3  14
                                          13       3  11   6  31
                                           6       2   8   4  22
                                              17  21  56  20  74
                                                   9  34  21 102
                                                           9  46
                                                              44
TIME SLICE 2  FB72N
 1.38
 183 229 232  76  99 325   7  88  46  26  30  69  10  53  28 334
       2   3   2   8  12       2   2   8           3  12   5  36
           5   2  13  23      10   3  17   9  11   9  33  17 103
                      43      19   8  17   8   2   6  25  14  81
                  13  16   3  13  10  19  16       6  26  15  86
                                           3       3   9   4  23
                                   3  10   6   6   4  14   8  44
                                       3   5   3   3  11   6  33
                                                  33  55  32 119
                                                  13  39  23 124
                                                          10  56
                                                              44
TIME SLICE 3  FB72N
 1.38
 183 226 214  42 161 314   9 123  51  39  40 112  19  82  33 213
       5       9      14       9      14   5       3   8   8  31
               3      20       9   4   6   7   2   7  21  11  69
                      51       4   7  27   2       9  22  12  73
                  16  12       7   7  31   2       7  22  12  75
                                           2       1   6   3  20
                                   2   5   2   2   3  10   4  36
                                       2   5       6  16   7  49
                                                  33  95  27 115
                                                  13  39  19 128
                                                           9  57
                                                              44
TIME SLICE 4  FB72N
 1.38
 203 191 171  45 129 215   4  71      76  33  67   5  27  10 562
               4   6  19   4  16                   4  12   7  40
           2       7  58   2  15  10  17  12  28  10  33  19 103
                      34       4  11  11  20  13  12  41  24 126
                  20  83      20  48  21      10   6  20  12  66
                                   2           2   3   9   5  30
                                       3   6   9   4  19   7  39
                                       7   3       5  14   9  44
                                                  34 108  40 153
```

In this more elaborate case four steps are required: First, inasmuch as $D_{ij} > C_{ij}$, then V_{ij} is equal to C_{ij}, not D_{ij}. Second, because $(D_{ij} - C_{ij})$ vehicles are being stored upstream of this bottleneck, the previously predicted downstream demands must be reduced. Third, the backward-moving shock wave is determined, and new flow and travel time situations upstream of the bottleneck are recalculated. Finally the excess demand at this bottleneck during the time slice $D_{ij} - C_{ij}$ is added to the origin and destination table for the next time slice.

This procedure becomes extremely complicated when several bottlenecks occur, and the resulting queues may collide and split at different times and at different locations. Of course, the decreasing queuing situation, which was not described above, is also handled (19).

FREQ3 has been computerized and is written in FORTRAN IV for the University of California's CDC 6400 computer. The computer program consists of the main program, which essentially is a "calling" program, 17 subroutines, and one function. A flow chart of the computer program is shown in Figure 6.

The FORTRAN program deck consists of approximately 2,000 statements. The computer takes approximately 4 seconds to run the FREQ3 program, which includes 10 miles of congested freeway (30 subsections) and a 2½-hour period (ten 15-minute time slices). The computer charge is approximately $1.70.

The computer program results have been calibrated with real-world data obtained from the northbound East Shore Freeway in the San Francisco Bay area.

Model Output

Output from FREQ3 includes speeds, densities, flows, and travel times for each cell; individual trip times and total travel times for each time slice; and total travel times and travel distances for the entire study section during the study period.

The user may request different detail of output information depending on the particular problem. A sample of the basic detail that is provided with every output is shown in Figure 7. This sample output is for one time slice, and one such output is provided for the user for every time slice in the study period.

COST-EFFECTIVENESS EVALUATION OF FREEWAY DESIGN ALTERNATIVES

This section describes an extension of the FREQ3 freeway model, called the FREQ3D design model and demonstrates its application for evaluation, on a cost-effectiveness basis, of alternative design improvements on the northbound East Shore Freeway (20). This is a second-generation version of the FREQ3D design model and is considered to be in essentially final form pending its ultimate incorporation into a freeway corridor model.

A schematic representation of the FREQ3D process is shown in Figure 8. The model consists of the FREQ3 with an iteration procedure for evaluating preselected design improvement alternatives (a design improvement alternative consists of adding a lane at a bottleneck location). The iterative procedure consists of two major tasks: (a) generation of design alternatives and (b) evaluation of design alternatives. Before these two major tasks are described, the structure of the problem will be introduced.

Problem Structure

The directional freeway can be thought of as a sequence of facilities, represented by each subsection, designed to handle arriving traffic. Each subsection acts essentially as a service facility serving a queue. The whole system behaves like a series of queues being processed through a series of service facilities.

Two important features of the system can be identified from this description. First, the travel times are nonlinear functions of the flow-capacity ratio. The result is that improvements in the system are nonlinear functions of investment in capacity. The second important feature is the highly interactive nature of a series of queues. This leads to the realization that the value of improvements in one subsection is not independent of the improvements in other subsections.

Figure 5. Distance-time diagram for FREQ3.

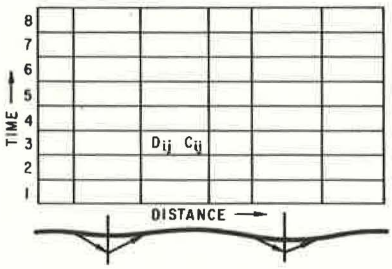


Figure 6. Flow chart of FREQ3 computer program.

