HIGHWAY RESEARCH RECORD

Number 49

Traffic and Operations—General 1963

6 Reports

Presented at the
43rd ANNUAL MEETING
January 13-17, 1964

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of the
Division of Engineering and Industrial Research
National Academy of Sciences—
National Research Council
Washington, D. C.
1964

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Traffic and Parking Requirements of Off-Center Medical Office Buildings

PAUL W. SHULDINER, DONALD S. BERRY, and JAMES M. MONTGOMERY, JR.
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Decentralization of population and construction of large community hospitals in suburban areas have been attracting an increasing number of medical office buildings and clinics to suburban and off-center locations. In many cases, parking requirements for these buildings have been based on existing specifications derived from data for facilities in central areas well served by public transportation. These requirements are inadequate, principally due to the high rate of auto use characteristic of suburban areas.

To obtain more appropriate estimates of parking requirements for medical buildings, intensive studies of two such facilities, one in central Evanston, Ill., the other in nearby suburban Skokie, Ill., were conducted to determine the relationships that exist between the various functions these facilities serve and the traffic and parking demands they generate. The results showed that demand varied not only with floor space and number of doctors, but also with location, services available for treatment and diagnosis, scheduling procedures, and other operational aspects of the facility. Wide variations also exist between medical specialties in the rate at which the patients are treated, ranging from an average of more than 3.5 patients per office hour in the case of pediatricians, for example, to less than 1.5 patients per hour for general surgeons.

The impact of location of the facility on parking demand was apparent in the greater use of private autos for travel to the suburban clinic as compared to auto use associated with the office building in central Evanston. Approximately 82 percent of the patients using the suburban clinic required parking, in contrast to 67 percent for the Evanston medical building. An even more pronounced difference was noted in the mode of travel used by employees. Only 9 percent of the employees interviewed at the Evanston building drove to work, whereas 74 percent at the suburban clinic used a car.

An interesting sidelight to the study of the Evanston facility was that approximately 50 percent of all patients interviewed indicated that they had shopped, gone to the bank, eaten, or made other errands as part of their medical trip. An average of 1.8 errands per person was made by this group.

•DURING RECENT YEARS an increasing number of medical buildings and clinics has been constructed at suburban or other off-center locations. In part, this policy reflects the general trend in suburbanization of many consumer-oriented or service activities. In part, too, medical clinics and offices have congregated about the larger community hospitals constructed on large tracts of land in outlying locations. The concentration of such activities in the vicinity of hospitals is a reflection of the trend in

Paper sponsored by Committee on Parking.

modern medical practice to be as closely linked as possible to large hospitals with their array of facilities for diagnosis, treatment and care.

As with other land uses, there is a danger that newly constructed medical office buildings or clinics will generate excessive parking and traffic demands on the streets adjacent to them. To guard against this, many communities have adopted zoning ordinances which specify the number of parking spaces required for a clinic of a specific size, either as a function of the floor area of the structure or in terms of the number of doctors that it will house. Most of these specifications are based on fairly old data for facilities in central areas well served by public transit. Their application to current suburban or off-center situations is open to question.

If experience with other types of land uses is at all applicable in this case, medical traffic generation should reflect not only differences in floor space or number of doctors, but also variations in location, services available, office hours, scheduling procedures, and other operational aspects of the facility. Therefore, an intensive study was undertaken of a medical office building and a clinic in order to determine the relationships existing between the various functions these facilities serve and the traffic demands they generate. As an initial effort, two parking-demand studies were made for the two types of medical buildings. The first was an investigation of the office procedures of 26 physicians practicing in a multistoried office building in downtown Evanston, Ill. The second study analyzed the office practices of 21 physicians at a suburban clinic in Skokie, a village directly west of Evanston.

MEDICAL OFFICE BUILDING, EVANSTON

In 1961, a group of 20 doctors in Evanston, Ill., initiated a plan to construct and operate a medical office building near Evanston General Hospital. At present, most of the doctors are housed in a downtown building about 1½ mi from the hospital. The proposed building will provide 40 medical office suites and will contain two X-ray and clinical laboratories, as well as a prescription pharmacy and an optical dispensary. It is to be within 500 ft walking distance of the hospital and will have bus connection to the central business district the (CBD) and many of the other areas of Evanston. The location of both buildings and their relation to other features in the Evanston area are shown in Figure 1.

The off-center location also will provide for improved off-street loading and parking, as compared to the eight-story office building fronting on a busy street in downtown Evanston.

Recognition of the effect that this new facility might have on the adjacent residential neighborhood, as well as the potential impact on the Evanston CBD of the relocation of a significant portion of its medical services, led to a study of the nature and amount of traffic which this group of doctors generated. Reported here are three phases of this study:

1. Characteristics of the parking demand generated by the various doctors and estimated parking requirements of the new facility;
2. Estimated traffic impact on streets adjacent to the new facility;
3. Potential effects on the Evanston CBD of the relocation of these medical services.

The analyses of all three of these phases were based on observations of current office procedures of 26 doctors actively practicing in Evanston with offices in the eight-story downtown office building and on results of questionnaires completed by patients and employees of these doctors. The study was directed toward the practices of doctors who were proposing to relocate, in order to insure complete cooperation in what otherwise might have been construed as an invasion of privacy. Also, by concentrating on the group most directly involved, a more accurate estimate of potential parking and traffic demands at the new building could be obtained than if the practices of a random set of doctors had been studied.

A follow-up study is scheduled upon completion of the new building to test the actual traffic and parking demands generated.

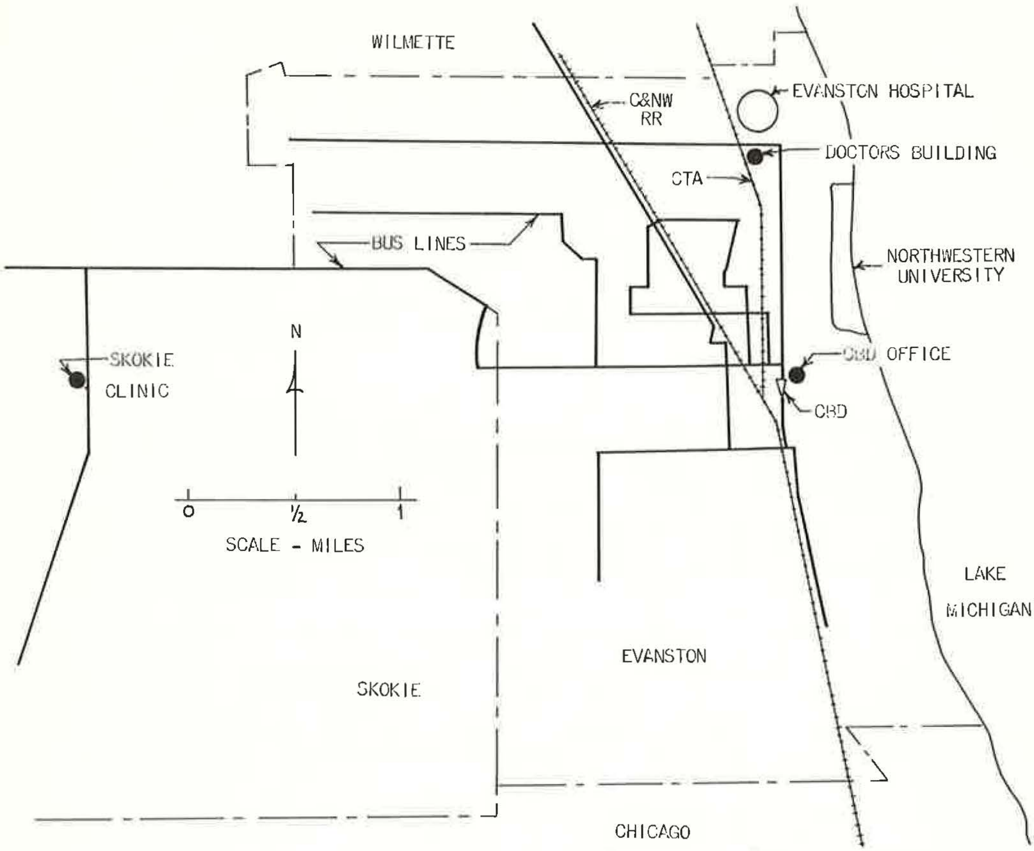

Figure 1. Location of study sites in Evanston and Skokie.

Parking Requirements

To determine the maximum probable number of parking spaces required for the new medical building, it was necessary to estimate the peak accumulation of persons likely to be on the premises on a typical active weekday and the proportion of these persons likely to come by car. Separate estimates were made for doctors, patients, employees, and other users of the building.

For Doctors. —The number of doctors expected to be present at any one time in the new building provided the basis for estimating traffic and parking demands for prospective users of the facility. The group of doctors sponsoring the project estimated that a maximum of 40 doctors would hold office hours at the same time in the new building. This figure was substantiated by a study of the intensity of the present use of medical office space in downtown Evanston.

Fourteen office suites, housing 41 doctors, in the eight-story building were selected and checks were made of actual office hours of these doctors on 3 weekdays. About a third of the doctors whose names were listed on the doors were not in active practice; they either rarely visited their offices, or were retired or deceased. The 26 doctors in active practice include internists, surgeons, orthopedic surgeons, opthalmologists, neurologists, psychiatrists, obstetricians, gynecologists, a dermatologist, and eye, ear, nose and throat specialists. Of these 26 doctors, no more than two thirds were ever observed to be in their offices at any one time (Table 1).

The proposed building, with approximately 30,000 sq ft is planned to provide offices for 60 active doctors in 40 suites. Thus, a maximum of 40 doctors will be in their

TABLE 1

OBSERVED PEAK PATIENT LOADS AT 14 DOCTORS' SUITES[a]

Day and Date	Time	No. of Doctors in Attendance	No. in Suite: All Persons	No. in Suite: Patients[b]	No. per Doctor: All Persons	No. per Doctor: Patients[b]
Thursday	1:32 - 2:17	10	30	-	3.0	-
	2:19 - 2:53	15	35	-	2.3	-
	2:57 - 3:35	16	46	-	2.9	-
	3:37 - 3:52	17	38	-	2.2	-
Friday	2:14 - 2:40	15	53	44	3.5	2.9[c]
	2:42 - 3:06	15	61	39	4.1	2.6
	3:10 - 3:45	15	37	34	2.5	2.3
Monday	2:30 - 3:07	17	58	46	3.4	2.7
	3:12 - 3:53	18	49	43	2.7	2.4

[a]Study made in Evanston, Ill. [b]Patients were not counted separately on Thursday.
[c]Maximum observed load.

offices to receive patients at one time. This figure agrees with that originally suggested by the doctors themselves. Using a 1:1 auto-driver ratio, a total of 40 spaces would be required to meet the parking demands of the doctors themselves.

For Patients.—The number of patients expected to visit doctors in the new building at one time was estimated by studying office procedures currently being used by some of the same doctors. Actual counts were made of the number of patients present and persons accompanying them at different times in the 14 medical suites. Results are summarized in Table 1 for nine periods covering three days.

The pertinent totals for the determination of parking requirements for patients are the numbers of patients in the suites at one time, because accompanying persons will generally use the same car as the patient. The figures in Table 1 represent the total number of patients either being examined or waiting to be examined at any single time, including those to be given injections or undergoing laboratory tests by the doctors' employees.

In addition to the observed peak patient loads, Table 1 gives the ratios between the total number of patients in the 14 suites for any period and the number of doctors having office hours during the same period. The maximum figure of 2.9 patients per doctor in attendance was selected as the basis for determining the peak patient demand at the new building. This was based on the assumption that the doctors' office procedures will remain essentially the same in the new facility.

After determining the number of patients expected to be in the doctors' suites at one time, it was necessary to estimate the proportion of these patients requiring parking facilities. The mode of travel presently used by patients served as a partial basis for this estimate. This information was obtained from a questionnaire distributed to 300 patients at the time they visited the doctors.

A summary of the responses on the questionnaires (Table 2) shows that about 64 percent of the patients came to the doctors' offices in automobiles that were parked in the downtown area. The percentage of patients requiring parking at the new building would undoubtedly be higher because of the effect the off-central location of this new facility

TABLE 2

MODE OF TRAVEL OF PATIENTS COMING TO 3 MEDICAL ESTABLISHMENTS

Building	Location	Total No.	Arrived by Car: Required Parking No.	Required Parking %	No Parking Required No.	No Parking Required %	Via Taxicab No.	Via Taxicab %	Via CTA or Bus No.	Via CTA or Bus %	Walked No.	Walked %	Other No.	Other %
Present med. off.[a]	Evanston CBD	300	192	64	10	3	19	6	47	16	21	7	11	4
Proposed med. off.[b]	Evanston off-center	300	225	75	10	3	19	6	40	13	2	1	4	2
Clinic[c]	Skokie suburban	429	-	82	-	4	-	1	-	2	-	11	-	-

[a]Study made November-December 1961. [b]Estimated from present Evanston data. [c]Study made June 1963.

will have on the proportions traveling by various modes. After adjusting upward the proportion expected to drive, it was estimated that no more than 75 percent of the patients will require parking space at the new building. It should be noted that the data in Table 2 are used only to obtain the proportion of patients requiring parking; they are not used to estimate the number of spaces needed. This number, based on the peak load of 40 doctors in attendance in the new building at one time on a typical active day, is estimated to be 88 spaces.

For Doctors' Employees. —The mode of travel currently used by doctors' employees in coming to work at the present offices was also obtained by questionnaire and is summarized in Table 3. At present, only 9 percent of the employees drive to work. This figure will probably rise to 26 percent for these same employees at the new building, due to improved parking conditions and to the need for riders of certain bus lines to transfer to another bus line in order to reach the new building (Fig. 2). Eventually, 40 to 50 percent of the doctors' employees may drive to work. A 46 percent figure was used to compute the number of parking spaces required for doctors' employees.

The 20 active doctors in the nine suites covered in the employee questionnaire employed a total of 23 persons. If 60 doctors actively use space in the new building, a total of 32 parking spaces would be needed for doctors' employees.

For Other Users of Building. —There are four ancillary facilities to be included in the new building: an X-ray laboratory, a clinical laboratory, a pharmacy, and an opti-

TABLE 3

MODE OF TRAVEL OF EMPLOYEES COMING TO 3 MEDICAL ESTABLISHMENTS

Building	Location	Total No.	Employees Surveyed: Arrived by Car: Drove No.	%	Passenger No.	%	Via Bus No.	%	Walked No.	%	Other No.	%
Present med. off.[a]	Evanston CBD	23	2	9	5	22	9	38	5	22	2	9
Proposed med. off.[b]	Evanston off-center	23	6	26	7	30	8	35	-	-	2	9
Clinic[c]	Skokie suburban	35	-	74	-	6	-	9	-	11	-	-

[a]Study made November-December 1961. [b]Estimated from present Evanston data. [c]Study made June 1963.

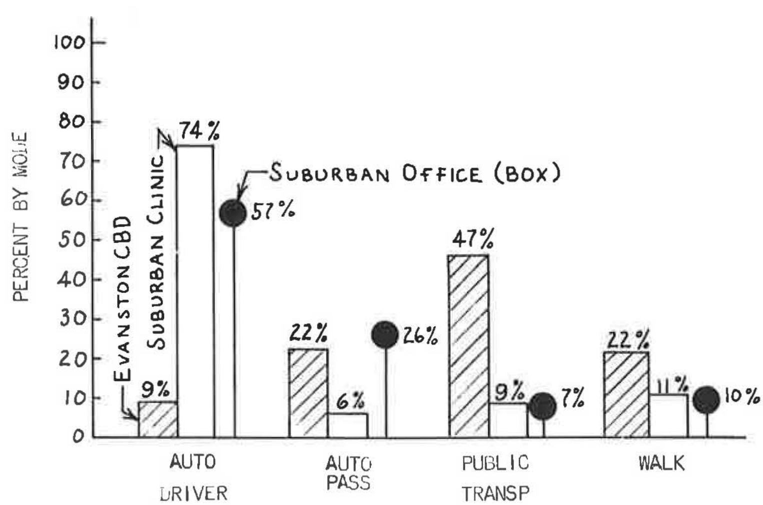

Figure 2. Mode of transportation used by employees in coming to two medical establishments.

TABLE 4

ESTIMATE OF MAXIMUM NUMBER OF VEHICLES REQUIRING PARKING AT PEAK HOUR ON TYPICAL ACTIVE WEEKDAY FOR PROPOSED MEDICAL OFFICE BUILDING (Evanston)

Parking Generators	No. Vehicles
Patients	88
Doctors' employees	32
Doctors	40
X-ray and clinical lab. (2)	
Employees	2
Patrons	8
Pharmacy	
Employees	4
Patrons	4
Optical Co.	
Employees	2
Patrons	2
Building employees	3
Detail men, sales, service, etc.	5
Total Parking Needed	190

cal dispensary. In addition, building maintenance and various sales and service personnel must also be accommodated. Table 4 specifies the number of parking spaces estimated for those purposes, based on observations of the existing operation of present medical facilities in the area.

Total Parking Requirements.—Based on the preceding information, summarized in Table 4, the total parking requirements for the new building are estimated to be 190 spaces, or approximately 3.2 spaces per doctor.

Traffic Impact

The vehicular traffic volume generated by the proposed doctors' building was estimated from studies of pedestrian volume at the entrance to the downtown office building as follows:

1. Studies were made of the total number of persons entering and leaving the downtown building (excluding the first floor in 15-min periods from 4:00 to 5:30 PM on each of three active weekdays. Separate tallies were made for children (less than 16 yr old) and for those older than 16 yr. An average for 2 days indicated 250 persons entering and 470 persons leaving the building.

2. Estimates then were made of the total numbers of persons entering and leaving the proposed new building from 4:15 to 5:15 PM. These estimates were taken as 36 percent of the numbers determined for the downtown office building, assuming that the new building will generate a demand in proportion to the total number of doctors and dentists housed in the two buildings.

3. An estimate of the total numbers of vehicles entering and leaving the new building in the evening peak hour was made by assigning a mode of travel for each of the 90 persons entering and the 170 persons leaving the building in the evening peak hour. The mode-of-travel percentages conformed to those previously used in the parking analysis.

TABLE 5

ESTIMATED TRAFFIC VOLUME TO AND FROM PROPOSED MEDICAL OFFICE BUILDING[a] (Evanston)

Direction of Travel and Mode	Peak Hour Traffic, PM[b]		
	Persons	Inbound Veh.	Outbound Veh.
Inbound			
Drivers	50	50	-
Passengers, vehicle parked	20	-	-
Passengers dropped off or via taxi	10	10	10
By transit	10	-	-
Total	90		
Outbound			
Drivers	80	-	80
Passengers, vehicle parked	30	-	-
Passengers picked up or via taxi	20	20	20
Walked	10	-	6
By transit	30	-	-
Total	170	80	116

[a]Based on counts of all persons entering and leaving from upper floors of 8-story downtown office building housing 126 doctors, 51 dentists and 28 other offices. Counts were made 4:00-5:30 pm for 3 weekdays in 1961. Peak-hour results for Inbound: 250 persons, including 55 persons below driving age. Outbound: 470 persons, including 130 children.
[b]Based on 60 active doctors end 4 less active doctors in new building of 30,000 sq ft. No. of persons taken as 36% of the number using 8-story building in the peak hour.