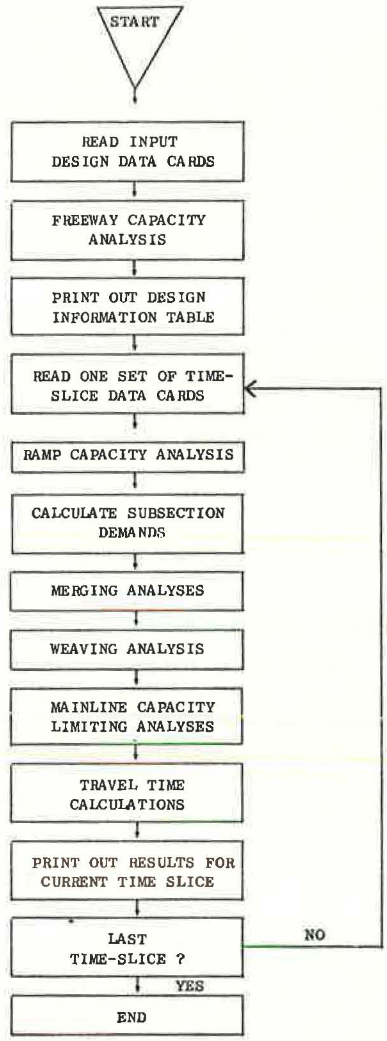


Figure 7. Sample output of FREQ3 computer program.

TIME SLICE 4 F872C/C 3

OCCUPANCY 1.38

QUEUE COLLISION 8 T2= .118

SUB SEC	..INPUT DEMAND... ORG	DES	TOTAL	VOL	FWRY CAP	WEAVE EFF	V/C	DENS V/M/L	MPH		TRAV TIME	LENG	QUEUE FEET	N-PRIME
1	7236	812	7236	7236	9434	0	.77	25	56		.33	1630	0	0
2	0	0	6424	6424	9453	0	.68	22	57		.39	1960	0	0
3	448	764	6872	6872	8809	806	.78	24	56		.31	1550	0	0
4	0	0	6108	6013	7619	0	.79	34	43	*	.52	1960	592	95
5	1264	0	7372	7277	7277	273	1.00	51	35		.16	500	0	0
6	0	692	7372	6064	7277	273	.83	45	33	*	1.94	4790	4330	1213
7	0	0	6680	5382	7692	0	.70	51	26	*	1.43	3030	3029	1213
8	1184	156	7864	6566	7399	293	.89	63	26	*	.96	2160	2159	833
9	0	0	7668	6384	7563	0	.84	69	23	*	1.00	2030	2029	833
10	1224	648	8852	7608	7608	1522	1.00	43	35		.40	1250	0	0
11	0	1636	8244	7061	7342	277	.96	32	54		.19	900	0	0
12	0	40	6608	5646	5671	0	1.00	49	38		.39	1320	0	0
13	0	0	6568	5602	5750	0	.97	41	45	*	.18	720	137	10
14	204	504	6772	5806	5806	0	1.00	55	35		.84	2610	0	0
15	0	0	6268	5376	5728	0	.94	32	55		.34	1660	0	0
16	348	284	6616	5724	5806	0	.99	43	44		.48	1890	0	0
17	0	0	6332	5451	5520	0	.99	42	43		.60	2310	0	0
18	240	512	6872	5691	5831	119	.98	38	49		.33	1460	0	0
19	0	0	6060	5249	5806	0	.90	31	55		.78	3800	0	0
20	900	0	6960	6149	6866	454	.90	27	55		.23	1100	0	0
21	0	284	6960	6145	6936	454	.89	27	55		.14	660	0	0
22	0	516	6676	5505	7930	0	.74	26	56		.30	1480	0	0
23	0	0	6160	5460	5728	0	.95	33	55		.31	1480	0	0
24	576	292	6736	6036	6331	519	.95	27	55		.17	800	0	0
25	0	868	6444	5761	5800	0	.99	48	40		1.33	4690	0	0
26	0	0	5576	4960	5806	0	.85	30	55		.45	2190	0	0
27	264	580	5840	5224	5800	0	.90	31	55		.48	2320	0	0
28	0	0	5260	4675	5049	0	.93	28	55		.17	830	0	0
29	0	0	5260	4679	4746	0	.99	35	44		.30	1180	0	0
30	0	5260	5260	4679	4700	0	1.00	41	38		.75	2560	0	0

	CURRENT TIME INTERVAL	CUMULATIVE VALUES
FREEWAY TRAVEL TIME=	394. VEH-HRS= 544. PASS-HRS	1145. VEH-HRS= 1580. PASS-HRS
INPUT DELAY=	0. VEH-HRS= 0. PASS-HRS	0. VEH-HRS= 0. PASS-HRS
TOTAL TRAVEL TIME=	394. VEH-HRS= 544. PASS-HRS	1145. VEH-HRS= 1580. PASS-HRS
TOTAL TRAV DISTANCE=	16015. VEH-MI.= 22100. PASS-MI.	57710. VEH-MI.= 79640. PASS-MI.

As a result of these factors, the most common optimization techniques are not suitable. Linear and piece-wise linear programming are not appropriate because of the nonconvexity of the objective function. Dynamic programming and marginal analysis are not appropriate because of the interdependence between the subsections. Complete enumeration is not possible because of the large number of alternatives (between 2^{30} and 3^{30}).

Fortunately some practical considerations aid in limiting the number of alternatives and, more importantly, in simplifying the structure of the problem. First, all desired improvements cannot be built simultaneously, nor is it likely that previous improvements would be destroyed. Consequently, this limits the procedure to a unique sequence of design improvement stages. Second, it is unlikely that an operating agency would improve a nonbottleneck subsection while a bottleneck subsection exists nearby (even if it were shown to be more cost-effective). Consequently, this significantly limits the number of alternatives to be investigated at each stage of improvement. The net result of these practical considerations is a sequence of stages that give the appearance of a branch and bound mathematical programming technique combined with comparing alternatives at each stage in a marginal analysis manner.

Generation of Design Alternatives

The procedure employed in generating design alternatives consists of (a) identifying bottleneck subsections from the previously selected stage of improvement and (b) formulating a design improvement alternative for each identified bottleneck.

The output from the FREQ3 freeway model (this model is actually a submodel in the FREQ3D freeway design model), which represents the previously selected stage of improvement (Fig. 7), is inspected. Any subsection in any time slice that exhibits a flow-capacity ratio of unity is identified as a bottleneck subsection.

Then, a design improvement alternative is formulated for each bottleneck subsection. Only feasible lane arrangements are considered so that good highway design practice is followed. A lane improvement plan is feasible if it satisfies the following constraints:

1. The number of lanes downstream of an off-ramp must either be equal to or be one less than the number of lanes upstream of the off-ramp,
2. The number of lanes downstream of an on-ramp must either be equal to or be one more than the number of lanes upstream of the on-ramp,
3. Only one lane can be added or dropped at a subsection boundary, and
4. The maximum number of lanes permitted in a subsection is six.

Evaluation of Design Alternatives

The procedure used in evaluating design alternatives consists of (a) calculating the annual cost (in dollars) of each design alternative and its measure of effectiveness (savings in passenger-hours) and (b) selecting the design alternative, at this stage, that exhibits the smallest marginal cost-effectiveness (dollars per passenger-hour saved).

The estimated cost of adding a lane to each subsection and parameters such as interest rate, improvement life, and maintenance cost are placed as input to FREQ3D. The model is then used to calculate the annual cost of each design alternative. The freeway model calculates the total passenger-hours expended with and without the design improvement, and the difference in passenger-hours reflects the savings due to the design improvement.

The marginal cost-effectiveness is calculated for each design alternative at this stage, and the design alternative that has the smallest cost-effectiveness is selected. This leads to the next stage of improvement, and a new series of design alternatives is generated.

Numerical Results

The design alternative method was used to calculate the cost-effectiveness curve for the northbound East Shore Freeway. The successive stages of improvements from the existing freeway conditions to noncongested freeway conditions are shown in block-diagram form in Figure 9 and in cost-effectiveness form in Figure 10.

Figure 8. The FREQ3D freeway design model.

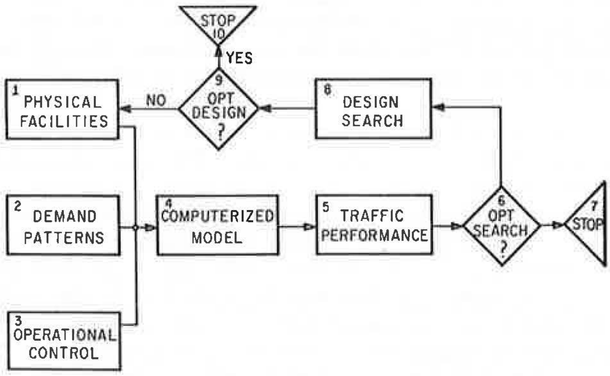
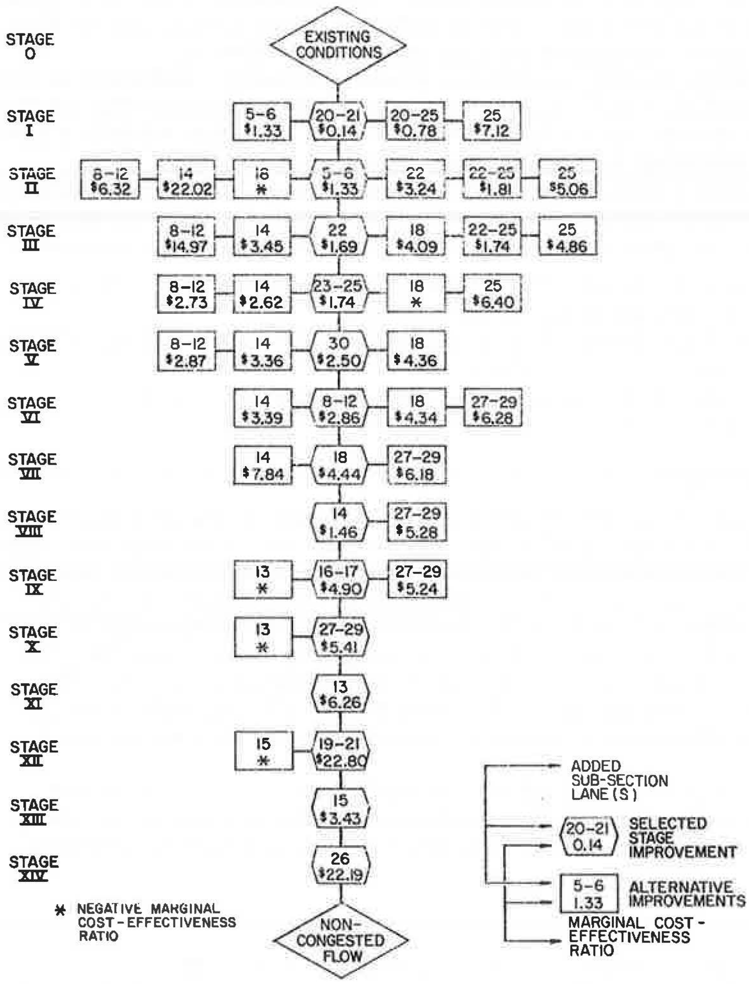


Figure 9. Block diagram of most cost-effective method applied to northbound East Shore Freeway.



Numerous attempts have been undertaken to improve this approximation to the optimal solution (15, 20). One attempt proceeded as in the previously described manner, but at each stage the most effective rather than the most cost-effective design alternative was selected. Another attempt used study teams consisting of experienced engineers with the California and Nevada highway departments who generated and evaluated design alternatives based on their experience and judgment. The final attempt to improve the approximation to the optimal solution used the first as well as the second and third most cost-effective design alternatives. The net results of all three attempts were that the number of iterations was significantly increased and the resulting solutions were almost identical to the solutions of the initially developed procedure.

ANALYSIS OF FREEWAY ON-RAMP CONTROL STRATEGIES

An extension of the FREQ3 freeway model called the FREQ3C freeway control model and its application for developing ramp control strategies on the northbound East Shore Freeway in the San Francisco Bay area are discussed in this section. This is a second-generation version of the FREQ3C control model and is considered to be in essentially final form pending its ultimate incorporation into a freeway corridor model.