Results of the study (Table 5) indicate that 110 vehicles can be expected to exit from the proposed driveway onto Ridge Avenue in Evanston, with approximately half going north and half southbound. About 80 vehicles will enter the driveway in the same hour. Traffic volume on Ridge Avenue was estimated to increase by 95 vehicles in the evening peak hour and 510 vehicles in a 24-hr period.

These estimates are high because some of the people now traveling to the doctors' offices in downtown Evanston are using Ridge Avenue now. When these trips are diverted to the new building, they will not represent additional travel on Ridge Avenue.

The additional vehicular volume generated by the proposed doctors' building was estimated to add about 17 percent to the existing peak-hour flow of 568 vehicles per hr on Ridge Avenue. The total 24-hr traffic volume was estimated to increase 10 percent because of the new building.

TABLE 6

SUMMARY OF ERRANDS LINKED TO TRIPS FOR MEDICAL VISITS[a]

Errands	No.	%
Other errands in CBD:		
No	142	50
Yes[b]	143	50
Total	285	100
Place for errand if medical trip not made:		
CBD	66	46
Elsewhere	56	39
Unspecified	21	15
Total	143	100

[a]From interviews of patients at 14 medical suites in downtown Evanston.
[b]Shop 108, 60%; bank 17, 9%; eat 37, 20%; other 21, 11%.

Potential Effect on Evanston CBD

A natural concern of the City of Evanston and of others involved in the establishment of the new medical facility is the potential impact on the Evanston CBD of the relocation of a portion of its medical services to an outlying location. Of prime importance are the extent to which trips for shopping and other commercial purposes are linked to visits to doctors' offices, and the proportion of the linked trips no longer attracted to the Evanston CBD when medical visits are made elsewhere.

Table 6 summarizes the results of a survey conducted among patients visiting the 14 medical suites in downtown Evanston. Of the 285 respondents to this survey, 143, or 50 percent, indicated that they had made errands in downtown Evanston as part of their trip to the doctor. A total of 183 errands, or 1.28 errands per person, is included in this 50 percent. Viewed in another way, each medical trip resulted in approximately 0.65 other errands in the Evanston CBD. Most of these linked errands were devoted to shopping, with the next largest proportion for the purpose of eating a meal.

The offices vacated by the doctors who relocated would undoubtedly be occupied by other physicians, dentists or professionals, who would tend to mitigate the loss of patronage through the activities of their own clientele. There is no way of determining from this study the degree of patronage replacement which would take place. It should be noted, however, that the proposed new medical building would be constructed within Evanston, about 1½ mi from the CBD, in an area not served by many commercial establishments. Therefore, many of the patients at the new facility could be expected to continue to patronize downtown establishments as frequently as before.

MEDICAL CLINIC, SKOKIE

The second facility studied is a medical clinic located in Skokie, Ill., a suburb of Chicago. The clinic has been in operation for more than six years, is modern in appearance and typifies present and near-future suburban medical clinics. The area served by the clinic is distinctly suburban in nature and offers a basis of comparison with similar facilities located in both CBD and urban off-center areas. Since only one site in one specific area was studied, the data and results drawn from the study are necessarily limited. However, when this work is combined with that from other studies of a similar nature, design criteria of more general applicability may be developed.

Services offered by the clinic are given in Table 7. In addition, there are in the building a pharmacy, dental services, and an optical service not affiliated with the clinic but renting office space from the clinical association. These will be referred to hereafter as "auxiliary services." Every specialist listed in Table 7 is actively engaged in medical practice at the clinic during certain scheduled hours of the week.

There are also 35 persons employed at the clinic. In contrast to the minimal demand produced by the Evanston employees, these people generate a significant portion of the traffic and parking demand (Table 3). At present most employees driving to work must park their cars on a lot across the street from the clinic.

TABLE 7

AVERAGE HOURLY PATIENT LOADS CARRIED BY VARIOUS TYPES OF SPECIALISTS[a]

Type	No.	Total Patients Seen	No. of Hr Worked[b]	Range of Avg. Individual Hourly Loads (pph)	Avg. Hourly Load (pph)
Pediatrician	3	397	88	3.0-7.1	4.5
Ear, Nose and Throat Specialist	1	43	12	3.6	3.6
Orthopedic Surgeon	2	129	44	2.9-3.0	2.9
Dermatologist	1	22	8	2.8	2.8
Internist	5	329	144	1.6-3.4	2.3
Gynecologist	2	57	36	1.0-2.0	1.6
General Surgeon	3	35	32	0.7-2.8	1.1
Opthalmologist	1	21	28	0.8	0.8
Clinical Lab.	1	32	48	0.7	0.7[c]
Physical Therapist	1	30	48	0.6	0.6[c]
X-ray	1	18	48	0.4	0.4[c]

[a]Study conducted at suburban clinic, Skokie, Ill., in June 1963.
[b]Approximate.
[c]Patients coming directly from other offices within clinic are not included.

Since all patients wait in a common central waiting room, it was quite difficult to isolate them according to service desired. In this case, a questionnaire was considered to be both unwieldy and potentially bothersome to the patients, and, as a result, this method was rejected. The problem of determining patient loads and services desired was met by counting the billing slips used for each patient receiving clinical service. Through the tabulation of this information, average loads were computed on both a daily and hourly basis. These data were also grouped according to specialty or service received.

To attempt to correlate traffic and parking demand with patient loads, the billing slips were counted for those days on which traffic data were obtained. The billing slips do not include dental patients, optical patients, or pharmacy customers, yet these patrons account for a significant portion of the traffic activity. Data for these services were obtained on an individual basis through interviews with key personnel and examination of appointment books.

Parking requirements for doctors were determined by examination of their scheduled office hours. Travel modes and parking demand for employees were determined through printed questionnaires. The remaining pertinent information was gathered through informal interviews with clinic personnel.

Parking, Scheduling, and Medical Specialties

The traffic generated at a medical clinic is dependent on the types of medical services rendered and the related scheduling procedures. For example, an allergist, who may wish to see his patients only long enough to administer injections, will certainly generate a higher traffic flow than a surgeon, who may wish to give each patient a thorough examination.

A second consideration arises from the variation of rate of scheduling among individuals. For instance, a very active physician may see 50 patients per day and still practice good medicine, whereas another slower, more methodical man may see only 20 patients per day. For example, the opthalmologist at the Evanston facility generated a parking demand four to five times greater than that produced by his Skokie counterpart.

Adherence to these schedules is another controlling factor; since many specialists tend to fall behind schedule, a backlog of patients will accumulate in the waiting rooms. People who do not have scheduled appointments, or "walk-ins," also contribute to disruption of scheduling. Much of this walk-in traffic is caused by minor accidents requiring immediate medical treatment.

To meet patient parking demands, the preceding factors must be converted to numerical terms. From examination of billing slips and posted office hours, average hourly patient loads were calculated for each type of specialist currently practicing at the clinic. Table 7 summarizes these computations. The figures given in this table merit some explanation. Due to the very small sample sizes used, the accuracy of the figures is by no means certain. Further data are needed from other clinics in other locations to either verify or revise these numbers. Many patients use more than one service while at the clinic. For instance, an internist may send a patient to the X-ray room after having examined him. Table 7 indicates that only 0.4 patient per hour (pph) comes to the clinic for the sole purpose of using the X-ray facilities. The actual patient load for this service is much higher, but most of the load comes from directly within the clinic and, therefore, has little bearing on parking demand. For the same reason, the clinical laboratory and physical therapy facilities are also considered to have low patient loads from a parking standpoint.

Proposed Parking Design Criteria as a Function of Medical Specialty

Inherent differences in parking demands due to type of specialty may make it expedient to classify each specialty according to average hourly load, from this devising a set of design criteria. Table 8 illustrates one possible grouping. Due to the limited data, the figures are highly arbitrary and subject to revision on completion of further research. Although no patient load data were available for dentists, they were assigned to Group I, which generates a relatively high parking demand. Eventually this table might be used for medical clinics in any area, providing that an appropriate factor is applied to account for change in patient travel modes. Placement of pediatricians in Group I is questionable, because although they have very high patient loads, much of this is "group traffic," and thus requires a fairly low parking demand per patient.

Some specialists (allergists, chiropodists) are not considered here; pertinent data should be gathered to cover these and other medical specialties and services.

Travel Modes to Medical Establishments

Several possible sources of error exist in the travel mode data for the Skokie clinic (Table 2). In data collection, a figure of 1.0 patients per group of people coming to the clinic was assumed. Examination of the billing slips revealed this to be a faulty assumption. Of 355 incoming groups of people, approximately 50, or 14 percent, of these contained two or more patients. In counting the billing slips, it was assumed that persons from the same family came to the clinic as a group. Most of this group traffic was generated by the pediatric services. To account for group traffic, the number of patients traveling to the clinic by automobile was adjusted upward. Adjustments were purposely made high, because persons from two different families may have come to the clinic as a group, and this would not be detected in the billing slips.

Examination of travel mode data for medical establishments in the three different areas suggests several apparent trends. As medical facilities become located farther from central urban areas and transit lines, the proportion of patients traveling to these facilities by automobile increases. This proportion ranged from 64 percent in downtown Evanston to 82 percent in the suburban area at Skokie. The estimated proportion for

TABLE 8

OBSERVED PARKING DESIGN RATES FOR PATIENTS AT SUBURBAN MEDICAL CLINICS (Skokie)

Group No.	Parking Demand	Range of Hourly Patient Loads[a] (pph)	Specialists in Group	No. of Parking Spaces per Specialist
I	High	≥3.5	Pediatricians; Dentists; Ear, Nose, and Throat	4.0
II	Avg.	2.5-3.5	Orthopedic Surgeons, Dermatologists	3.0
III	Low	1.5-2.5	Gynecologists, Internists	2.0
IV	Very low	<1.5	General Surgeons, Clinical Lab., Physical Therapy, X-ray	1.0

[a]Table 7.

TABLE 9

ESTIMATE OF MAXIMUM NUMBER OF VEHICLES REQUIRING PARKING AT PEAK HOUR ON MOST ACTIVE WEEKDAY (Skokie)

Parking Generators	No. Vehicles
Doctors[a]	14
Employees[b]	20
Patients	
Group I[c]	20
Group II	-
Group III[d]	14
Group IV[e]	5
Miscellaneous[f]	5
Total	78

[a]14 doctors scheduled.
[b]Derived from questionnaire.
[c]5 doctors in attendance.
[d]7 doctors in attendance.
[e]2 doctors plus 2 lab. and 1 physical therapist.
[f]Salesmen, visitors, servicemen, pharmacy customers.

TABLE 10

PEAK PARKING LOT OCCUPANCY[a]

Day	No. of Doctors in Attendance	No. of Vehicles	Time of Peak PM	Remarks
Monday	10	39	3:45-4:00	
Tuesday	12	52+[b]	3:30-3:45	Lot overloaded
Wednesday	6	47	4:30-4:45	Near capacity
Thursday	4	31	4:15-4:30	
Friday	13	49	3:34-4:00	Near capacity
Saturday	8	44	9:30-9:45[c]	Near capacity

[a]Study made at suburban clinic, Skokie, Ill., in June 1963.
[b]Lot capacity: 52 cars.
[c]Morning count taken.

the off-center area in Evanston was 75 percent. As would be anticipated, the proportion of transit riders dropped off sharply from 16 percent in the CBD to an estimated 2 percent in the suburban area. A corresponding value of 13 percent was used for the off-center area.

A similar analysis was made for employee travel modes. Results (Table 3) indicate that the distribution of employee travel modes tends to be similar to those of the patients, reinforcing the previous statements regarding travel modes and clinic location.

Parking Demand Analysis for Skokie Clinic

Until more data are gathered, it does not appear reasonable to apply a single general formula to determine adequate parking requirements for medical clinics in all locations and under every specific condition likely to be encountered. The analysis of parking demand is similar to that made for the Evanston facility. Differences arise partly from site conditions which include limited transit facilities and little on-street parking. The clinic patronage is stable. As before, the design criterion used is provision for the maximum number of vehicles requiring parking on the most active weekday of a typical week. On the basis of specialties and number of doctors scheduled, it is estimated that the peak parking demand occurs on Tuesday afternoon, when 14 specialists are scheduled.

It has been stated (1) that "... doctors come first where parking privileges are concerned, not simply as a matter of convenience, but primarily because service to patients requires that parking space for doctors be quickly accessible." Although this statement was intended for use in design of hospital parking space, provision of service to patients at a medical clinic is also rather important. Reservation of parking space for doctors insures them a place to park and promotes better service to clinic patients.

Data summarized in Table 9 indicates that 78 parking spaces are required at the most active period. Actual parking lot occupancies for a full week are shown in Table 10. It was not possible from these observations to check adequately the preceding estimate, because the capacity of the lot was limited to 52 spaces. However, unless frequent overloading is accepted, it is clear that many zoning ordinances based on gross area would not provide for adequate parking at suburban clinics.

Comparison With Existing Standards

To provide sufficient parking space at medical clinics and office buildings, adequate design criteria must be developed. Table 11 gives the off-street parking requirements as specified in the zoning ordinances in 15 cities throughout the nation. Several other formulas using "net usable floor area" are available, but these figures are often a source of confusion and disagreement, and, therefore, were not considered. If the specifications had been applied to the Skokie study site, only four would have yielded adequate design values. Apparently most of these formulas are based on centrally oriented clinics where parking requirements per patient are relatively low.

TABLE 11

OFF-STREET PARKING REQUIREMENTS FOR MEDICAL CLINICS AS SPECIFIED IN 15 ZONING ORDINANCES (2)

City Number	Formula Specified	Estimated Spaces Required at Skokie[a]
2	G/200	101[b]
3	3D	66[b]
4	2D + E/2	62[b]
5	G/250	82[b]
6	G/1,000 + D + E/5	50
7	50 + ((G-20,000/300))	51
8	G/400	51
9	G/400	51
10	G/400	51
11	G/400	51
12	G/800	26
13	Sq Ft Parking: G/4	17[c]
14	6 + ((G-3,000/1,000))	24
15	6 + ((G-3,000/1,000))	24
16	10 + ((G-10,000/1,000))	21

[a]Requirements based on G = gross sq footage of building, 20,300 sq ft; D = no. of doctors, 22; E = no. of employees, 35.
[b]Only adequate values.
[c]Assuming 300 sq ft per parking space.

LIMITATIONS AND COMPARISON WITH OTHER STUDIES

Available time and manpower limited the amount of data collected. Further data regarding traffic flow, travel modes, and patient loads are needed. More sites should be studied. Some medical specialties are not considered in this study and data on these are needed.

Besides the formulas given in Highway Research Board Bulletin 99 (2), some more recent work has been done in relating gross floor area to parking demand for office buildings. Box (3) found indicated design values ranging from one space per 220 sq ft to one space per 250 sq ft for large office buildings in the suburban Chicago area. He also states "... it is probable, for example, that smaller office buildings, serving a variety of professional tenants such as doctors, dentists, lawyers, architects, and engineers, would have somewhat different parking demands." Although clinic doctors are not professional tenants but actual associates of the clinic, the above design values were applied to the Skokie study site with the following results:

$$(1 \text{ space}/220 \text{ sq ft}) \times 20,300 \text{ sq ft} = 93 \text{ spaces} \quad (1)$$

$$(1 \text{ space}/250 \text{ sq ft}) \times 20,300 \text{ sq ft} = 82 \text{ spaces} \quad (2)$$

Compared with the computed value of 78 spaces, these appear slightly high but certainly reasonable. Perhaps medical clinics do generate a slightly lower demand per square foot of floor area than office buildings. At any rate, this criterion yields more reasonable design values than the earlier formulas given in Table 11.

To better understand traffic flows, travel modes, and directional splits, it may be prudent to determine the area of attraction commanded by a medical clinic. For the particular site studied here, area of attraction could have been determined through examination of billing slips, which give the addresses of practically all patients.

CONCLUSIONS

Recognizing that this study was limited to only two locations, the following general conclusions are indicated:

1. Parking and traffic demands at medical clinics and offices vary not only with floor space and number of doctors but also with location in the community, services available for treatment and diagnosis, scheduling procedures, and other operational aspects of the facility.
2. Wide variations exist between medical specialties in the rate at which patients are treated, ranging from an average of more than 3.5 pph in the case of pediatricians, for example, down to fewer than 1.5 pph for general surgeons.
3. For suburban locations of the type studied, approximately four to five parking spaces should be provided for each doctor in attendance. The figures should be modi-

fied where necessary to reflect differences in location of the facility, compostion of the medical staff, and operational practices.

4. Many of the older formulas now still in use will not provide sufficient parking space if applied without modification to suburban clinics.

5. The effect on parking demand of location of the facility in the community is apparent in the greater use of the private auto for travel to the suburban clinic as compared to auto usage associated with the office building in the CBD. Approximately 82 percent of the patients using the suburban clinic required parking, in contrast to 67 percent for the downtown medical office building. Comparable figures for employees were 74 percent and 9 percent, respectively.

REFERENCES

1. "The Modern Hospital." 87:1 (July 1956).
2. "Parking Requirements in Zoning Ordinances." HRB Bull. 99 (1964).
3. Box, P., "Parking Generation Studies." HRB Abs., 32:17-23 (April 1962).

Significant Visual Properties of Some Fluorescent Pigments

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High object visibility is a necessary characteristic of traffic control devices and a significant factor in highway safety. Fluorescent pigments possess unique physical properties that provide high visibility characteristics not provided by conventional pigments. As a result, fluorescent colors are now used for safety markings and to a limited extent in the traffic field. This study compares the daylight visibility properties of fluorescent and conventional pigments.

Fluorescent and conventional pigments have substantially different spectral energy radiation patterns. Fluorescent pigments absorb energy from the near visible ultraviolet blue and green region of the electromagnetic spectrum and reemit this energy in a very narrow band of the spectrum. Conventional pigments simply absorb and reflect incident light. The properties of a selected group of fluorescent and conventional pigments are shown, as well as the spectral response of the human eye and various source illumination distributions.

The field study considered variations of daylight energy distribution under clear and overcast sky conditions, representative solar altitudes, and the cardinal directions. Two fluorescent and four conventional high visibility pigments were viewed against representative backgrounds. Detection and identification of fluorescent pigments are comparable to conventional high visibility pigments under optimum viewing conditions; however, fluorescent pigments show a substantial improvement as illumination levels decrease or when the target situation is least advantageous.

•NUMEROUS STUDIES by Armed Forces research groups and others have established that, under natural illumination, objects marked with fluorescent pigments have greater average conspicuity than those marked with conventional pigments. Some of this research has been directed toward specific applied sitations, such as aircraft and life raft detectability (1, 2, 3, 4, 5), whereas Blackwell (6), Siegel and Crain (7, 8, 9), Cowling and Noonan (10), and Kazenas (11) have conducted work of a more basic nature. It is not within the scope of this paper to report the specific findings of all these research efforts.

The literature cited suggests that additional efforts be made to measure the improved visibility that arises from the unusual properties of fluorescent pigments. Improved visibility appears to depend on certain daylight illumination conditions and surround. Therefore, studies were conducted to determine visibility differences under conditions representative of the traffic environment.

The theory of visibility of achromatic targets and backgrounds does not adequately explain the established conspicuity of fluorescent targets. In his study of the effect of

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Paper sponsored by Committee on Traffic Control Devices.

color on target detectability, Blackwell (6) found that an empirical conspicuity factor was required to obtain agreement between predicted and observed target visibility. He states: "there is clear evidence that the chromatic samples are more visible than we would expect on the basis of reflectance alone." Middleton (12), however, concludes that because with increasing distance objects tend to become achromatic, "no special theory of the visual range of colored objects is necessary, and that colored marks will behave in the same way at the visual range as gray ones of the same luminance factor." Siegel and Lanterman (13) contend there is no clear theoretical indication that greater detectability can be expected from fluorescent paints. Judd and Wyszecky (14) point out that with targets of identical dominant wavelength and luminance factor, those of the greatest purity will appear brightest. These several views indicate recognition of an inherent dichotomy in the visual properties of fluorescent and conventional pigments.

This paper compares the properties of fluorescent and conventional pigments and presents results of field studies conducted to determine the magnitude of visibility differences existing between the two types of pigments.

ANALYSIS OF MATERIALS AND CONDITIONS

Six targets were selected for study. Four are commonly used high visibility conventional pigments, red, yellow, white, and international orange; two are fluorescent pigments, red-orange and yellow-orange. The colorimetric characteristics of the pigments are given in Table 1.

Conventional red is considered to afford the best contrast with the wide variety of colors found in nature (10). The particular hue chosen, insignia red, is Federal Standard No. 595-11136. The yellow used has high luminance and a wavelength approaching 555 mμ, the wavelength to which the average eye is most sensitive. White has a very high luminance and for this reason is frequently used as a high visibility marking. International orange offers an optimum balance between eye sensitivity and contrast with average backgrounds. It has a dominant wavelength matching the fluorescent yellow-orange studied, thus allowing direct comparison under identical viewing conditions.

The color properties of surfaces are graphically represented by reflectivity curves. Each of the target surfaces was examined with a Beckman DK-2 spectrophotometer and the percentage reflection of incident radiation, compared to that of a standard white surface, was calculated through the visible spectrum of radiation. The standard white surface is defined as having 100 percent reflectivity at all wavelengths. Reflectivity curves have been calculated under illumination by standard source "C" for the six target colors used (Fig. 1).

Conventional pigments work by a subtractive process in which certain wavelengths of incident energy are partially absorbed, and the remaining energy reflected. The reflectivity curves of fluo-

TABLE 1

COLORIMETRIC PROPERTIES OF TARGETS

Target	Luminance Factor (%)	Dominant Wavelength (m μ)	Excitation Purity (%)
White	94.7	476.0	1.0
Yellow	61.5	580.0	95.5
International orange	15.6	601.8	94.5
Red	10.4	660.0	57.0
Fluorescent red-orange	46.9	612.6	99.8
Fluorescent yellow-orange	68.6	602.8	99.9

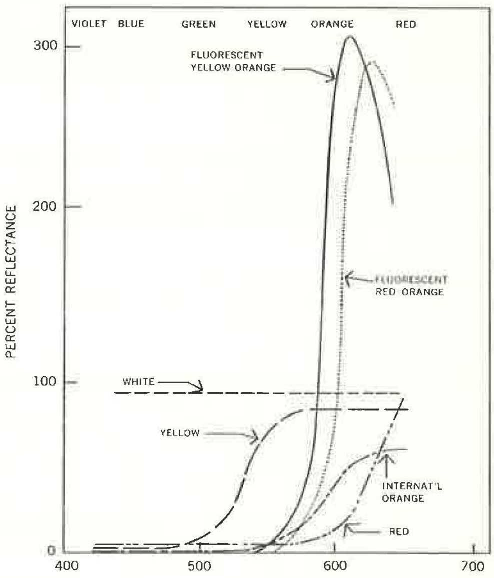

Figure 1. Reflectance curves of pigmented targets illuminated with source "C".

rescent pigmented materials show the strikingly different property of apparently returning more than 100 percent of the incident energy in a narrow spectral region. Reflectivity values can exceed 100 percent at a specific wavelength through the emission of energy absorbed at other wavelengths. This is precisely what fluorescent pigments do. Energy is absorbed in the near ultraviolet, blue and green regions of the spectrum, and is reemitted in the yellow-red region, thus adding to the energy also conventionally reflected.

The reflectance curves (Fig. 1) of yellow-orange fluorescent and international orange, which have similar dominant wavelengths, graphically illustrate the substantial gain afforded by fluorescent pigments. The energy shift noted in fluorescent pigments is characteristic of certain organic dyes having different absorption and fluorescent emission regions. The combined emitted and reflected energy, however, has only a single peak. This peak is the result of a "cascade effect" or progressive absorption and emission to the point of final energy emission. Emission curves for dyes used in fluorescent yellow-orange are shown in Figure 2.

The energy conversion process taking place in fluorescent pigments is known to have limited life. The pigments are selected primarily for their color and efficient energy conversion properties. However, recent technological advances provide improved protection to fluorescent dyes and have extended their useful life. The Armed Forces and independent industrial laboratories have established that the useful fluorescent life has been reached when the pigment loses 33 percent of its original brightness as measured on a NRL 45° fluorescence photometer (10). Fluorescent yellow-orange materials of high quality construction now have useful fluorescent life of 2 yr when exposed vertically, facing south, in Texas; fluorescent red-orange has a useful life of 2.5 yr under the same exposure conditions. Useful life of fluorescent materials is a direct function of the amount of solar radiation incident to the target surface; for this reason, exposure

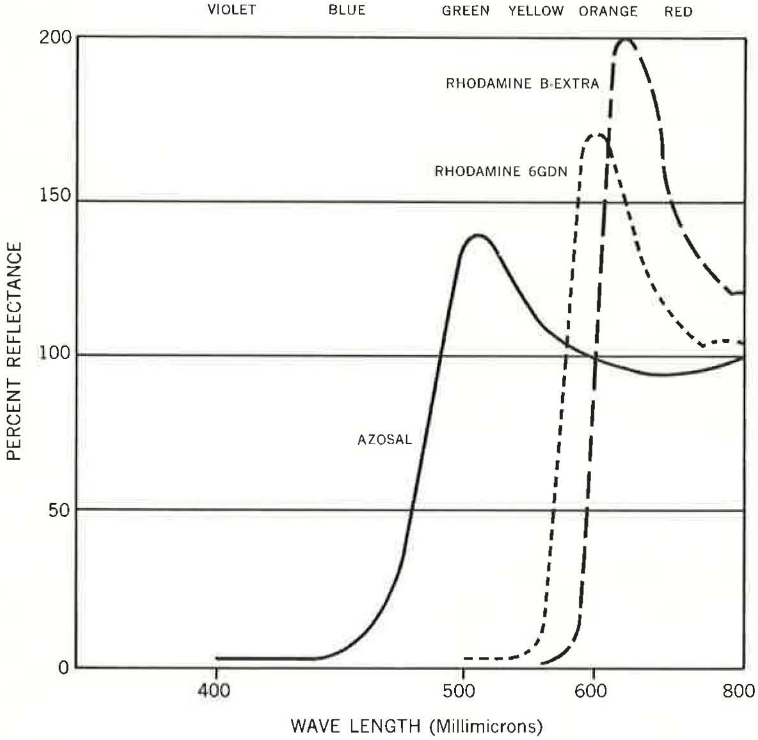

Figure 2. Emission characteristics of dyes required for fluorescent yellow-orange.

in directions other than south facing, or in more northerly latitudes, will result in greater useful life.

Incident Illumination Distribution

The fluorescent energy conversion process can be demonstrated by illuminating yellow-orange fluorescent and international orange targets on both white and black backgrounds using red, blue and unfiltered tungsten lamp light. With unfiltered tungsten light, as with daylight, both targets have good color and contrast with both backgrounds. With red light, both targets appear to have the same color and almost disappear on the white background. They appear white against the black background. With blue light, the international orange target disappears on the black background and appears black against the white background. The fluorescent target, however, shows its usual orange color and has good contrast with both backgrounds. This brightness is due to the conversion of the blue light to orange.

The observations of this demonstration are significant because natural illumination contains a greatly varying proportion of red and blue light under various directions, sky conditions and times of the day. The curves shown in Figures 3 and 4 indicate the relative spectral energy distributions of various sky conditions and several solar altitudes. Figure 3 compares the energy distributions of direct sunlight, overcast sky and north skylight. It can be seen that during the skylight condition, that is, when targets are in the shade, blue light is significantly predominant in the distribution. Figure 4 shows that on a clear day with the Sun at the zenith there is the greatest amount of total energy available and this energy is greatest in the blue region of the spectrum. Skylight is produced by the scattering of solar energy and contains a larger proportion of shorter wavelength (blue) than direct sunlight. It follows then that objects in the shade would be illuminated by greater proportions of blue light than objects illuminated directly by sunlight. As the Sun approaches the horizon, however, the blue component is filtered during its long atmospheric path and direct sunlight becomes relatively rich in red light.

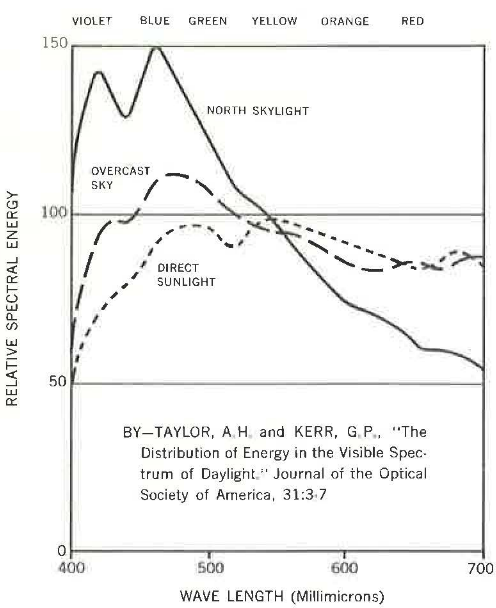

Figure 3. Relative spectral energy distribution of three phases of sunlight on basis of equal illumination.

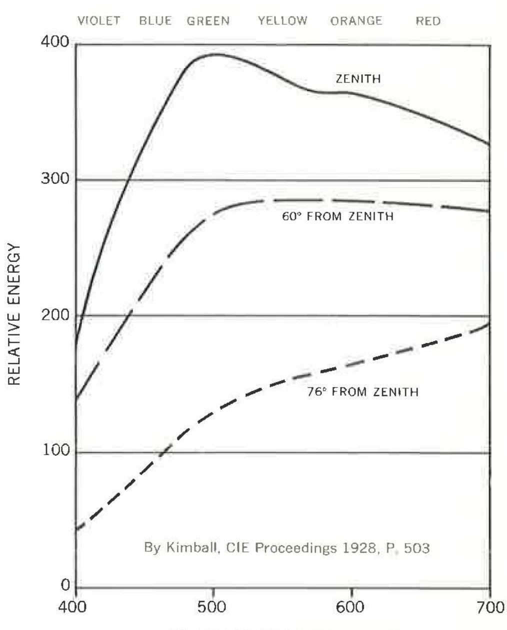

Figure 4. Spectral irradiation from sun.

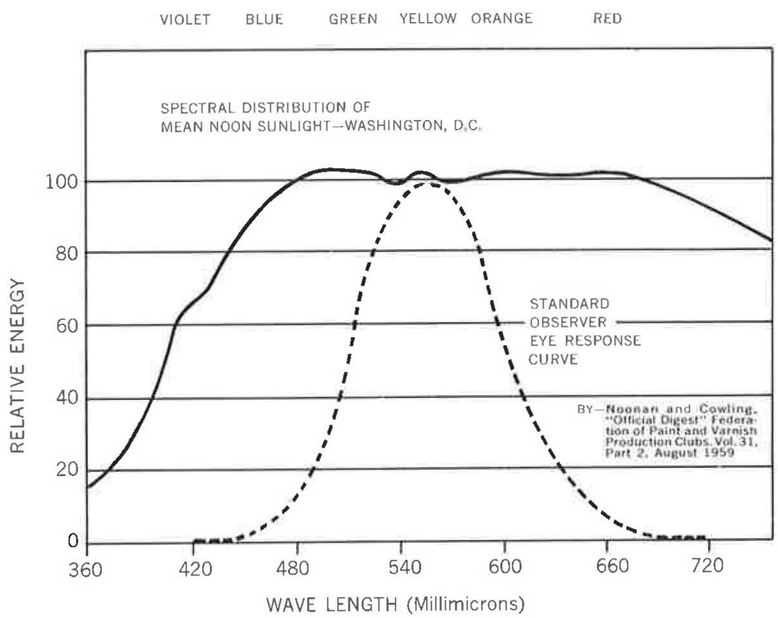

Figure 5. Comparison of spectral energy distribution with sensitivity response of average human eye.

Target-Background Contrasts

The conspicuity of a target is the measure of its effect on the viewer. The stimulus supplied to a viewer is usually measured in terms of the brightness alone with no consideration of color and is called the luminance factor. This factor considers the radiation coming from the target and the spectral sensitivity of the observer's eye. Colorimetrists have defined as standard observer (eye response) curve on the basis of many observations (Fig. 5). The eye sensitivity peaks at 555 mμ, the yellow-green region, and decreases toward both the red and blue regions.

The detectability of a target is also influenced by its brightness contrast with the background. Contrast ratio is the luminance factor of the target minus the luminance factor of the background divided by the luminance factor of the background (15). With constant intensity white illumination, each target would be detected by contrast alone. The detection distance of the targets would be in proportion to the contrast ratio. Sunlight, however, varies considerably from constant intensity white light, and one objective of this study is to evaluate the influence of solar illumination on both the distance at which the target is detected and the distance at which its hue is recognized.

FIELD STUDY

With this background information, a field study was designed that would take into account the necessary range of variables. The study considered the six targets previously discussed, three backgrounds, three time periods, and four directions, under two different sky conditions, using 19 adult male observers. The specific target size (circles 0.01 sq ft in diameter) was selected because nomographs (12) for predicting object detection distance use increments of area on a logarithmic scale and the predicted distances were appropriate for normal highway viewing distances.

Observers

In a test of vision it is essential that a significant number of observers be employed, because variations of response among observers, and by the same observer viewing the same target on different occasions, may be substantial. The average number of

TABLE 2

OBSERVER ACUITY

Acuity	No. Observers
20/17	14
20/18	6
20/20	5
20/22	2
20/29	1
Total	28

TABLE 3

COLORIMETRIC PROPERTIES OF BACKGROUND PANELS

Background	Luminance Factor (%)	Dominant Wavelength (m μ)	Excitation Purity (%)
White	82.6	567.0	2.0
Tan	34.9	581.2	44.6
Olive Drab	8.6	573.2	23.5

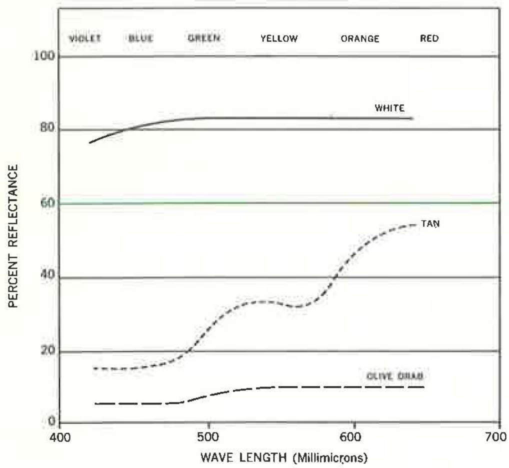

Figure 6. Reflectance curves of background materials.

TABLE 4

TARGET VS BACKGROUND CONTRAST RATIOS

Target	Background		
	Olive Drab	Tan	White
White	10.0	1.71	0.11
Fluorescent yellow-orange	6.96	0.96	-0.17
Yellow	6.15	0.76	-0.26
Fluorescent red-orange	4.45	0.34	-0.43
International orange	0.81	-0.55	-0.81
Red	0.21	-0.70	-0.87

observers used, 19, comprised a sufficiently large group to establish statistical reliability. Each observer was checked for visual acuity on a Bausch and Lomb Ortho Rater and for color blindness using a S. Ishihara color plate book. The observers had normal biocular acuity (Table 2). Of three observers with red-green color confusion, two were very mild, one more severe. Most observers participated in each set of observations.

Background Colors

The background colors selected were white, tan, and olive drab. Their colorimetric properties are given in Table 3; reflectance curves are shown in Figure 6. These three background colors were chosen because they offer not only representative maximum, intermediate and minimum brightness levels but also a variety of background colors encountered in nature: white represents snow, bright overcast sky and buildings; tan and olive drab represent the colors of fields, shrubbery and wooded backgrounds of many varieties. The tan and olive drab colors selected are U.S. Army Corps of Engineers' Standard Camouflage Colors (16) Nos. 6 and 9, respectively. Target vs background contrast ratios are given in Table 4.

Conduct of Test

Representative viewings by direction were obtained by facing the targets north, south, east and west. The study was conducted during three 1-hr time periods—noon, 3:00 PM, and 6:00 PM, with observations commencing ½ hr prior to the time period noted. The series is thus representative of solar altitudes for all daylight hours, because AM viewings would be for all practical purposes a duplication of PM viewings. Observations were made on Sept. 3 and 9 under two sky conditions, clear and solidly overcast. The observation conditions are, therefore, representative of the daylight range under which devices employing the target materials would be used.

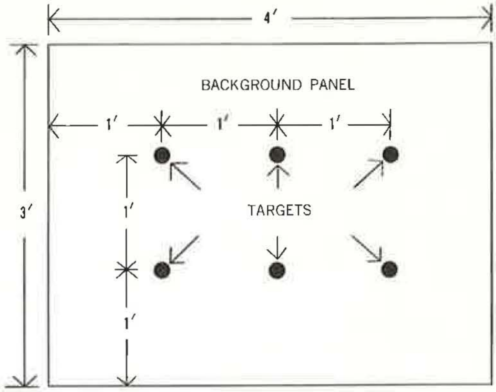

Figure 7. Background panel and target layout.