A schematic representation of the FREQ3C control model process is shown in Figure 11. It consists of the FREQ3 model and a linear programming decision model (LINCON), which together provide the user with three important outputs for each time slice: (a) freeway traffic performance without ramp control, (b) optimum metering rate for each on-ramp and a traffic diversion table, and (c) freeway traffic performance with the selected ramp control strategy.

An iterative procedure is required because ramp control affects the weaving flows, weaving flows affect the weaving capacities, and weaving capacities affect the optimum capacities affect the optimum ramp control. A convergence algorithm is incorporated into the model to handle the iteration process automatically.

The analytical techniques for ramp control developed in this research work will be presented in three parts: (a) linear programming formulation, (b) various options in using the freeway control model, and (c) numerical results of real-world application.

Linear Programming Formulation

The objective in the linear programming formulation can be either to maximize vehicle-input or to maximize vehicle-miles of travel on the directional freeway

$$\text{Max} \sum_{i=1}^{n} X_i \quad \text{or} \quad \text{Max} \sum_{i=1}^{n} \ell_i X_i \tag{1}$$

where

X_i = desired input rate from on-ramp i or the metering rate,
ℓ_i = average trip length of all traffic from on-ramp i, and
n = number of on-ramps.

The constraints in the linear programming formulation include the following:

$$\sum_{i=1}^{n} A_{ik} X_i \le B_k \qquad \text{for } k = 1, 2, \ldots, m \tag{2}$$

$$X_i \le D_i \qquad \text{for } i = 1, 2, \ldots, n \tag{3}$$

$$X_i \ge 0 \qquad \text{for } i = 1, 2, \ldots, n \tag{4}$$

$$S_i \le X_i \le T_i \qquad \text{for } i = 1, 2, \ldots, n \tag{5}$$

Figure 10. Cost-effectiveness diagram of most cost-effective method applied to northbound East Shore Freeway.

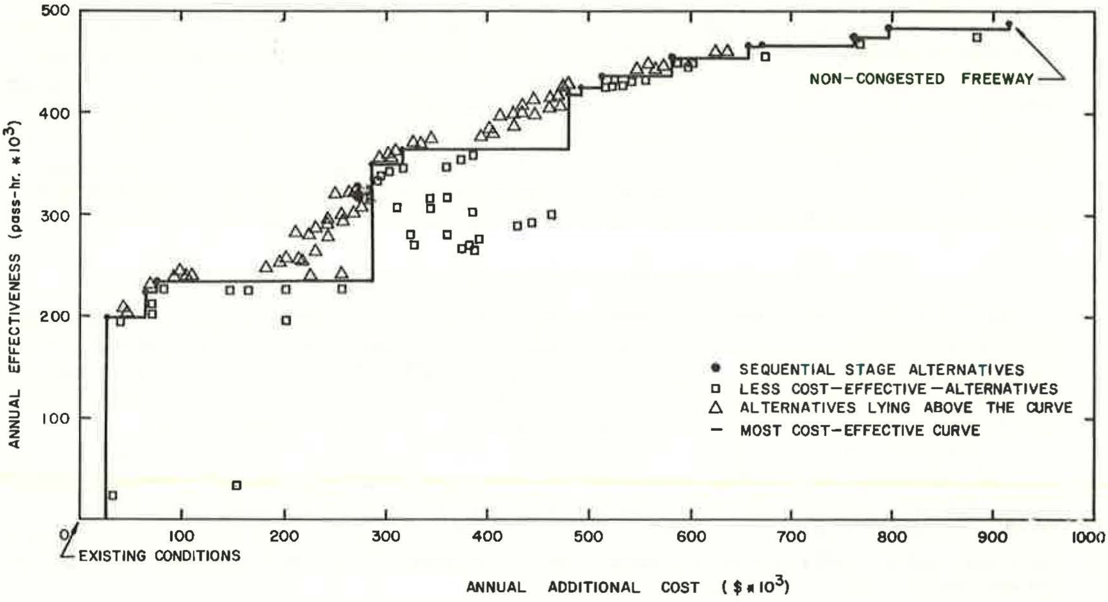
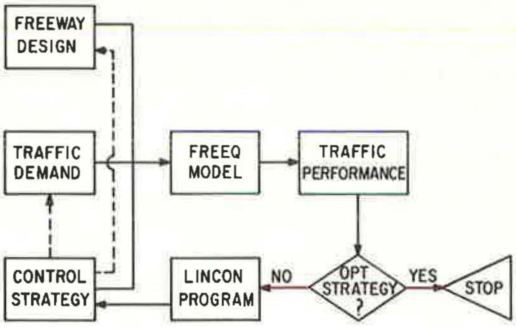


Figure 11. The FREQ3C freeway control model.



where

A_{ik} = fraction of traffic from on-ramp i going through subsection k,
B_k = the capacity of subsection k,
D_i = the demand rate at on-ramp i,
S_i = minimum metering rate,
T_i = maximum metering rate, and
m = number of subsections.

The first set of constraint equations limits the flow demand on every subsection to the subsection capacity. The second set of constraint equations limits the on-ramp flows to the demand rate. The third set of constraint equations limits the on-ramp flows to nonnegative values. The fourth set of constraint equations sets upper and lower limits on the metering rate.

Model Options

Numerous options have been incorporated into the model in order to give flexibility to the user. As indicated in Eq. 1, the objective function can be either to maximize vehicle-input or to maximize vehicle-miles of travel.

At the end of each time slice, the optimum metering rate for certain ramps may be lower than the demand rate. In this situation a queue will exist at the end of the time slice. One of two options is available: (a) It is assumed that no vehicles are diverted and excess demand from one time slice is transferred to the demand of the next time slice (no diversion); or (b) it is assumed that excess vehicles will divert to parallel routes and will not use the freeway (total excess vehicle diversion).