The six target colors, 0.01 sq ft circles, were placed in a random sequence on panels of the three background colors. The panels (Fig. 7) 3- by 4-ft in size, were mounted on top of a stationary automobile and presented to the observers one at a time in a random manner. Observations began from a distance of 2,000 ft, at which no target was detectable. Observers approached the panels in automobiles, traveling at a speed of 5 mph, and recorded two distances, the distance at which each of the targets became visible, or detection range and the distance at which each target could be identified by chromatic hue, or recognition range. Approaches were made on the east-west range until all three backgrounds had been viewed in each direction. The observers then followed the same procedure on the north-south range. A total of 16,400 individual observations were made.

RESULTS AND ANALYSIS

Mean detection and recognition distances are shown for each direction, time period, sky condition and background in Appendix A. Additionally, mean distances were calculated for each target by day for time of day, direction and background, and combined for both days. The over-all means for the entire study were also computed for each target by day and for both days.

Inspection of the over-all means in Table 5 shows that the fluorescent yellow-orange target was detected at the greatest distance and that is was recognized by hue identification first on both the overcast and the clear sunny day. The mean differences of detection and recognition ranges existing between fluorescent yellow-orange and any other target were statistically significant. All targets were detected at a greater distance on the sunny day; however, the loss in detectability distance of the overcast day is greater for conventional pigments than for fluorescent pigments. This indicates that fluorescent pigments provide visibility properties less sensitive to reductions in illumination. Although international orange and fluorescent yellow-orange have similar dominant wavelengths and high excitation purities (Table 1), a substantial difference exists in both recognition and detection range. The target with the greatest detection range, fluorescent yellow-orange, is followed by yellow. It is noteworthy that both have similar luminance factors, dominant wavelengths and excitation purity. The supe-

TABLE 5

MEAN DETECTION, RECOGNITION RANGES AND RANK ORDER OF 0.01-SQ FT CIRCULAR TARGETS[a]

Target	Both Days				Overcast Day				Clear Sunny Day			
	Detection		Recognition		Detection		Recognition		Detection		Recognition	
	Range (ft)	Rank	Range (ft)	Rank	Range (ft)	Rank	Range (ft)	Rank	Range (ft)	Rank	Range (ft)	Rank
Yellow	570	2	315	4	553	2	311	4	587	2	319	4
Fluorescent red-orange	556	4	394	2	545	3	391	2	567	4	396	2
International orange	505	5	242	5	490	5	242	5	519	5	242	5
Red	489	6	190	6	476	6	192	6	502	6	187	6
White	559	3	342	3	537	4	345	3	581	3	338	3
Fluorescent yellow-orange	604	1	441	1	595	1	438	1	612	1	443	1

[a]Over-all means.

TABLE 6

MEAN DETECTION AND RECOGNITION RANGES OF 0.01-SQ FT CIRCULAR TARGETS[a]

Target	Range (ft)					
	Both Days		Overcast Day		Clear Sunny Day	
	Detection	Recognition	Detection	Recognition	Detection	Recognition
North facing:						
Yellow	563	308	566	311	559	305
Fluorescent red-orange	557	398	557	402	557	394
International orange	488	206	491	204	484	207
Red	476	134	488	139	464	130
White	547	355	536	353	558	317
Fluorescent yellow-orange	607	457	617	460	596	453
East facing:						
Yellow	507	301	558	335	455	266
Fluorescent red-orange	513	382	559	428	467	336
International orange	454	209	498	243	410	175
Red	432	160	475	190	388	130
White	489	332	509	357	468	306
Fluorescent yellow-orange	540	436	602	480	477	391
South facing:						
Yellow	633	344	611	321	654	366
Fluorescent red-orange	620	457	587	437	632	476
International orange	543	236	508	223	578	248
Red	520	164	476	153	564	174
White	615	369	592	360	638	378
Fluorescent yellow-orange	689	524	678	514	699	533
West facing:						
Yellow	662	366	592	344	731	387
Fluorescent red-orange	636	475	596	456	676	493
International orange	565	284	525	277	605	290
Red	538	216	505	210	570	222
White	655	421	588	404	722	437
Fluorescent yellow-orange	699	541	651	518	747	564

[a]Means by direction.

riority of fluorescent yellow-orange is more pronounced when the recognition ranges are compared, indicating the relative importance of high reflectance (Fig. 1).

Target Comparison by Direction

Table 6 presents the data for the over-all averages by direction. It is apparent that fluorescent yellow-orange has the greatest detection and recognition range of the targets studied. The mean difference in recognition range between fluorescent yellow-orange and the other targets is substantial, whereas the differences in mean detection ranges are not all significant. These differences are compared graphically (Fig. 8).

Target Comparison by Background Color

Table 7 gives the mean target detection and recognition ranges by background color. There is statistical significance between practically all mean detection range differences on any given background, and the range of the target depends on the background being considered. The fluorescent yellow-orange target has the greatest over-all detection range, although it is not greatest on any particular background. The factors influencing detection range are luminance contrast ratio, color and reflectance.

The mean recognition ranges of the fluorescent targets are significantly greater than those of the conventional targets. The recognition distance varies directly as the background color becomes darker (Fig. 9). An analysis of variance (Appendix B) of the variables considered in this study confirms that recognition range is closely dependent on background.

Target Comparison by Time of Day

Table 8 gives the mean detection and recognition ranges for the three time periods during which observations were made. Fluorescent yellow-orange has the greatest mean detection and recognition ranges. The differences in these ranges between fluo-

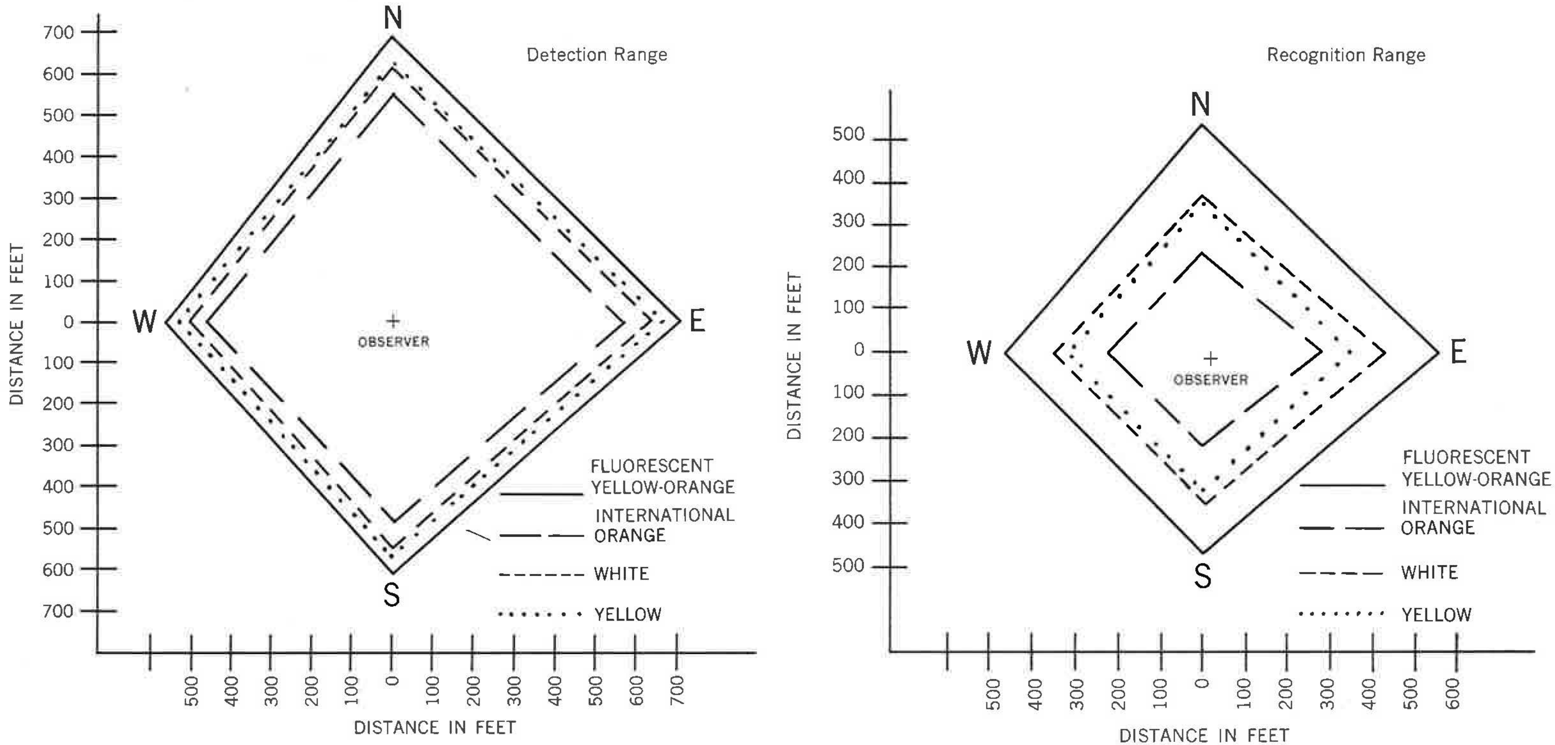

Figure 8. Mean detection and recognition ranges of 0.01 sq ft circular targets by direction.

TABLE 7

MEAN DETECTION AND RECOGNITION RANGES OF 0.01-SQ FT CIRCULAR TARGETS[a]

Target	Range (ft)					
	Both Days		Overcast Day		Clear Sunny Day	
	Detection	Recognition	Detection	Recognition	Detection	Recognition
White background:						
Yellow	400	240	394	234	406	246
Fluorescent red-orange	572	346	591	340	592	352
International orange	613	182	589	183	636	180
Red	611	134	602	143	630	125
White	143	87	142	87	144	86
Fluorescent yellow-orange	504	363	494	356	513	369
Tan background:						
Yellow	490	317	486	320	493	314
Fluorescent red-orange	479	347	526	345	432	346
International orange	479	249	472	250	486	247
Red	528	204	515	207	542	203
White	693	442	655	437	730	446
Fluorescent yellow-orange	521	413	525	418	517	407
Olive drab background:						
Yellow	825	391	781	381	866	400
Fluorescent red-orange	671	490	663	489	678	491
International orange	420	296	408	293	431	299
Red	319	230	309	225	329	234
White	847	499	813	509	881	488
Fluorescent yellow-orange	789	549	767	542	810	555

[a]Means by background.

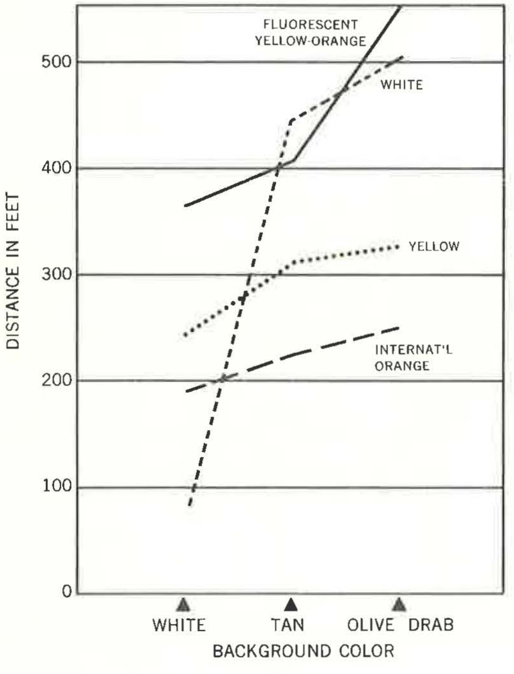

Figure 9. Recognition ranges of 0.01 sq ft circular targets by background color.

rescent yellow-orange and the other targets are statistically significant for all conditions except for the detection range on a clear sunny day at 3 PM. Figure 10 shows graphically the data for several of the targets.

Further Analyses

The analysis of variance (Appendix B) shows that of the three variables—time, direction and background—time had the least significant effect. To examine the data more closely, a specific time was selected, held constant, and the data for the remaining two variables compared. Figures 11 and 12 give the detection and recognition ranges for chromatically comparable targets. Distances are given for south facing targets on each background. The detection ranges indicate that distance is primarily a function of luminance contrast. The recognition ranges show a definite advantage for fluorescent yellow-orange. The differences in mean recognition range are significant.

In the previous discussion concerning source distribution and its effect on fluorescent and conventional pigments it was indicated that fluorescent pigments would be more advantageous during conditions of predominant blue light (overcast or illumination by skylight) than when red light is predominant. The following analysis supports this conclusion. The recognition range of fluorescent yellow-orange was compared to those of international orange and conventional yellow under four specific conditions:

TABLE 8

MEAN DETECTION AND RECOGNITION RANGES OF 0.01-SQ FT CIRCULAR TARGETS[a]

Target	Range (ft)					
	Both Days		Overcast Day		Clear Sunny Day	
	Detection	Recognition	Detection	Recognition	Detection	Recognition
(a) Noon, CST						
Yellow	609	350	581	348	636	351
Fluorescent red-orange	593	438	574	442	611	433
International orange	541	250	518	263	563	236
Red	505	188	497	210	513	165
White	595	384	548	375	641	393
Fluorescent yellow-orange	641	498	628	498	654	498
(b) 3 PM, CST						
Yellow	633	345	642	344	621	345
Fluorescent red-orange	615	454	621	449	609	458
International orange	548	240	542	239	556	240
Red	531	163	518	160	543	165
White	621	397	614	402	627	392
Fluorescent yellow-orange	670	517	684	511	655	523
(c) 6 PM, CST						
Yellow	531	292	523	289	538	294
Fluorescent red-orange	528	393	530	403	525	382
International orange	443	209	449	204	437	214
Red	436	154	437	145	434	162
White	511	307	501	323	520	291
Fluorescent yellow-orange	587	451	593	467	580	434

[a]Means by time of day.

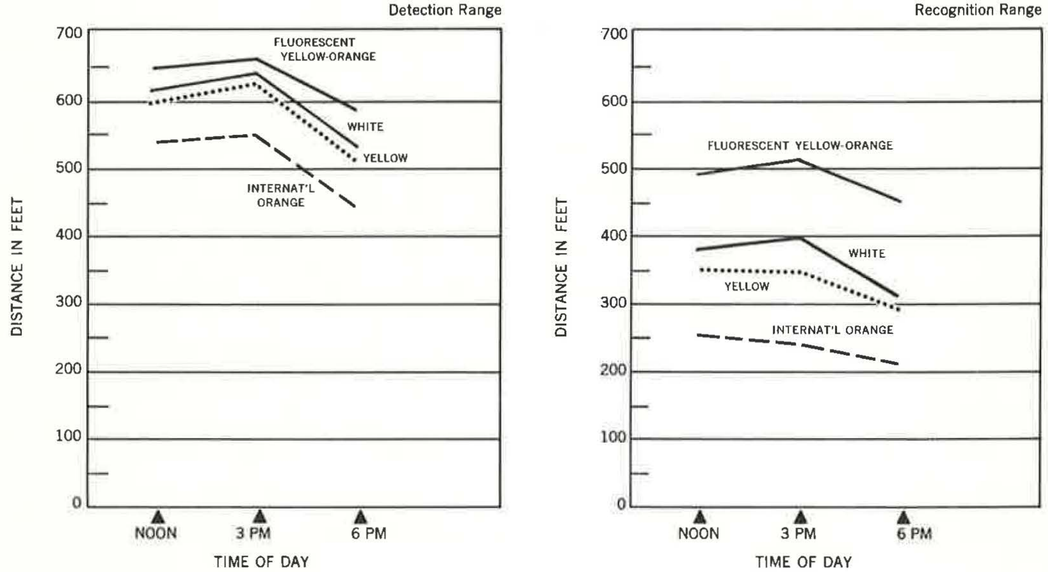

Figure 10. Mean detection and recognition ranges of 0.01 sq ft circular targets by time of day.

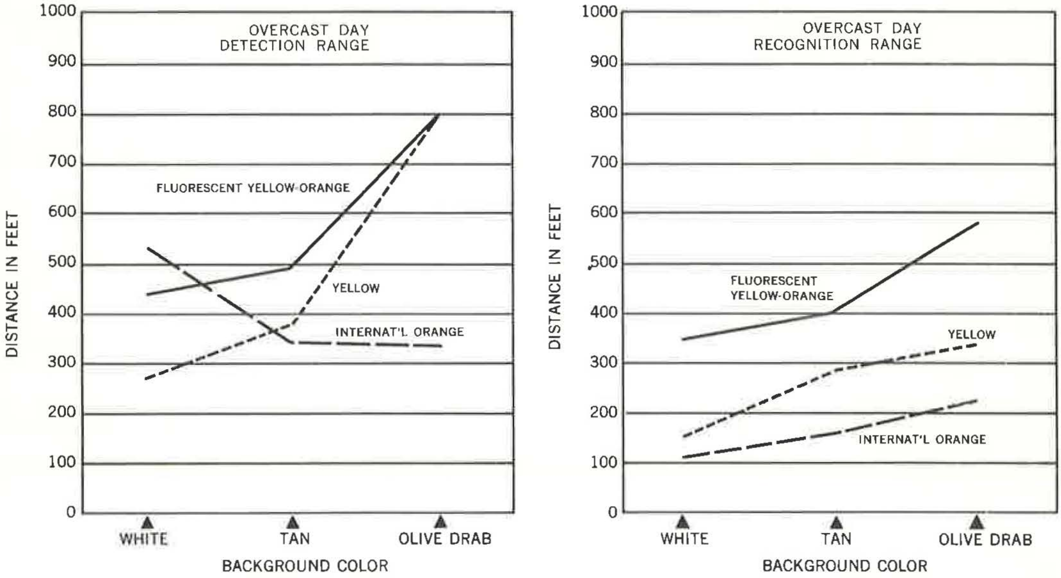

Figure 11. Mean detection ranges of 0.01 sq ft circular targets at 6 PM with targets facing south on overcast day.

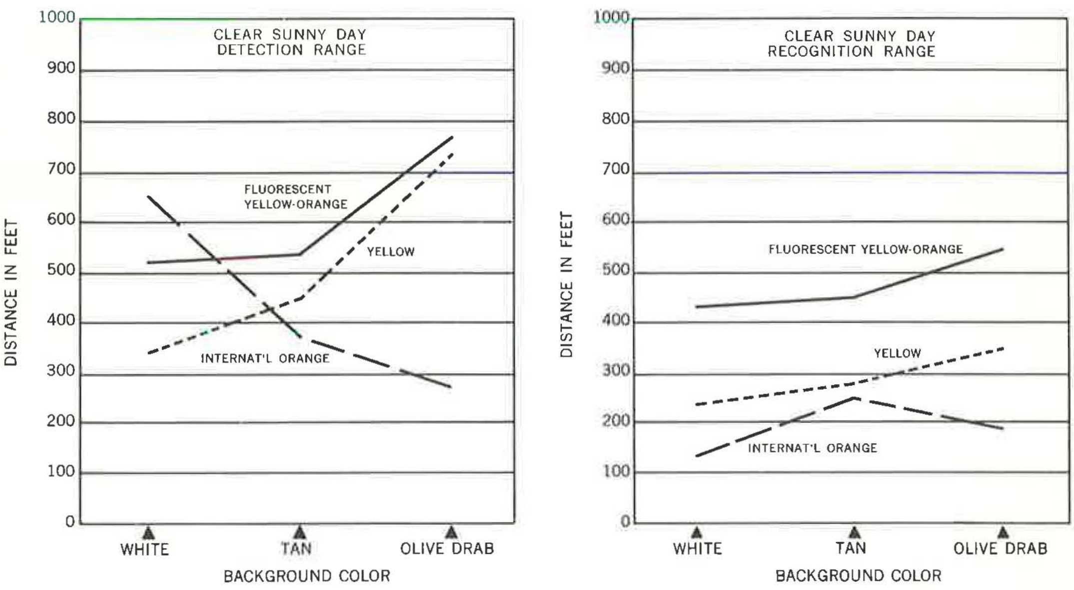

Figure 12. Mean recognition ranges of 0.01 sq ft circular targets at 6 PM with targets facing south on clear sunny day.

TABLE 9

RECOGNITION RANGE DISTANCES AND RATIOS

Condition	Predominant Illumination	Recognition Range				
		Fluorescent Yellow-Orange (ft)	International Orange (ft)	Ratio[a]	Conventional Yellow (ft)	Ratio[b]
South facing at 6 PM	Blue	523	187	2.8	334	1.6
East facing at 3 PM	Blue	500	193	2.6	278	1.8
South facing at noon	Red	810	404	2.0	600	1.3
West facing at 6 PM	Red	742	442	1.7	572	1.3

[a]Fluorescent yellow-orange to international yellow.
[b]Fluorescent yellow-orange to conventional yellow.

1. South facing on a clear sunny day at 6 PM with the targets in the shade and illuminated by skylight—a condition of predominant blue illumination;
2. East facing on a clear sunny day at 3 PM with target illumination by skylight—a condition of predominant blue illumination;
3. South facing on a clear sunny day at noon with direct sunlight—a condition of predominant red illumination;
4. West facing on a clear sunny day at 6 PM with direct sunlight—a condition of the most predominant red illumination.

The olive drab background was used in all cases. Recognition ranges and their ratios for the targets are given in Table 9. The ratios illustrate the superiority of fluorescent yellow-orange to international orange and conventional yellow. The relative magnitude of superiority is directly dependent on the blue-red distribution of available natural illumination.

The higher luminance contrast ratio of conventional yellow provides greater recognition range than international orange and is also a color more commonly employed where high visibility is required. Although the fluorescent yellow-orange has slightly longer dominant wavelength and hue than conventional yellow, it has greater recognition range due to the increased luminance provided by the conversion process.

Target Size Extrapolation

The extrapolation of detection and recognition ranges for target sizes other than those studied is not a straight line arithmetic function of target size. This is due largely to the effect of atmospheric attenuation, which is the scattering of light caused by the presence of minute particles, such as dust, between the observer and the object. Attenuation does not, however, alter the rank order at which targets are seen if the size of targets being compared remains equal. Middleton (12) has published a series of nomographs clearly illustrating this principle. From this it can be reasonably assumed that the results of the present study would hold true for target sizes other than those studied.

SUMMARY

Well-established principles of colorimetry and vision were combined with measured properties of target materials and field studies to obtain quantitative differences of target visibility. A selection of common conventional and fluorescent target colors was compared against natural background colors under representative conditions of daylight illumination to obtain numerical values of their performance.

Comparison of the over-all means for the entire study indicated that the fluorescent yellow-orange target had detection and recognition ranges 6 and 29 percent greater, respectively, than any of the conventionally pigmented targets. The differences were statistically significant. Fluorescent yellow-orange had a recognition range 82 percent greater than international orange, its comparable conventional color.

A comparison of results by direction or time of day indicated that the fluorescent yellow-orange target had the greatest detection and recognition range. A comparison by background color establishes that contrast ratios were of primary importance in determining the rank order of detection range, with other factors, such as color contrast and target luminance, participating to a lesser extent. The only target with a consistently high detection range on all backgrounds was fluorescent yellow-orange.

Analysis of results under specific illumination conditions indicated that when blue light was predominant, the superiority of fluorescent pigments increased significantly. This illustrates the useful property of conversion of blue wavelength light to orange, inherent only in fluorescent colors. With the single exception of a white target on a tan background, fluorescent yellow-orange has the greatest recognition range for the targets and backgrounds studied. On an olive drab background the fluorescent yellow-orange target had a recognition range 3.8 times greater than the international orange target when both were in the shade at 6 PM, whereas this ratio was reduced to 2.0 when the targets were illuminated by direct sunlight at noon. Under the same conditions the fluorescent yellow-orange recognition range was 1.6 times greater than conventional yellow at 6 PM and 1.3 times greater at noon.

Under selected conditions other targets had slightly greater detection or recognition ranges; however, fluorescent yellow-orange was the only target with consistently high performance under all conditions and provided the best over-all performance. Of significance is the fact that as visibility conditions deteriorated the relative performance of fluorescent targets increased.

The principal reason for the superiority of fluorescent pigments is their unnaturally high color purity and reflectance resulting from an energy conversion process. This process causes the pigments to fade at a rate proportional to their exposure to sunlight. Recent improvements result in a useful life of 2 yr in southern states when facing south. Exposure in other directions or areas will result in a longer useful life.

The results of this study indicate that where high target visibility is the primary objective, fluorescent pigments should be given serious consideration.

REFERENCES

1. Siegel, A. I., "Aircraft Detectability and Visibility; IV. Detectability of Stimuli Painted with Fluorescent and Ordinary Paints when Viewed Against Clear and Cloudy Backgrounds." U. S. Naval Air Material Center, Philadelphia, NAMC-ACEL Rep. 460 (1961).
2. Federman, P., and Siegel, A. I., "Aircraft Detectability and Visibility; V. Detectability of Stimuli Coated with Fluorescent and Ordinary Paints, A Further Study." U. S. Naval Air Material Center, Philadelphia, NAMC-ACEL Rep. 470 (1962).
3. Halsey, Rita M., Curtis, Charles E., and Farnsworth, Dean, "Field Study of Detectability of Colored Targets at Sea." U. S. Navy, Bur. Medicine and Surgery, Medical Res. Lab. Rep. 2:65 (1955).
4. Richards, O., Woolner, R., and Panjian, A., "What the Well-Dressed Deer Hunter Will Wear." National Safety News.
5. Fitzpatrick, J. T., and Wilcox, R. S., "Properties of Daylight Fluorescent Color Systems Pertinent to the Consideration of Their Use on Navigation Aids." U. S. Coast Guard 6th Internat. Tech. Conf. on Lighthouses and Other Aids to Navigation, Washington, D. C. (1960).
6. Blackwell, R. H., "Visibility Assessment of Gray and Chromatic Paints by a New Technique." U. S. Coast Guard 6th Internat. Tech. Conf. on Lighthouses and Other Aids to Navigation, Washington, D. C. (1960).
7. Siegel, A. I., and Crain, K., "Aircraft Detectability and Visibility; I. Visual Fields for Fluorescent and Ordinary Paints." U. S. Naval Air Material Center, Philadelphia, NAMC-ACEL Rep. 440 (1960).
8. Siegel, A. I., and Crain, K., "Aircraft Detectability and Visibility; III. The Effects of Varying Stimulus Characteristics on Tachistoscopic Thresholds." U. S. Naval Air Material Center, Philadelphia, NAMC-ACEL Rep. 452 (1961).
9. Crain, K., and Siegel, A. I., "Aircraft Detectability and Visibility; II. Tachis-

toscopic Thresholds for Fluorescent and Ordinary Paints." U. S. Naval Air Material Center, Philadelphia, NAMC-ACEL Rep. 444 (1960).
10. Cowling, Jack S., and Noonan, Frank M., "Fluorescent High Visibility Paints for Aircraft." Offic. Dig. Fed. Paint Varnish Prod. Clubs (Aug. 1959).
11. Kazenas, Z., "Daylight Fluorescent Pigments and Their Uses in Coatings." Paint Ind. Mag. (Feb. 1960).
12. Middleton, W. E. Knowles, "Vision Through the Atmosphere." Univ. of Toronto Press (1958).
13. Siegel, A. I., and Lanterman, R. S., "Aircraft Detectability and Visibility; VI. A Qualitative Review and Analysis of the Utility of Fluorescent Paint for Increasing Aircraft Detectability and Conspicuity." U. S. Naval Air Material Center, Philadelphia, NAMC-ACEL Rep. 492 (1963).
14. Judd, D. B., and Wyszecky, G., "Color in Business Science and Industry." John Wiley and Sons, Inc., New York and London (1963).
15. Blackwell, R. H., "Specification of Interior Illumination Levels." Illum. Eng. (June 1959).
16. Breckenridge, R. P., "Modern Camouflage." Farrar & Rinehart, Inc., New York and Toronto (1942).

Appendix A

MEAN DETECTION AND RECOGNITION RANGES OF 0.01-SQ FT CIRCULAR TARGETS

Target	Background: White		Tan		Olive Drab	
	Detection Range (ft)	Recognition Range (ft)	Detection Range (ft)	Recognition Range (ft)	Detection Range (ft)	Recognition Range (ft)
(a) Overcast Day, 12 PM						
North facing:						
Yellow	420	254	408	291	736	382
Fluorescent Red-Orange	573	362	354	317	638	456
International Orange	651	125	397	216	336	257
Red	704	74	427	154	216	176
White	16	11	636	417	857	464
Fluorescent Yellow-Orange	478	369	488	416	737	511
East facing:						
Yellow	472	268	506	405	767	487
Fluorescent Red-Orange	616	433	459	403	645	550
International Orange	676	294	570	275	380	287
Red	716	260	564	314	266	205
White	27	26	719	577	850	627
Fluorescent Yellow-Orange	563	403	572	520	723	603
South facing:						
Yellow	423	201	545	362	1001	522
Fluorescent Red-Orange	597	378	473	404	797	639
International Orange	638	158	554	250	472	364
Red	599	104	623	172	373	281
White	51	39	826	562	965	545
Fluorescent Yellow-Orange	530	419	605	529	1001	707
West facing:						
Yellow	347	222	510	331	866	467
Fluorescent Red-Orange	558	348	431	417	757	605
International Orange	594	260	496	358	435	323
Red	576	223	588	319	287	245
White	111	82	675	583	928	617
Fluorescent Yellow-Orange	470	357	544	487	860	669
(b) Overcast Day, 3 PM						
North facing:						
Yellow	472	242	513	380	908	369
Fluorescent Red-Orange	680	394	468	367	675	508
International Orange	731	115	523	233	347	273
Red	778	69	527	160	275	180
White	45	22	760	521	1001	812
Fluorescent Yellow-Orange	628	403	567	480	812	573
East facing:						
Yellow	454	288	498	359	788	334
Fluorescent Red-Orange	608	390	428	366	682	507
International Orange	660	192	514	230	356	278
Red	703	106	535	177	225	188
White	62	53	639	457	873	526
Fluorescent Yellow-Orange	562	410	532	441	764	593
South facing:						
Yellow	368	232	582	323	1073	465
Fluorescent Red-Orange	646	393	437	373	836	607
International Orange	705	145	532	212	459	347
Red	698	96	582	156	285	225
White	33	17	812	446	1056	631
Fluorescent Yellow-Orange	559	403	578	469	1015	700
West facing:						
Yellow	376	264	554	375	1004	482
Fluorescent Red-Orange	644	375	480	425	832	649
International Orange	669	169	578	281	478	356
Red	672	73	659	204	343	271
White	60	39	844	580	1043	645
Fluorescent Yellow-Orange	544	410	601	504	990	701

MEAN DETECTION AND RECOGNITION RANGES OF 0.01-SQ FT CIRCULAR TARGETS (Cont'd.)

Target	Background					
	White		Tan		Olive Drab	
	Detection Range (ft)	Recognition Range (ft)	Detection Range (ft)	Recognition Range (ft)	Detection Range (ft)	Recognition Range (ft)
(c) Overcast Day, 6 PM						
North facing:						
Yellow	370	217	533	325	775	348
Fluorescent Red-Orange	544	357	420	369	680	510
International Orange	623	146	484	233	292	244
Red	712	100	523	181	187	180
White	11	9	748	488	821	521
Fluorescent Yellow-Orange	480	362	598	459	796	591
East facing:						
Yellow	366	230	410	272	653	340
Fluorescent Red-Orange	543	350	377	355	570	460
International Orange	616	153	412	211	285	231
Red	653	111	475	185	176	146
White	27	14	625	421	721	498
Fluorescent Yellow-Orange	469	362	510	433	645	508
South facing:						
Yellow	288	154	382	284	771	339
Fluorescent Red-Orange	488	321	352	320	688	510
International Orange	525	105	355	166	318	211
Red	542	50	428	132	165	128
White	2	2	562	375	845	508
Fluorescent Yellow-Orange	446	346	491	400	797	585
West facing:						
Yellow	305	209	481	342	938	390
Fluorescent Red-Orange	575	362	376	340	752	606
International Orange	618	158	415	242	431	336
Red	634	118	481	204	246	225
White	41	7	648	486	942	569
Fluorescent Yellow-Orange	484	414	511	460	901	688
(d) Clear Sunny Day, 12 PM						
North facing:						
Yellow	421	261	376	261	747	338
Fluorescent Red-Orange	705	408	279	251	522	375
International Orange	749	144	343	178	274	224
Red	666	88	382	97	148	123
White	37	30	647	403	881	472
Fluorescent Yellow-Orange	497	358	439	337	635	494
East facing:						
Yellow	580	251	397	305	892	468
Fluorescent Red-Orange	736	365	467	384	681	529
International Orange	727	147	678	201	337	283
Red	551	95	652	175	341	255
White	298	131	768	503	926	573
Fluorescent Yellow-Orange	582	385	516	462	796	612
South facing:						
Yellow	433	251	607	430	1245	600
Fluorescent Red-Orange	685	388	535	438	877	711
International Orange	746	169	687	298	532	404
Red	712	75	770	223	385	319
White	13	8	1008	619	1277	781
Fluorescent Yellow-Orange	585	430	592	505	1157	810
West facing:						
Yellow	437	266	506	334	966	443
Fluorescent Red-Orange	663	365	407	392	755	578
International Orange	729	171	513	274	437	335
Red	688	87	604	226	260	212
White	28	26	849	576	924	579
Fluorescent Yellow-Orange	551	426	603	503	879	635
(e) Clear Sunny Day, 3 PM						
North facing:						
Yellow	451	282	413	299	872	378
Fluorescent Red-Orange	653	414	364	345	666	517
International Orange	734	146	440	213	351	268
Red	763	96	493	126	206	162
White	41	32	759	487	993	542
Fluorescent Yellow-Orange	551	428	500	453	828	610
East facing:						
Yellow	443	260	386	278	629	278
Fluorescent Red-Orange	551	380	461	323	513	404
International Orange	636	138	384	199	245	193
Red	682	114	420	121	153	112
White	68	59	618	421	722	478
Fluorescent Yellow-Orange	453	391	458	418	622	500
South facing:						
Yellow	427	258	518	368	1175	539
Fluorescent Red-Orange	656	415	523	443	823	689
International Orange	742	180	675	268	468	364
Red	767	94	743	194	397	317
White	71	43	968	602	1102	635
Fluorescent Yellow-Orange	588	453	544	481	1052	767

MEAN DETECTION AND RECOGNITION RANGES OF 0.01-SQ FT CIRCULAR TARGETS (Cont'd.)

Target	Background					
	White		Tan		Olive Drab	
	Detection Range (ft)	Recognition Range (ft)	Detection Range (ft)	Recognition Range (ft)	Detection Range (ft)	Recognition Range (ft)
(e) Clear Sunny Day, 3 PM (Cont'd.)						
West facing:						
Yellow	385	245	610	411	1234	579
Fluorescent Red-Orange	619	383	531	451	948	760
International Orange	721	212	655	296	555	422
Red	647	92	748	237	401	333
White	83	75	1005	677	1254	752
Fluorescent Yellow-Orange	552	409	617	547	1149	856
(f) Clear Sunny Day, 6 PM						
North facing:						
Yellow	462	265	446	313	868	356
Fluorescent Red-Orange	666	375	408	365	759	510
International Orange	629	172	464	241	354	284
Red	768	113	498	189	233	186
White	165	117	596	367	955	426
Fluorescent Yellow-Orange	542	414	549	430	855	572
East facing:						
Yellow	187	150	232	193	331	197
Fluorescent Red-Orange	260	189	202	189	302	252
International Orange	278	134	184	138	184	138
Red	318	66	209	122	131	100
White	92	91	301	228	388	254
Fluorescent Yellow-Orange	215	189	270	247	363	301
South facing:						
Yellow	344	237	438	279	722	334
Fluorescent Red-Orange	576	402	385	339	588	454
International Orange	640	131	381	245	278	187
Red	630	88	493	172	120	97
White	19	18	665	351	742	411
Fluorescent Yellow-Orange	508	414	528	431	764	523
West facing:						
Yellow	288	225	748	412	1356	572
Fluorescent Red-Orange	641	398	530	463	993	647
International Orange	698	167	450	289	700	442
Red	675	114	547	297	576	400
White	62	30	1034	600	1242	600
Fluorescent Yellow-Orange	554	422	613	529	1201	742

Appendix B

ANALYSIS OF VARIANCE FOR MEAN DETECTION AND RECOGNITION RANGES OF 0.01-SQ FT CIRCULAR TARGETS

Variable	Overcast Day			Clear Sunny Day		
	Degrees of Freedom	Mean Square		Degrees of Freedom	Mean Square	
		Detection Range	Recognition Range		Detection Range	Recognition Range
Direction target faced	3	455,633	465,207	3	10,860,200	3,642,490
Time of day	2	2,635,100	846,535	2	4,283,700	1,349,475
Background color	2	9,993,200	14,564,190	2	14,023,150	16,725,825
Target color	5	1,726,000	8,111,802	5	1,827,240	9,924,018
Direction × time	6	429,900	215,770	6	4,741,950	1,127,103
Direction × background	6	649,617	179,105	6	2,118,517	734,521
Time × background	4	44,300	46,402	4	101,075	261,798
Direction × target color	15	37,773	24,994	15	61,520	54,378
Time × target color	10	23,260	45,196	10	40,100	109,079
Background × target color	10	12,418,190	1,900,834	10	15,976,690	1,823,426
Direction × time × background	12	102,767	42,042	12	611,700	254,004
Direction × time × target color	30	87,867	17,052	30	37,627	24,342
Direction × background × target color	30	317,867	18,943	30	271,953	31,924
Time × background × target color	20	27,865	21,774	20	183,480	41,454
Error	3,253	13,073	21,870	4,014	12,547	15,542
Total	3,407			4,169		

Some Measurable Qualities of Traffic Service Influenced by Freeways

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Data is evaluated on travel time and fuel consumption, not a frequent or easily obtainable measurement. In addition, traffic volume relief is presented for one study area to reflect non-user benefits in the form of accessibility and also to present an insight into significant changes in travel time and fuel consumption.