In the event that the ramp demand exceeds the optimum metering rate, some vehicles will wait or be diverted. Two options are available to establish which vehicles will have priority entry: (a) It is assumed that the waiting or diverted vehicles will be determined on a proportional basis—i.e., if 10 percent of the vehicles at a particular ramp must wait or be diverted, then 10 percent of each destination set is selected; or (b) it is assumed that the diverted vehicles will be determined on a trip length basis—i.e., short freeway trips will be diverted before longer freeway trips.

The set of bottleneck constraints can be selected at the capacity level or at a lower service volume level. When the capacity level is selected, there is no slack capacity or safety factor and the emphasis is on production or facility efficiency alone. When a service volume level is selected (service volume is defined as a traffic volume level below capacity that results in higher operating speeds), there is some slack capacity or safety factor and emphasis is diverted between facility efficiency and freeway level of service.

The weaving effect may or may not be considered. If the user considers the weaving effect, capacities (or service volumes) will be estimated more accurately, but an iterative procedure between capacity analysis and ramp control will be required. If the user does not consider the weaving effect, no iterative procedure will be necessary, but there may be some sacrifice in capacity calculation accuracy.

All input demands to the directional freeway are controlled through ramp metering except the main-line input demand. Inasmuch as the main-line input demand may vary from day to day and if the expected demand rate is selected, on approximately 50 percent of the days the actual demand will exceed the expected demand and congestion may occur on the freeway. Therefore, the user may select either the expected demand rate or a slightly higher demand rate that is selected on the basis of the variance-to-mean ratio and desired confidence limits.

Numerical Results

The input to the FREQ3C model includes the basic input requirements to FREQ3 and the specifications of the linear programming formulation. The output for each time slice consists of the freeway traffic performance without ramp control (Fig. 12), the optimum metering rate for each on-ramp and a traffic diversion table (Fig. 13), and the freeway traffic performance with the selected ramp control strategy (Fig. 14).

Figure 12. Freeway traffic performance without ramp control.

F072N DATA - TIME SLICE 6

TIME SLICE 5 OF 5
GROWTH PERIOD 0 OF 0
OCCUPANCY 1.38

ORIGIN - DESTINATION TABLE (VEHICLES PER HOUR)

ORIGINS DOWN	DESTINATIONS ACROSS									
0	1	2	3	4	5	6	7	8	9	SUM
1	536	356	420	212	372	180	584	248	2496	5404
2	8	0	40	16	40	20	64	28	232	448
3	0	20	0	0	0	24	68	32	264	408
4	0	0	0	0	16	12	40	20	156	244
5	0	0	0	0	12	152	388	128	576	1256
6	0	0	0	0	0	88	244	116	920	1368
7	0	0	0	0	0	0	0	32	276	308
8	0	0	0	0	0	0	0	0	220	220
SUM	544	376	460	228	440	476	1388	604	5140	9656

ROW SUMS = TOTAL INPUT DEMANDS, COLUMN SUMS = TOTAL OUTPUT DEMANDS

QUEUE COLL. SECTION 12 T2= .025
QUEUE OUT OF SECTION 1

SUB SEC	NO. LNS	SSEC LENGTH	O-D DATA DEMANDS ORG.	DES.	DEM.	ADJUSTED VOLUMES ORG.	DES.	VOL.	FRWY CAP.	WEAVE EFF	V/C	DENS. V/M/L	SPEED MPH	TRAVEL TIME	QUEUE-LENGTH	STORAGE RATE
1	3	720.	5404.	0.	5404.	4539.	0.	4589.	5750.	0.	.80	80.0	19.1 **	.43	720.	815.
2	3	2610.	448.	544.	5852.	448.	539.	5037.	5806.	0.	.87	76.3	22.0 **	1.35	2610.	769.
3	3	1660.	0.	0.	5308.	0.	0.	4498.	5728.	0.	.79	80.5	18.6 **	1.01	1660.	769.
4	3	1890.	408.	376.	5716.	408.	373.	4906.	5806.	0.	.84	77.7	21.0 **	1.02	1890.	769.
5	3	2310.	0.	0.	5340.	0.	0.	4533.	5520.	0.	.82	75.3	20.1 **	1.31	2310.	769.
6	3	1460.	244.	460.	5584.	244.	456.	4777.	5877.	73.	.81	80.8	19.7 **	.84	1460.	769.
7	3	3800.	0.	0.	5124.	0.	0.	4320.	5806.	0.	.74	84.6	17.0 **	2.54	3800.	769.
8	3	1100.	1256.	0.	6380.	1256.	0.	5334.	5334.	546.	1.00	50.5	35.2	.36	0.	0.
9	3	660.	0.	228.	6380.	0.	181.	4947.	5404.	546.	.92	37.5	44.0 *	.17	169.	388.
10	3	1480.	0.	440.	6152.	0.	352.	4765.	5857.	93.	.81	40.4	39.3 **	.43	1480.	388.
11	3	1480.	0.	0.	5712.	0.	0.	4413.	5728.	0.	.77	63.3	23.2 **	.72	1480.	388.
12	4	800.	1368.	476.	7080.	1368.	423.	5780.	6096.	754.	.95	55.7	25.9 **	.35	800.	316.
13	3	4690.	0.	1388.	6604.	0.	1153.	5357.	5357.	443.	1.00	50.7	35.2	1.51	0.	0.
14	3	2190.	0.	0.	5216.	0.	0.	4204.	5806.	0.	.72	24.6	56.9	.44	0.	0.
15	3	2320.	308.	604.	5524.	308.	504.	4512.	5800.	0.	.78	26.6	56.4	.47	0.	0.
16	3	830.	0.	0.	4920.	0.	0.	4008.	5049.	0.	.79	23.7	56.3	.17	0.	0.
17	3	1180.	0.	0.	4920.	0.	0.	4008.	4746.	0.	.84	23.9	55.9	.24	0.	0.
18	3	2560.	220.	5140.	5140.	220.	4228.	4228.	4700.	0.	.90	25.4	55.4	.52	0.	0.
TOTAL		33740.											TOTAL	13.88		

			QUEUE LENGTH VEHICLES	DELAY VEH-HRS
ON-RAMP	1	INPUT POINT	404.61	75.69
		MERGING POINT	0.	0.