The first portion of the report presents the characteristics of benefits in the form of travel time, overall speed, delay, fuel consumption, distance, and volume for freeway bypass routes as compared to the older business routes.

The rest of the report analyzes the effect on travel time and fuel consumption of speed of operation and traffic volume. The economics of movement of vehicles at various speeds is also investigated to establish the most efficient speed on the basis of values of travel time and fuel.

The results of the freeway bypass evaluation show that the economic benefit as a result of the construction of a freeway will always be a positive quantity on the basis of travel time savings, even in cases where the freeway distance is 12 percent greater than the older business route. However, the fuel consumption may be negative depending on route distances, character of the speed change and the type of vehicle. By combining these two economic benefits the optimum economic speed of operation is indicated.

•RESEARCH on the operating characteristics of vehicles, particularly fuel consumption and travel time, has been conducted at the University of Washington for the past 5 yr. In the summer of 1962, the University of Washington research group entered into a contract with the Washington State Highway Commission and the U. S. Department of Commerce Bureau of Public Roads for measuring the fuel consumption, travel time, and delay experienced on an existing congested primary State highway between the cities of Seattle and Tacoma. Extensive measurements were made for various volume conditions and types of vehicles, ranging from a small compact to a large tractor and semitrailer combination. This 12.5-mi section of highway has been replaced by a 6-lane freeway, and subsequent measurements of fuel consumption and travel time have been made on the freeway and then again on the old section of the State highway. The primary purpose of the contract is to collect and assemble data on time, fuel, and volume on the existing route and also on routes in the City of Seattle before any segments of the freeway network are completed and open to traffic. These data will provide the "before" portion of a before-and-after study of freeway benefits. The contract also includes a limited "after" study because a 12.8-mi section of freeway between Seattle and Tacoma was opened to traffic during the period of the contract. Preliminary results are presented in this paper for the trip savings by vehicle type.

Paper sponsored by Committee on Quality of Traffic Service.

In addition, data has been collected on travel time and fuel consumption on freeway sections bypassing the City of Olympia and on the previous State highway routes through the business section of Olympia. This report presents the preliminary results of user benefits of savings in time and fuel consumption for a standard passenger car trip.

It is the intent in the first portion of the paper not to present new techniques of data collection but to provide factual information on the magnitude of benefits realized by the improvement of existing congested arterial routes in comparison to the operation of free-flowing traffic on freeways.

The latter portion of the paper attempts to correlate meaningful measurements important in the statistical analysis of field measurements. In addition, a preliminary investigation is presented to stimulate additional interest in a more precise evaluation of items directly related to the quality of traffic service.

TEST ROUTE DESCRIPTION

The research contract required the collection of data on six routes on the Olympia test site, two on the Seattle-Tacoma site and four on the Seattle test site. This report utilizes the first two test sites.

Olympia Test Site

On Dec. 12, 1958, the Washington State Highway Commission opened a section of US 99 freeway bypassing the City of Olympia. In addition, a section of US 101 freeway, bypassing Olympia to the west, was also opened to traffic. These two freeway sections resulted in bypasses in three general directions, i.e., south to east and east to south for US 99, south to west and west to south for US 101, and east to west and west to east for US 410. The study routes for these bypass and business routes are shown in Figures 1 and 1A. Figure 1 shows the data checkpoints for the 1958 before study and the 1959 after study. Figure 1A shows the 1963 checkpoints for the freeway and business routes.

Seattle-Tacoma Test Site

The first segment of the freeway between Tacoma and Seattle to be opened to traffic was the portion from the Port of Tacoma road to Midway at SSH-5A. This freeway route is 12.846 mi in length for Seattle-bound traffic and 12.746 mi for Tacoma-bound traffic. Figure 2 shows the general vicinity of the test site location, which also includes the old highway route of the primary State highway consisting of a 4-lane undivided highway in some sections and channelized for turn movements and access controls at other locations. The old route was equipped with fully actuated traffic signals at four locations.

TEST VEHICLE DESCRIPTION

The Olympia study in March of 1963 utilized a 1963 standard V8 4-door sedan. The vehicle is test unit 6 and nearly identical with test unit 2 (Fig. 3).

The Seattle-Tacoma routes utilized five different vehicle types, including a 1962 U. S. compact 4-door sedan, a 1962 standard V8 4-door sedan, a 1962 half-ton 6-cylinder pickup truck, a 1955 Diesel 3-S2, and a 1962 single unit truck with dual tires (Fig. 3). Detailed descriptions of the test vehicles are contained in Appendix A.

The after study on the Tacoma-Seattle route was conducted in December 1962 and utilized test units 1 and 2 (compact and standard passenger car, respectively).

DESCRIPTION OF DATA COLLECTION

This report utilizes the data collected on the US 99 Olympia bypass, which is of major significance from the standpoint of relief to traffic congestion for graphical presentation. Preliminary results are also tabulated for the other routes. The Washington State Highway Commission in November and December 1958 collected travel time and fuel consumption on the existing city business routes and in March 1959 collected after

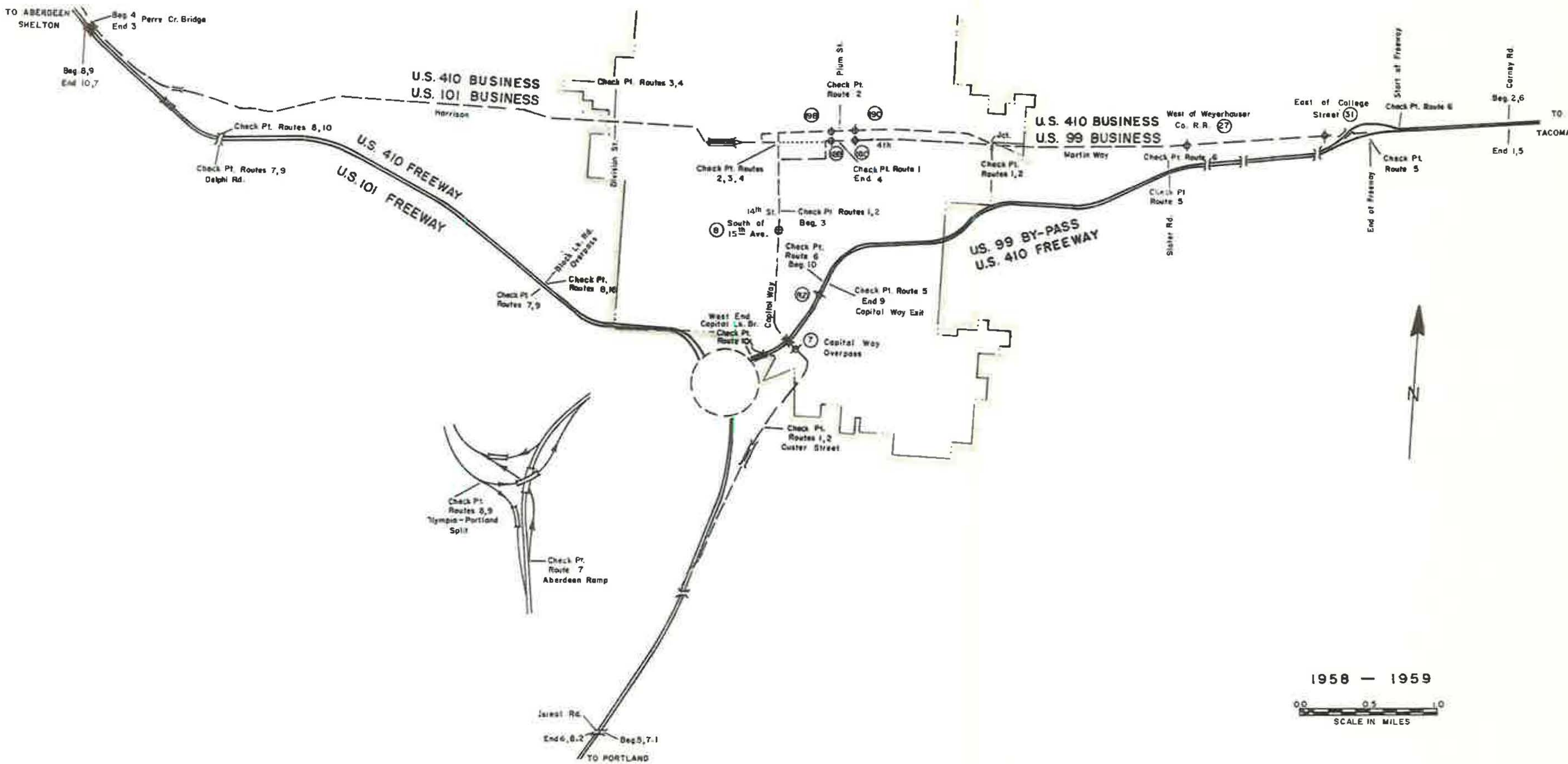


Figure 1. Olympia study routes, 1958-1959.

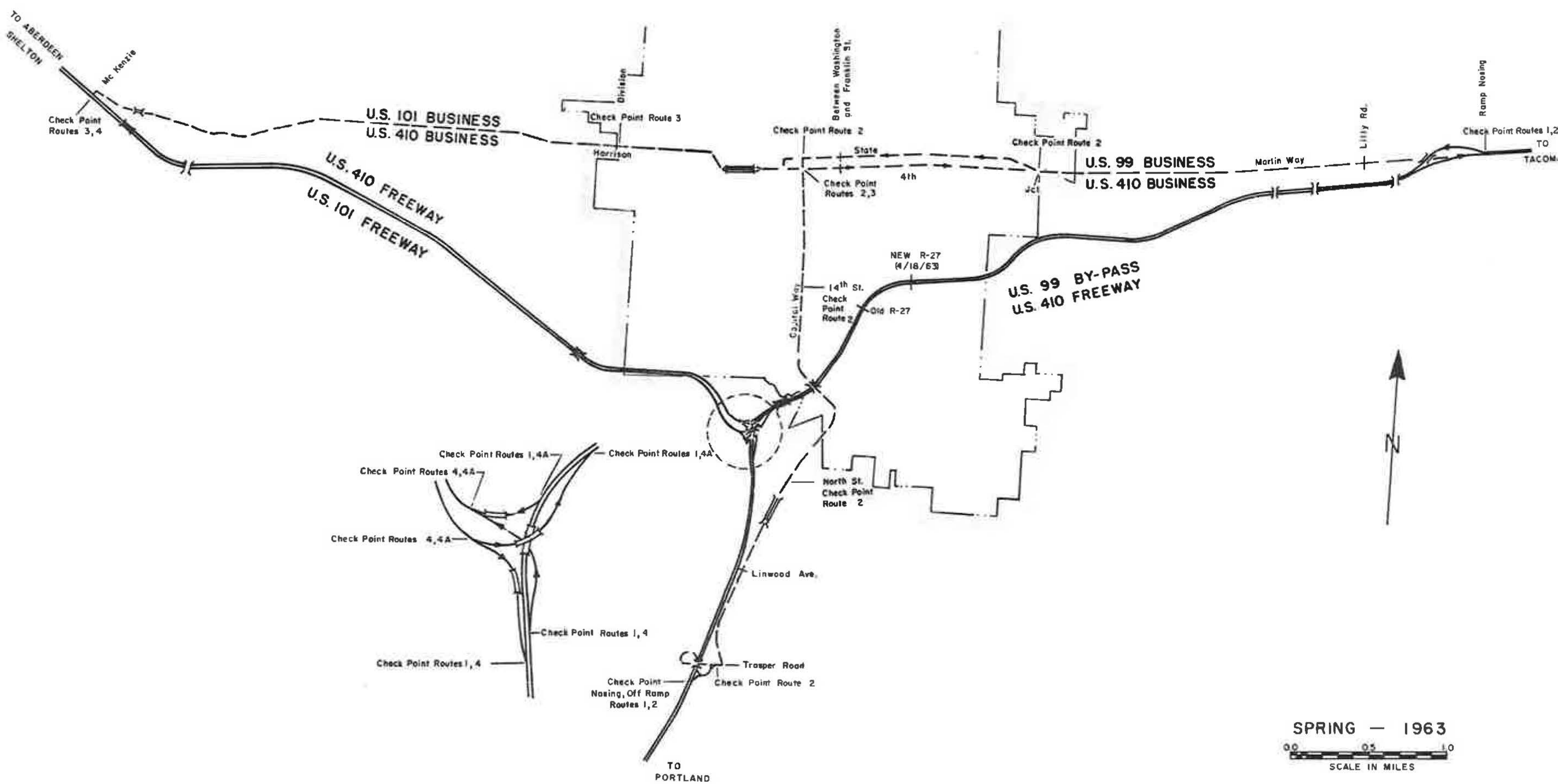


Figure 1A. Olympia study routes, Spring 1963.

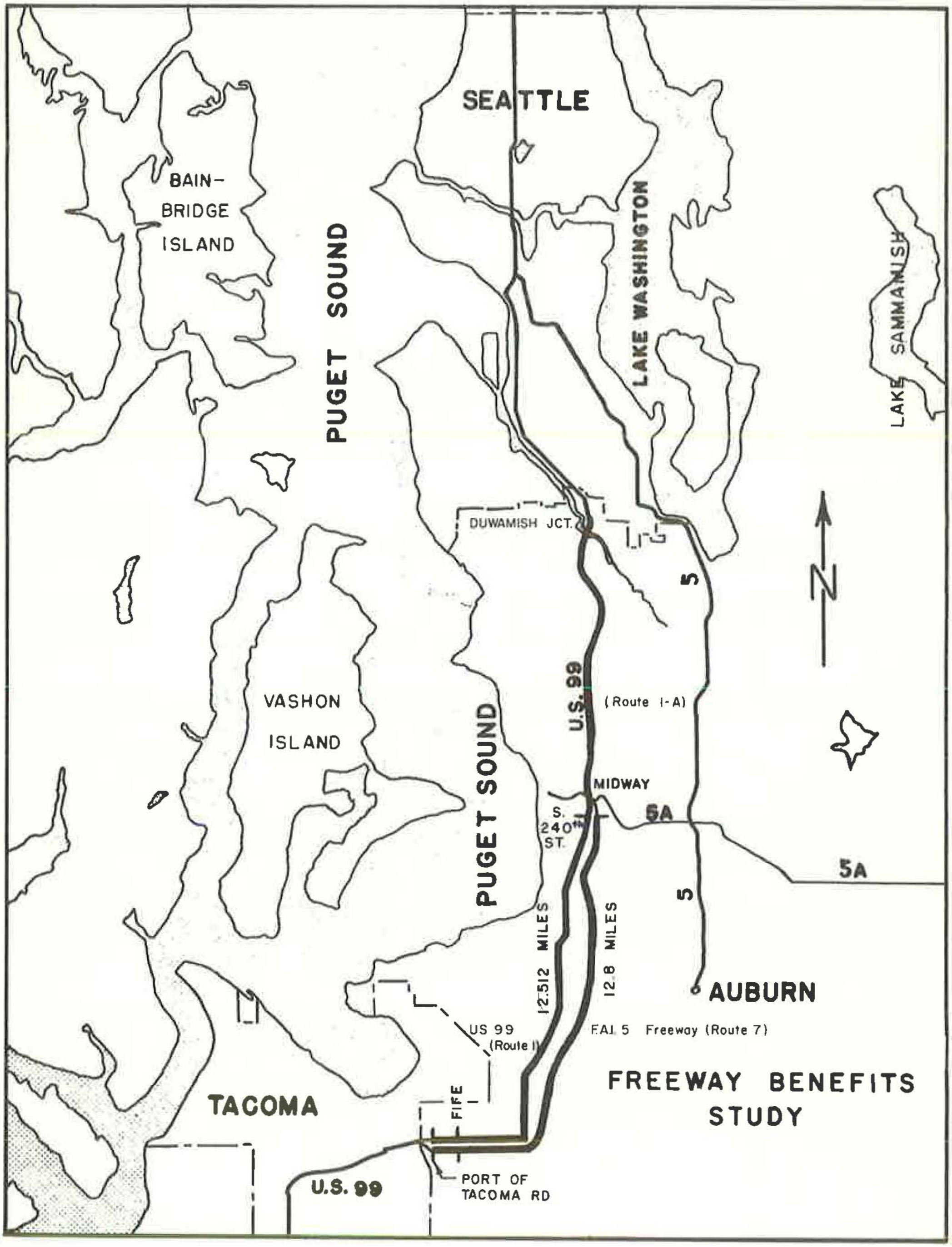


Figure 2. Vicinity map, US 99 test section.

Test Unit I- 1962 Compact 4-Door Sedan.

Test Unit 4- 1955 Diesel 3-S2

Test Unit 2- 1962 Standard V-8 4- Door Sedan.

Test Unit 5- 1962 Single Unit Truck With Dual Rear Tires.

Test Unit 3 – 1962 Half Ton 6 Cylinder Pick Up.

Test Unit 6– 1963 Standard V-8 4-Door Sedan.

Figure 3. Test vehicles.

data on the business and freeway routes. Due to the limitations in the fuel metering device used on these earlier studies the State Highway Commission requested the University research group to re-evaluate the fuel consumption and travel time of vehicles operating on the business routes vs those traveling on the freeway bypasses. These data were collected in March 1963 and is the primary source of information utilized in this report.

On the Seattle-Tacoma test site, fuel consumption, travel time and delay were observed on the old route under a wide variety of traffic volume conditions and on weekdays as well as weekends.

The unopened freeway route was utilized for calibrating the fuel consumption of the test vehicles for a range of constant speeds. This calibration is necessary for future comparison with similar vehicles to be utilized during the after portion of the study. Data were not collected using the average car method on the freeway route, because the speed of operation was at the discretion of the driver, and was not affected by traffic volume. However, a license check method was used to determine the actual travel time (overall speed) of various types of vehicles (Appendix B). This speed data can be used with the vehicle calibration curves to determine the fuel consumption. Data was again collected on the old route to evaluate the traffic service relief.

Fuel Consumption

Fuel consumption was measured by two methods in these series of tests. The fuel meter model FM 200 was utilized for measuring the fuel consumption of the passenger cars, whereas the burette board method was utilized on the trucks. Although the FM 200 meter is considered to be more accurate for instantaneous readings enroute, only one such meter was available. The burette boards were used so that data from two vehicles could be recorded simultaneously. Figure 4 shows typical installations of the FM 200 meter and the burette board and Figure 5 shows the FM 201 digital counter unit of the FM 200 fuel meter. More details of these metering devices are contained in Appendix C.

Both fuel metering devices utilized a fuel temperature gage for making a temperature correction to a standard 68 F. A calibration constant of 1529.3 counts per gal, valid for a wide range of flow rates, for converting the counts to gallons was determined in the laboratory for the FM 200 meter. The fuel consumption recorded by the burette boards was converted from milliliters to gallons.

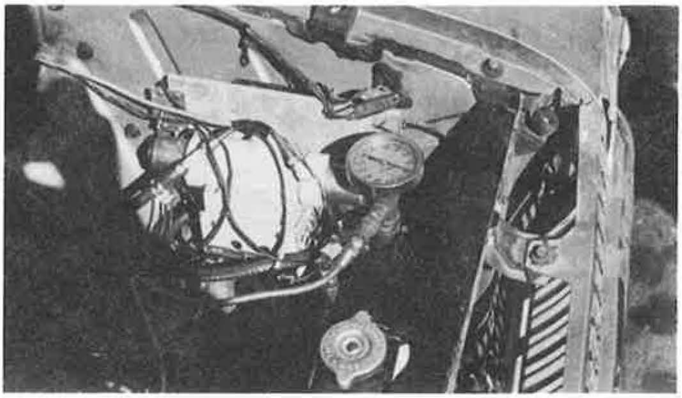
The frequency of recording the fuel consumption enroute is predicated on the volume of fuel required to traverse the section so that sufficient quantity is observed to obtain reliable accuracy. The frequency of fuel observations was generally less than the travel time observations.

Travel Time Measurements

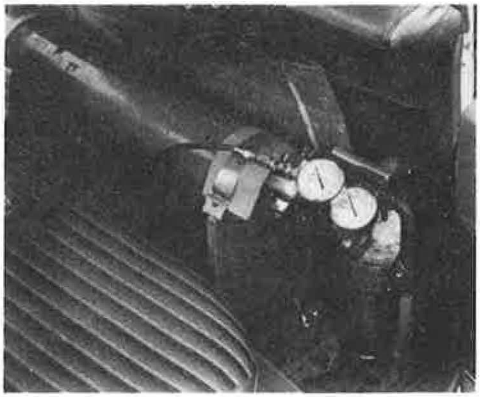
Two stop watches were used to measure the travel time along the route while utilizing the FM 200 fuel meter. These watches could be read to 0.01 min. One watch was stopped while the other was simultaneously started so that incremental times between checkpoints could be determined (Fig. 5). In using the burette board for measuring fuel consumption it was necessary to record accumulated travel time because the observer was occupied with switching the valves on the fuel meter at the checkpoint. An additional stop watch was utilized for measuring the delay time (any time the vehicle was stopped or traveling at less than 5 mph). As an overall check the driver operated a total route stop watch which was started at the beginning of a test route and stopped at the end for comparing with the accumulated observed travel time (Fig. 5, Appendix D).

Distance Measurements

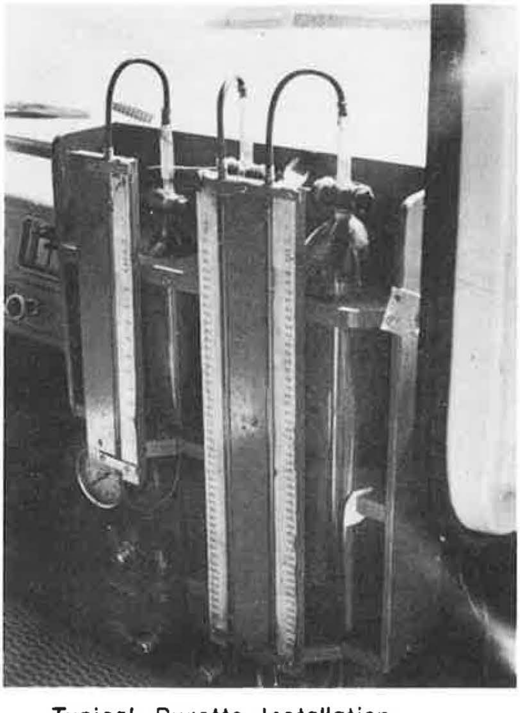
Routes on the Olympia test site were measured in 0.01 mi using a calibrated State highway department vehicle. All other routes were measured to 0.001 of a mile with a calibrated fifth wheel attachment (Fig. 5).



Typical FM 200 Fuel Meter Installation

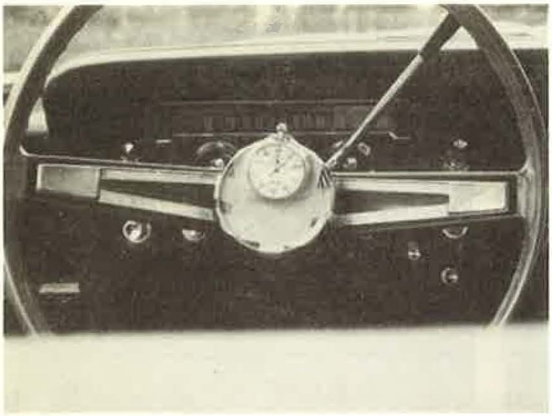


Typical Nitrogen Bottle Installation - Used With FM 200.



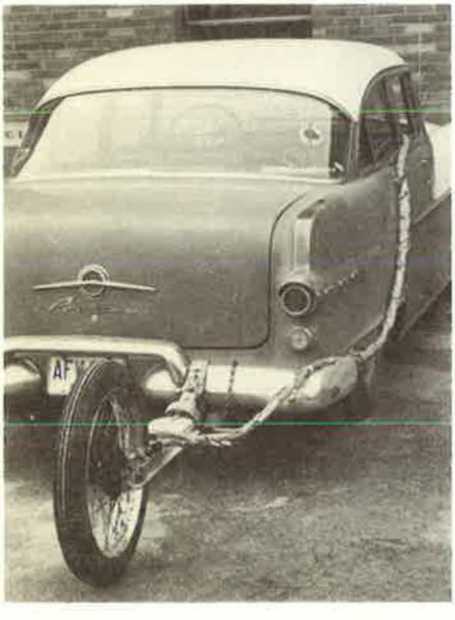
Typical Burette Installation.

Figure 4. Fuel meter installations.

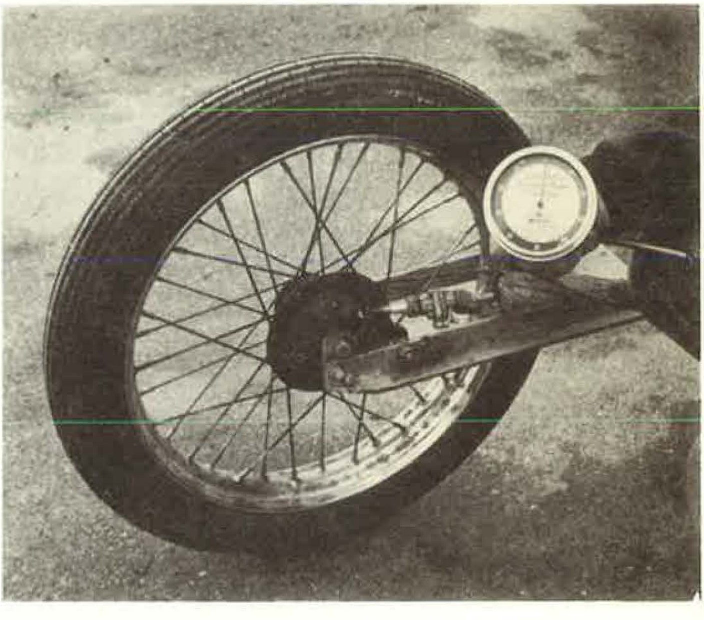


Typical Driver's Watch Installation.

Observers Watches on Data Recording Board. Note FM 201 Digital Counter.



Typical Fifth Wheel Installation.



Fifth Wheel With Survey Odometer Head.

Figure 5. Watch and fifth wheel installations.

Traffic Volume

Traffic volume data was recorded by the Highway Planning Division for both test sites. The data collected in 1958, 1959 and 1963 were not always taken at identical locations; therefore, there can be no direct comparison of the traffic volume section by section along the routes. This lack of control of the data has made it more difficult to make a complete evaluation of total benefits and, therefore, those presented in this report are for a single vehicle trip.

In general, the traffic volume was recorded on automatic recording traffic counters by 15 min intervals. The data for the Seattle-Tacoma freeway route is more complete and, therefore, is utilized in the detailed analysis of the relationship of volume to the other variables.

ANALYSIS OF THE DATA

All data collected, including fuel in counts or milliliters, travel time in minutes and traffic volume as recorded by the automatic recording counters, were processed on punch cards for electronic computer calculations. A computer program was prepared for making the necessary corrections to the measured fuel to adjust for fuel temperature and for calibration (Appendix C). The program required the distance between fuel checkpoints so that calculations could be tabulated not only for the total fuel in gallons, but also the gallons per mile and the miles per gallon. The program converted the travel time into overall speed, running speed, number of delays and delay time.

The traffic volume data for the Seattle-Tacoma test site were keypunched directly from recording counter tapes and a program was written to convert accumulated volumes to 15-min incremental volumes. The program was expanded to include the 24-hr total and also any comments indicating possible sources of error that had been noted on the counter tapes.

The two computer programs have not been interrelated due to some deficiencies in volume data resulting from counter malfunctions. Volume data were extracted manually from computer output for analysis or data plotting of fuel consumption, speed or travel time vs volume.

Route Savings in Fuel, Travel Time and Distance

The computer calculation of the fuel and travel time data was prepared for printout in such a way that each section of each route could be tabulated separately. The fuel and travel time consumed from checkpoint to checkpoint for the business routes were averaged and then weighted for traffic volume during the peak periods and off-peak periods and the accumulated fuel and travel time were calculated and plotted for the business and freeway routes bypassing Olympia. In addition, this plot also represents any savings in distance which should be considered in conjunction with time and fuel consumption savings. Figures 6 and 7 represent the directional savings derived from the US 99 bypass. The fuel and travel time savings resulting from the research were converted to economic benefits using values of time and fuel recommended by AASHO (1). These results are given in Table 1 for all three bypass routes.

An analysis of the savings and economic benefits of four of the five vehicle types traveling on the Seattle-Tacoma Freeway route is presented in Table 2. The values of fuel and travel time are considered accurate because the sample size varies in nearly the same proportion as the traffic volume and, therefore, a simple arithmetic average value can be calculated. The freeway average overall speed was obtained from Appendix B. Figures 8 and 9 were utilized for obtaining the directional fuel consumption consistent with the overall speed converted from the license check travel time. The comparison results in an economic benefit per round trip for a passenger car of -\$0.0333 for fuel and +\$0.2042 for travel time or a total benefit of +\$0.1709.

The total of travel time and fuel economic benefits range from +\$0.0578 for the pickup to +\$0.2705 for the diesel tractor and trailer for a northbound trip.

An analysis was also made of the benefits derived by those still using the old route. Data collected with the standard passenger car reveal an average fuel consumption of

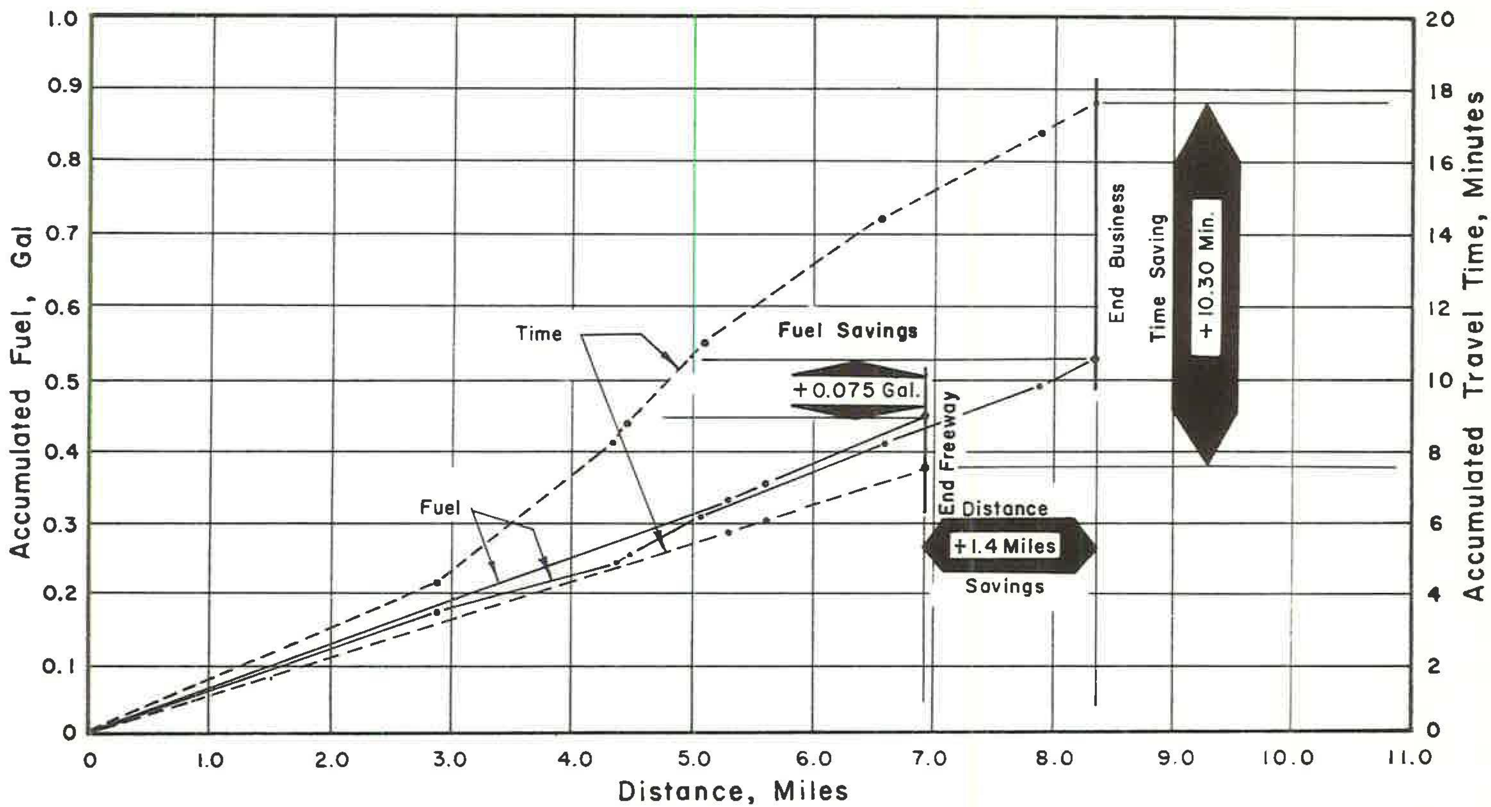


Figure 6. Accumulated fuel and travel time in 1963 by business and freeway routes for US 99 (E-S), Olympia, for standard passenger car (AWD values).

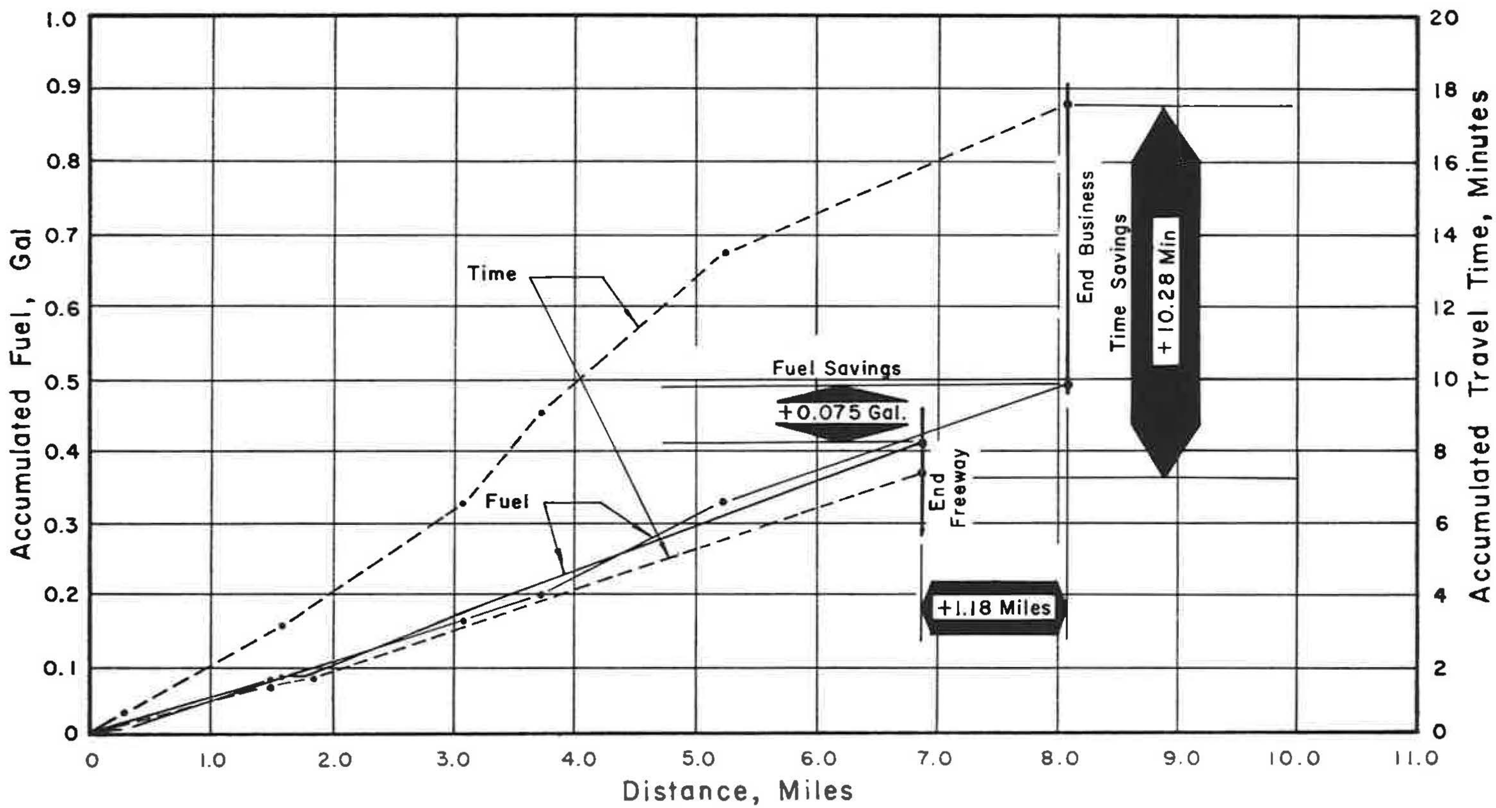


Figure 7. Accumulated fuel and travel time in 1963 by business and freeway routes for US 99 (S-E), Olympia, for standard passenger car (AWD values).