		TOTAL	404.61	75.69
ON-RAMP	5	INPUT POINT	0.	0.
		MERGING POINT	261.08	57.71
		TOTAL	261.08	57.71
ON-RAMP	6	INPUT POINT	0.	0.
		MERGING POINT	.15	.02
		TOTAL	.15	.02

	CURRENT TIME INTERVAL		CUMULATIVE VALUES	
FREEWAY TRAVEL TIME=	273. VEH-HRS=	376. PASS-HRS	1076. VEH-HRS=	1485. PASS-HRS
INPUT DELAY=	133. VEH-HRS=	184. PASS-HRS	225. VEH-HRS=	311. PASS-HRS
OUTPUT DELAY=	0. VEH-HRS=	0. PASS-HRS	0. VEH-HRS=	0. PASS-HRS
TOTAL TRAVEL TIME=	406. VEH-HRS=	560. PASS-HRS	1301. VEH-HRS=	1796. PASS-HRS
TOTAL TRAV DISTANCE=	7526. VEH-MI.=	10386. PASS-MI.	37026. VEH-MI.=	51096. PASS-MI.

TRAVEL TIME FOR ONE TRIP IN MINUTES

0	1	2	3	4	5	6	7	8	9
1	1.78	3.81	5.96	9.02	9.45	10.52	12.04	12.94	13.88
2	1.35	3.38	5.53	8.59	9.02	10.10	11.61	12.52	13.45
3	0.	1.02	3.17	6.23	6.65	7.73	9.25	10.15	11.09
4	0.	0.	.84	3.90	4.33	5.41	6.92	7.82	8.76
5	0.	0.	0.	.53	.95	2.03	3.54	4.45	5.38
6	0.	0.	0.	0.	0.	.35	1.86	2.77	3.70
7	0.	0.	0.	0.	0.	0.	0.	.47	1.40
8	0.	0.	0.	0.	0.	0.	0.	0.	.52

END OF SIMULATION FOR ABOVE CRITERION

Figure 13. Optimum metering rates and diversion table.

MEAN OD DEMAND+RAMP QUEUE(HOURLY RATE)**FB72N DATA - TIME SLICE 6

ORIGIN	TOTAL	DESTINATION 1	2	3	4	5	6	7	8	9
1	5404.	536.	356.	420.	212.	372.	180.	584.	248.	2496.
2	448.	8.	0.	40.	16.	40.	20.	64.	28.	232.
3	408.	0.	20.	0.	0.	0.	24.	68.	32.	264.
4	244.	0.	0.	0.	0.	16.	12.	40.	20.	156.
5	1256.	0.	0.	0.	0.	12.	152.	388.	128.	576.
6	1368.	0.	0.	0.	0.	0.	88.	244.	116.	920.
7	308.	0.	0.	0.	0.	0.	0.	0.	32.	276.
8	220.	0.	0.	0.	0.	0.	0.	0.	0.	220.
SUM	9656.	544.	376.	460.	228.	440.	476.	1388.	604.	5140.

THE STRATEGY IS TO BE FREE OF RECURSIVE CONGESTION WITH* .90 *PROBABILITY
TAKING INTO CONSIDERATION THE FLUCTUATION OF THE MAINLINE INPUT
THE MAINLINE INPUT IS ASSUMED TO BE NORMALLY DISTRIBUTED WITH VARIANCE= 1.00 *MEAN ****************
CAPACITIES TAKE 5 ITERATIONS TO CONVERGE***
****

O-D PATTERN UNDER RAMP CONTROL, IN HOURLY RATES

ORIGIN	METERING/OR INPUT RATE	DESTINATION 1	2	3	4	5	6	7	8	9
1	5498.	545.	362.	427.	216.	378.	183.	594.	252.	2539.
2	250.	0.	0.	0.	0.	0.	0.	0.	18.	232.
3	408.	0.	20.	0.	0.	0.	24.	68.	32.	264.
4	244.	0.	0.	0.	0.	16.	12.	40.	20.	156.
5	662.	0.	0.	0.	0.	0.	0.	14.	113.	536.
6	519.	0.	0.	0.	0.	0.	0.	15.	39.	466.
7	240.	0.	0.	0.	0.	0.	0.	0.	0.	240.
8	220.	0.	0.	0.	0.	0.	0.	0.	0.	220.
SUM	8041.	545.	382.	427.	216.	394.	219.	731.	473.	4653.

MAXIMUM METERING RATES ARE *MAINLINE* 720. 720. 720. 720.1080. 720. 720.
MINIMUM METERING RATES ARE *MAINLINE* 240. 240. 240. 240. 240. 240. 240.
V/C BUFFER= .010 CAPACITY BUFFER= -0. VPH

****MEASURE OF EFFECTIVENESS**** CURRENT TIME INTERVAL ****** ****** CUMULATIVE VALUES *****

	CURRENT TIME INTERVAL	CUMULATIVE VALUES
TOTAL INPUT	2010. VEH	9844. VEH
RAMP INPUT	636. VEH	3195. VEH
TOTAL VEHICLE FREEWAY TRAVEL	8304. VEH-MILE	39611. VEH-MILE
RAMP VEHICLE FREEWAY TRAVEL	2130. VEH-MILE	9596. VEH-MILE

****

TOTAL TRIPS DIVERTED,IN HOURLY RATE= 1709. WITH THE FOLLOWING O-D PATTERN

ORIGIN	TOTAL	DESTINATION 1	2	3	4	5	6	7	8	9
1	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
2	198.	8.	0.	40.	16.	40.	20.	64.	10.	0.
3	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
4	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.
5	594.	0.	0.	0.	0.	12.	152.	374.	15.	40.
6	849.	0.	0.	0.	0.	0.	88.	229.	77.	454.
7	68.	0.	0.	0.	0.	0.	0.	0.	32.	36.
8	0.	0.	0.	0.	0.	0.	0.	0.	0.	0.

Figure 14. Freeway traffic performance with ramp control.

FREEWAY DATA - TIME SLICE 6

TIME SLICE 5 OF 5
GROWTH PERIOD 0 OF 0
OCCUPANCY 1.38

ORIGIN - DESTINATION TABLE (VEHICLES PER HOUR)

ORIGINS DOWN / DESTINATIONS ACROSS

0	1	2	3	4	5	6	7	8	9	SUM
1	545	362	427	215	378	183	594	252	2539	5498
2	0	0	0	0	0	0	0	17	232	249
3	0	19	0	0	0	23	67	31	263	407
4	0	0	0	0	16	12	40	20	156	244
5	0	0	0	0	0	0	14	112	535	662
6	0	0	0	0	0	0	14	38	465	519
7	0	0	0	0	0	0	0	0	240	240
8	0	0	0	0	0	0	0	0	220	220
SUM	545	382	427	215	394	219	730	473	4652	8037

ROW SUMS = TOTAL INPUT DEMANDS, COLUMN SUMS = TOTAL OUTPUT DEMANDS

SUB SEC	NO. LNS	SSEC LENGTH	O-D DATA DEMANDS ORG.	DES.	DEM.	ADJUSTED VOLUMES ORG.	DES.	VOL.	FRWY CAP.	WEAVE EFF	V/C	DENS. V/M/L	SPEED MPH	TRAVEL TIME	QUEUE-LENGTH	STORAGE RATE
1	3	720.	5498.	0.	5498.	5498.	0.	5498.	5750.	0.	.96	33.4	54.8	.15	0.	0.
2	3	2610.	250.	545.	5748.	250.	545.	5748.	5806.	0.	.99	45.3	42.3	.70	0.	0.
3	3	1660.	0.	0.	5203.	0.	0.	5203.	5728.	0.	.91	31.3	55.4	.34	0.	0.
4	3	1890.	408.	382.	5611.	408.	382.	5611.	5806.	0.	.97	35.3	53.0	.41	0.	0.
5	3	2310.	0.	0.	5228.	0.	0.	5228.	5520.	0.	.95	31.7	55.0	.48	0.	0.
6	3	1460.	244.	427.	5472.	244.	427.	5472.	5926.	24.	.92	33.0	55.2	.30	0.	0.
7	3	3600.	0.	0.	5045.	0.	0.	5045.	5806.	0.	.87	30.2	55.7	.78	0.	0.
8	3	1100.	662.	0.	5707.	662.	0.	5707.	5772.	108.	.99	44.1	43.1	.29	0.	0.
9	3	660.	0.	216.	5707.	0.	216.	5707.	5842.	108.	.98	38.6	49.3	.15	0.	0.
10	3	1480.	0.	394.	5492.	0.	394.	5492.	5950.	0.	.92	33.1	55.2	.30	0.	0.
11	3	1460.	0.	0.	5097.	0.	0.	5097.	5728.	0.	.89	30.6	55.5	.30	0.	0.
12	4	800.	519.	219.	5616.	519.	219.	5616.	6434.	416.	.67	25.2	55.6	.16	0.	0.
13	3	4690.	0.	731.	5397.	0.	731.	5397.	5800.	0.	.93	32.6	55.2	.97	0.	0.
14	3	2190.	0.	0.	4666.	0.	0.	4666.	5806.	0.	.80	27.7	56.2	.44	0.	0.
15	3	2320.	240.	473.	4906.	240.	473.	4906.	5800.	0.	.85	29.3	55.9	.47	0.	0.
16	3	830.	0.	0.	4433.	0.	0.	4433.	5049.	0.	.88	26.6	55.6	.17	0.	0.
17	3	1180.	0.	0.	4433.	0.	0.	4433.	4746.	0.	.93	26.8	55.1	.24	0.	0.
18	3	2560.	220.	4653.	4653.	220.	4653.	4653.	4700.	0.	.99	36.6	42.3	.69	0.	0.
	TOTAL	33740.											TOTAL	7.34		

	CURRENT TIME INTERVAL		CUMULATIVE VALUES	
FREEWAY TRAVEL TIME=	159. VEH-HRS=	220. PASS-HRS	739. VEH-HRS=	1020. PASS-HRS
INPUT DELAY=	0. VEH-HRS=	0. PASS-HRS	0. VEH-HRS=	0. PASS-HRS
OUTPUT DELAY=	0. VEH-HRS=	0. PASS-HRS	0. VEH-HRS=	0. PASS-HRS
TOTAL TRAVEL TIME=	159. VEH-HRS=	220. PASS-HRS	739. VEH-HRS=	1020. PASS-HRS
TOTAL TRAV DISTANCE=	8304. VEH-MI.=	11460. PASS-MI.	39611. VEH-MI.=	54663. PASS-MI.

TRAVEL TIME FOR ONE TRIP IN MINUTES

0	1	2	3	4	5	6	7	8	9
1	.85	1.60	2.37	3.59	3.90	4.36	5.33	6.24	7.34
2	.70	1.45	2.22	3.44	3.75	4.21	5.18	6.09	7.19
3	0.	.41	1.18	2.40	2.71	3.17	4.14	5.05	6.15
4	0.	0.	.30	1.52	1.82	2.29	3.26	4.17	5.27
5	0.	0.	0.	.44	.75	1.21	2.18	3.09	4.19
6	0.	0.	0.	0.	0.	.16	1.13	2.04	3.14
7	0.	0.	0.	0.	0.	0.	0.	.47	1.57
8	0.	0.	0.	0.	0.	0.	0.	0.	.69

END OF SIMULATION FOR ABOVE CRITERION

PROGRESS TOWARD A FREEWAY CORRIDOR MODEL

The proposed structure of the freeway corridor model and the first-generation version of a major arterial street model (MART1) are described in this section. This is considered to be a second-level research activity because it involves the development of analytical techniques for evaluating design improvements or control strategies on a freeway corridor basis. The illustration in the background of Figure 1 is a schematic representation of this freeway corridor evaluation process.

Structure of Freeway Corridor Model

A freeway corridor consists of a directional freeway with at least one parallel major arterial street and with cross-arterials connecting the major arterials to the freeway. Geographically, the corridor can be thought of as being 5 to 10 miles (8-16 km) long and 1 to 2 miles (1.6-3.2 km) wide.

The structure of the freeway corridor model is similar to the freeway model (Fig. 1). The input is expanded from freeway design parameters to freeway corridor design parameters, and the freeway demand patterns are expanded to freeway corridor demand patterns. The freeway simulation model becomes the freeway corridor simulation model, and the freeway design and control decision models become the freeway corridor design and control models.

Obviously, the freeway corridor model will be much more complex. The handling of freeway corridor demands will be difficult, and route selection or traffic assignment will be required. The control alternatives will include intersection control as well as freeway ramp control. The complexities of demand and capacity analysis will be significantly increased. In addition to the added complexities, the quantity of analysis is grossly expanded, and the computer time and user ease must be maintained within certain limits.

It is anticipated that the freeway corridor modeling will be undertaken in stages. First, a major arterial street model will be developed. The second stage will be to extend the major arterial street model to a major arterial corridor model that would include the parallel and crossing arterial streets. At this point this developed model manually combined with the freeway model can be employed for simulating a freeway corridor.

The next stage would be to combine the major arterial corridor model with the freeway model to automatically simulate a freeway corridor with the assumption that a preselected routing of the demand pattern is specified. Then, the preselected routing would be replaced by a built-in traffic assignment subroutine. Finally, the freeway corridor design and control decision models (extensions of the freeway design and control decision models) would be developed and incorporated with the freeway corridor model.

Major Arterial Street Model

The major arterial street model can be thought of as the freeway model with two important extensions. First, the concern now is with two directions of flow, not one, and the two directions of flow may interfere with one another. Second, signalized intersections are introduced, and the question of signal timing and intersection queuing and delay must be handled. The developed major arterial street model is called the MART1 model.

Input to MART1 consists of design parameters, traffic demand patterns, and intersection control parameters (splits, cycle length, and offsets). The MART1 model is a deterministic macroscopic simulation model. It has been computerized, and is written in FORTRAN IV for the University of California's CDC 6400 computer. The FORTRAN program deck consists of approximately 1,200 statements.

The output from the MART1 model includes capacity, demand, volume, queuing characteristics, and average delay for each approach to each intersection, plus volume and travel time for each link. Also, total vehicle-hours on the system are presented. All of these results are provided for each time slice during the time period of interest, as with FREQ3. Some sample output is shown in Figure 15.

Figure 15. Sample output of MART1 computer program.

INT	ARM	L.T. TYPE	CAPCTY	L.T. CAPCTY	L.T. DEMAND	THR. DEMAND	R.T. DEMAND	QUEUE	L.T. QUEUE	QDELAY	L.T. QDELAY	DELAY	L.T. DELAY
1	1	1	1396.	0.	27.	206.	50.	0.	0.	0.	0.	14.1	0.
1	2	1	1045.	0.	5.	150.	38.	0.	0.	0.	0.	9.5	0.
1	3	1	1933.	0.	12.	316.	29.	0.	0.	0.	0.	14.1	0.
1	4	1	921.	0.	61.	203.	26.	0.	0.	0.	0.	9.9	0.
2	1	1	1889.	0.	24.	180.	9.	0.	0.	0.	0.	11.0	0.
2	2	1	707.	0.	17.	37.	20.	0.	0.	0.	0.	11.8	0.
2	3	1	1890.	0.	5.	294.	21.	0.	0.	0.	0.	11.4	0.
2	4	1	754.	0.	28.	79.	21.	0.	0.	0.	0.	12.1	0.
3	1	1	2105.	0.	21.	170.	10.	0.	0.	0.	0.	6.7	0.
3	2	1	753.	0.	19.	56.	27.	0.	0.	0.	0.	17.3	0.
3	3	1	2227.	0.	15.	295.	16.	0.	0.	0.	0.	7.0	0.
3	4	1	786.	0.	14.	41.	8.	0.	0.	0.	0.	17.1	0.
4	1	1	2371.	0.	2.	149.	19.	0.	0.	0.	0.	5.9	0.
4	2	1	408.	0.	2.	1.	8.	0.	0.	0.	0.	20.3	0.
4	3	1	2384.	0.	9.	312.	6.	0.	0.	0.	0.	6.2	0.
4	4	1	377.	0.	8.	1.	13.	0.	0.	0.	0.	20.3	0.
5	1	1	1606.	0.	20.	131.	27.	0.	0.	0.	0.	10.8	0.
5	2	1	1766.	0.	1.	293.	55.	0.	0.	0.	0.	12.5	0.
5	3	1	1751.	0.	14.	309.	33.	0.	0.	0.	0.	11.3	0.
5	4	1	1765.	0.	1.	222.	27.	0.	0.	0.	0.	12.2	0.
6	1	1	1754.	0.	37.	192.	9.	0.	0.	0.	0.	12.2	0.
6	2	3	1008.	992.	29.	57.	65.	0.	0.	0.	0.	10.5	9.8
6	3	1	1835.	0.	7.	279.	41.	0.	0.	0.	0.	12.5	0.
6	4	1	920.	0.	23.	131.	3.	0.	0.	0.	0.	10.7	0.
7	1	1	1762.	0.	7.	58.	22.	0.	0.	0.	0.	9.7	0.
7	2	1	1111.	0.	9.	53.	37.	0.	0.	0.	0.	13.0	0.
7	3	1	1836.	0.	27.	365.	14.	0.	0.	0.	0.	10.5	0.
7	4	1	1018.	0.	53.	95.	24.	0.	0.	0.	0.	13.2	0.

TOTAL VEHICLE HOURS 871.1 TOTAL PASSENGER HOURS 1202.1
CUM. TOTAL VEHICLE HOURS 2620.8 CUM. TOTAL PASSENGER HOURS 3616.8

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