TABLE 1

ECONOMIC BENEFIT FOR A STANDARD PASSENGER CAR: OLYMPIA BYPASS ANAYLSIS DURING OLYMPIA FREEWAY OPERATION

Routes	Business Routes			Freeway			Savings			Economic Benefits[a] ($)		
	Fuel (gal)	Travel Time (min)	Distance (mi)	Fuel (gal)	Travel Time (min)	Distance (mi)	Fuel (gal)	Travel Time (min)	Distance (mi)	Fuel	Travel Time	Total
US 99, E-S	0.5172	17.7293	8.35	0.442	7.43	6.95	+0.0752	+10.2993	+1.4	+0.0241	+0.2657	+0.2898
US 99, S-E	0.4870	17.5813	8.05	0.412	7.30	6.87	0.075	10.28	+1.18	+0.0240	+0.2652	+0.2892
US 101, W-S	0.568	17.48	8.47	0.412	6.93	6.36	+0.156	+10.55	+2.11	+0.0499	+0.2722	+0.3221
US 101, S-W	0.512	18.02	8.35	0.372	7.15	6.41	+0.140	+10.87	+1.94	+0.0448	+0.2804	+0.3252
US 410, W-E	0.587	17.11	10.08	0.628	10.90	8.94	-0.041	+ 6.21	-1.14	-0.0131	+0.1602	+0.1471
US 410, E-W	0.544	16.58	8.99	0.609	11.02	10.06	-0.065	+ 5.56	-1.07	-0.0208	+0.1434	+0.1226

[a]Values based on \$0.32 per gal and \$0.0258 per min (1).

TABLE 2

ECONOMIC BENEFITS FOR ALL VEHICLES DURING FREEWAY OPERATION[a]

Vehicle Direction	Old Route Before			Freeway			Savings			Economic Benefits ($)[b]		
	Fuel (gal)	Travel Time (min)	Distance (mi)	Fuel (gal)	Travel Time (min)	Distance (mi)	Fuel (gal)	Travel Time (min)	Distance (mi)	Fuel	Travel Time	Total
Veh 2, N. B.	0.795	17.240	12.511	0.859	12.975	12.846	-0.064	4.265	-0.335	-0.0205	0.1100	0.0895
Veh 2, S. B.	0.700	16.730	12.511	0.740	13.080	12.746	-0.040	3.650	-0.235	-0.0128	0.0942	0.0814
Veh 3, N. B.	0.749	17.255	12.511	0.913	12.980	12.846	-0.164	4.275	-0.335	-0.0525	0.1103	0.0578
Veh 3, S. B.	0.647	17.119	12.511	0.813	12.950	12.746	-0.165	4.169	-0.235	-0.0531	0.1076	0.0545
Veh 4, N. B.	2.396	21.395	12.511	2.280	16.670	12.846	+0.116	4.725	-0.335	+0.0342	0.2363	0.2705
Veh 4, S. B.	1.878	20.113	12.511	1.773	15.490	12.746	+0.105	4.623	-0.235	+0.0310	0.2312	0.2622
Veh 5, N. B.	1.482	18.944	12.511	1.908	14.710	12.846	-0.426	4.234	-0.335	-0.1363	0.2117	0.0754
Veh 5, S. B.	1.224	18.392	12.511	1.418	14.735	12.746	-0.194	3.657	-0.235	-0.0621	0.1829	0.1208

[a]On weekdays only.
[b]Data based on values of time for passenger cars, \$0.0258 per min (1); for trucks \$0.050 per min (from wage scale); gas, \$0.32 per gal; diesel fuel, \$0.295 per gal.

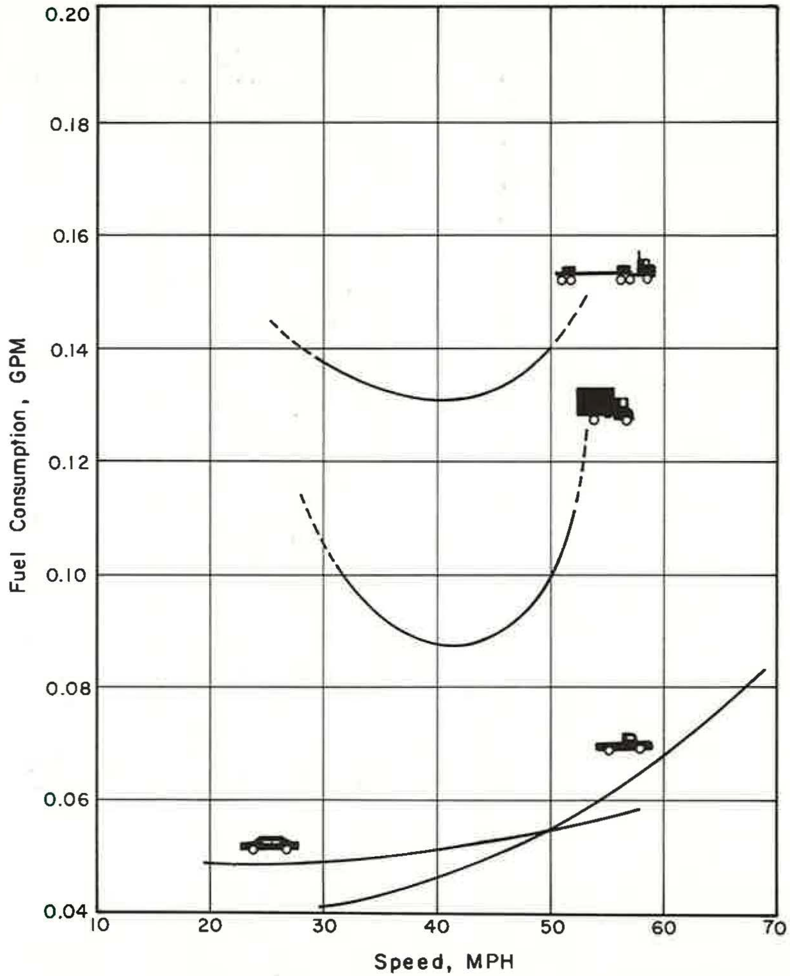


Figure 8. Fuel consumption for various constant speeds on the calibration test section for standard passenger car, pickup, single unit, and tractor and trailer (S.B.).

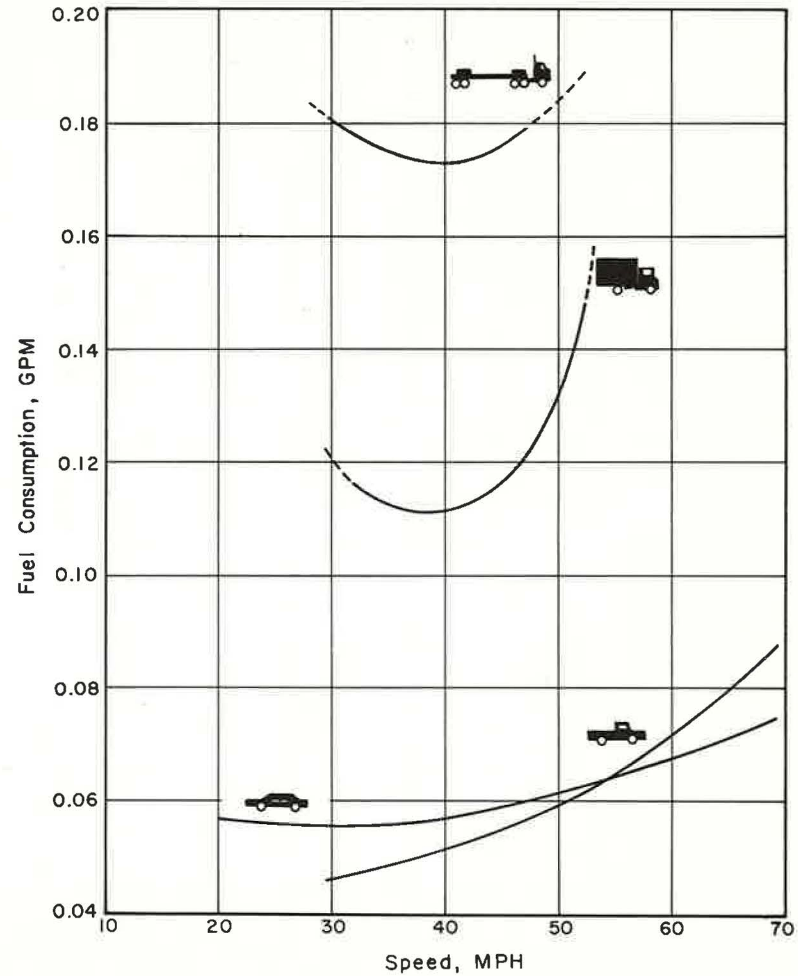


Figure 9. Fuel consumption for various constant speeds on the calibration test section for standard passenger car, pickup, single unit, and tractor and trailer (N.B.).

0.7967 gal N. B. and 0.6941 gal S. B., and an average travel time of 17.926 min N. B. and 16.780 min S. B.. Comparison of these values with those given in Table 2 (Old Route Before) shows a combined economic benefit of +$0.0418 for a round trip. The benefit in travel time would be +$0.0463 and in fuel -$0.0045.

Traffic Volume Adjustment of Observations

Considerable analysis was performed to determine the proper weighting of fuel consistent with traffic volume conditions. In selecting the sample size for making the observations for the Olympia bypass study it was necessary to secure a greater number of observations during peak hour traffic conditions than during the other hours of the day in order to obtain a sample large enough for calculating a statistically accurate average. However, these values could not be summed and a simple average taken, because the size of the sample during the peak hour would determine the amount of weighting of these observations. To establish the need to weight the observations, the following analysis was conducted for the US 99 business route through Olympia:

1. The hourly volume variation at each counter location was reviewed to determine the peak traffic volume period of 1 hr in the morning and 1 hr in the evening.
2. Fuel and travel time values during each of the peak periods and the off-peak time were averaged separately.
3. The three averaged values for fuel and travel time were multiplied by the percent of volume traveling during the corresponding period in each section along the route. The sum of these three products represents the true average daily fuel or travel time consumption for each section along the route. The total of these values represents the true volume weighted weekday average for the route (Appendix E).
4. The preceding method is used for each of the sections of the route and is plotted as a horizontal line representing the true volume weighted average value for each section and the route (Fig. 10).
5. The average of all recorded values, regardless of the time of day, was then calculated for each section and plotted as a percent of the true volume weighted average value (Fig. 10). This method can be considered an average weighted in relation to the sample size during the three time periods. The possible error in the average may be as great as +3.947 percent for one section, but the route error would be only +1.121 percent. Although the error is small the true method is recommended unless the sample size is proportionate to the traffic volume, as in the case of the Seattle-Tacoma test section. The volume weighting method has been used only for the US 99 Olympia bypass route.

An unweighted value might be considered to be the simple average of all runs made in one day (Fig. 10).

The variation of travel time and fuel values with corresponding volumes throughout the day are presented in Figure 11.

Overall Speed as Level of Service

Overall speed has been utilized in the past, and is verified in this report, as a reliable method of portraying a level of service along a route. The overall speed is calculated from the travel time but generally travel time is not utilized as an indicator of the point-to-point level of service. By utilizing overall speed, all unequal length sections along a route are standardized for comparison. The overall speeds for the US 99 Olympia bypass and business routes for 1963 are shown in Figures 12 and 13. In addition, the level of service by overall speed is also indicated in Figures 14 and 15 for the US 99 business route in 1958 before, and 1959 immediately after on both the business and freeway routes, utilizing different checkpoints than the 1963 study.

Delay as Level of Service

In utilizing overall speed as a level of service it is a misinterpretation to consider that a drop in the overall speed is directly associated with a reduction in level of ser-

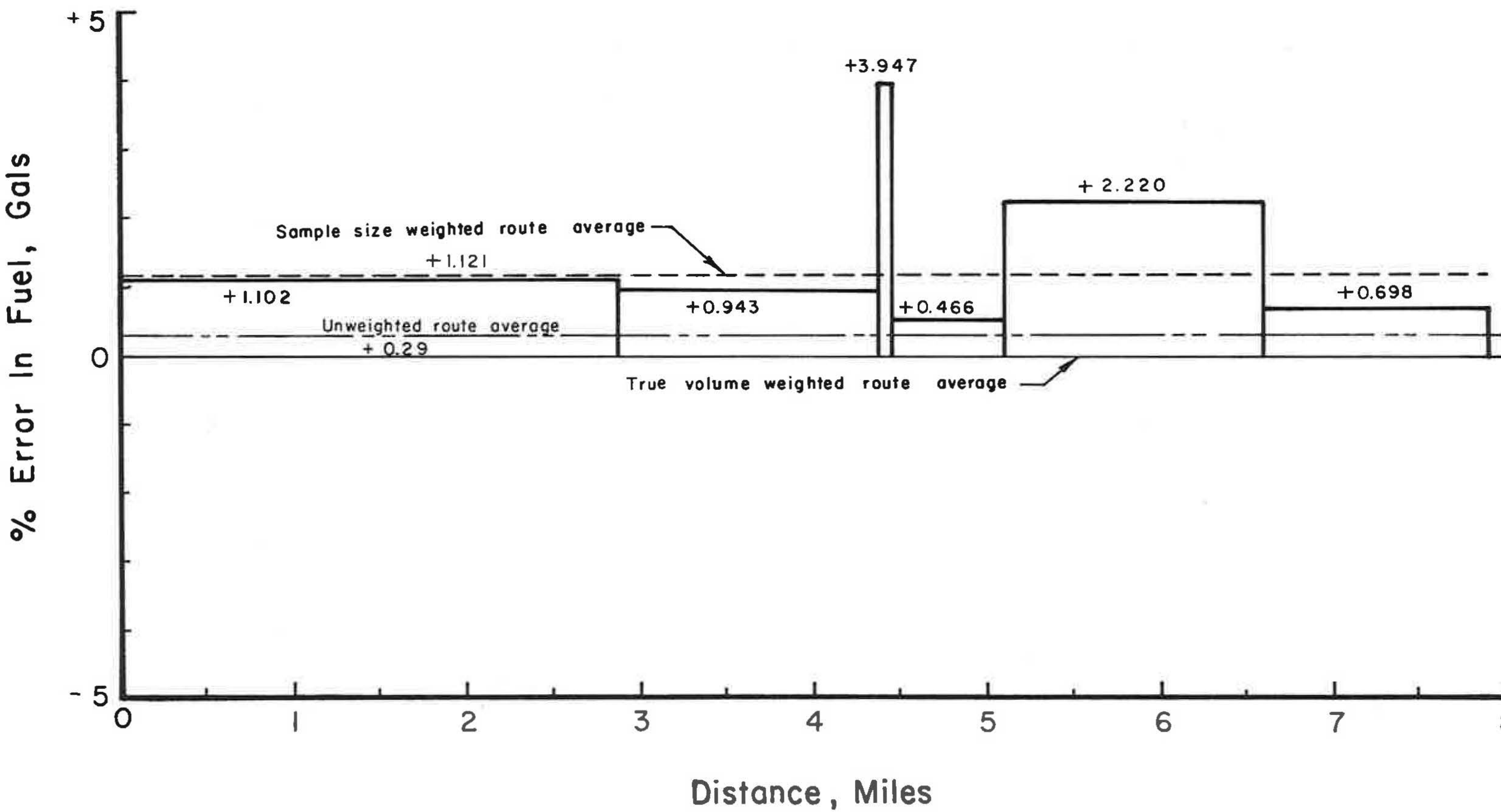


Figure 10. Possible error in percent using sample size weighted average fuel values vs volume weighted average for US 99 business route, Olympia (E-S).

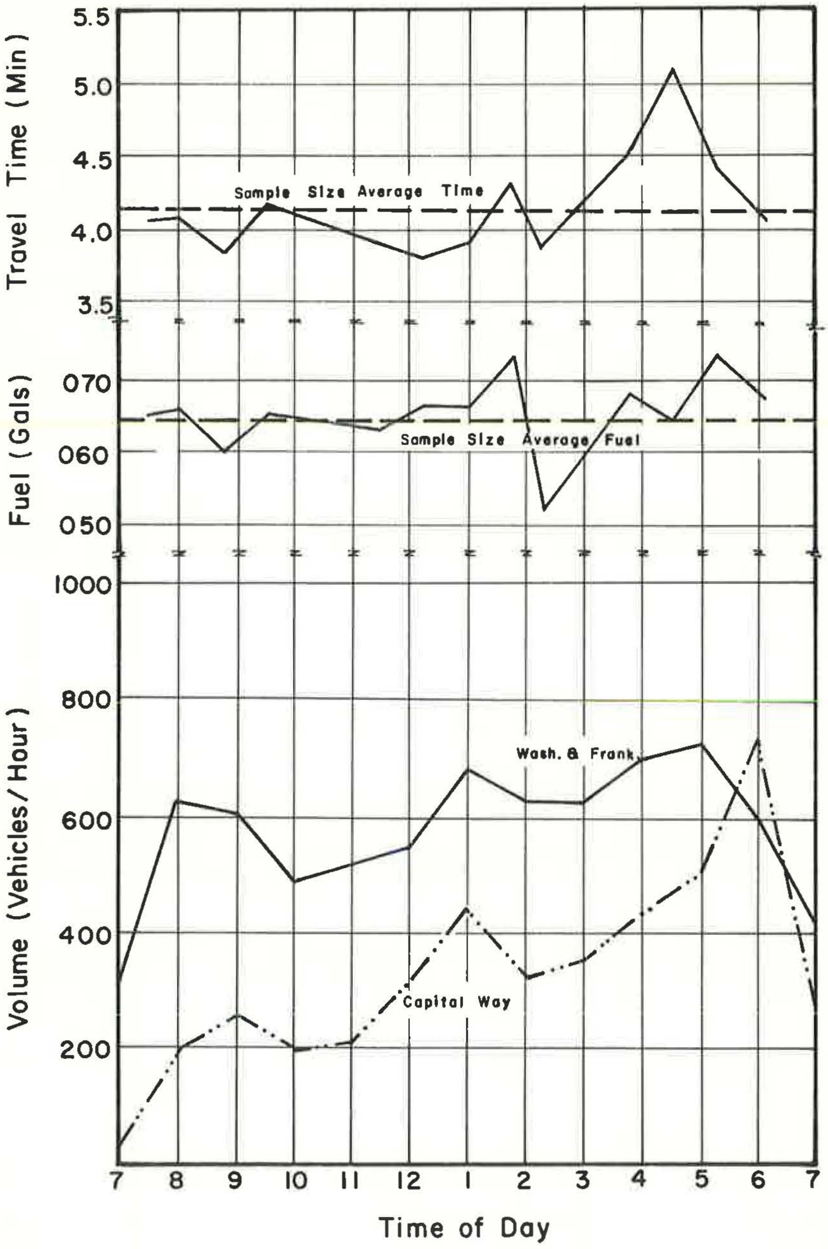


Figure 11. Route fuel consumption and traffic volume by time of day compared to daily average for US 99 business route, Olympia (E-S).

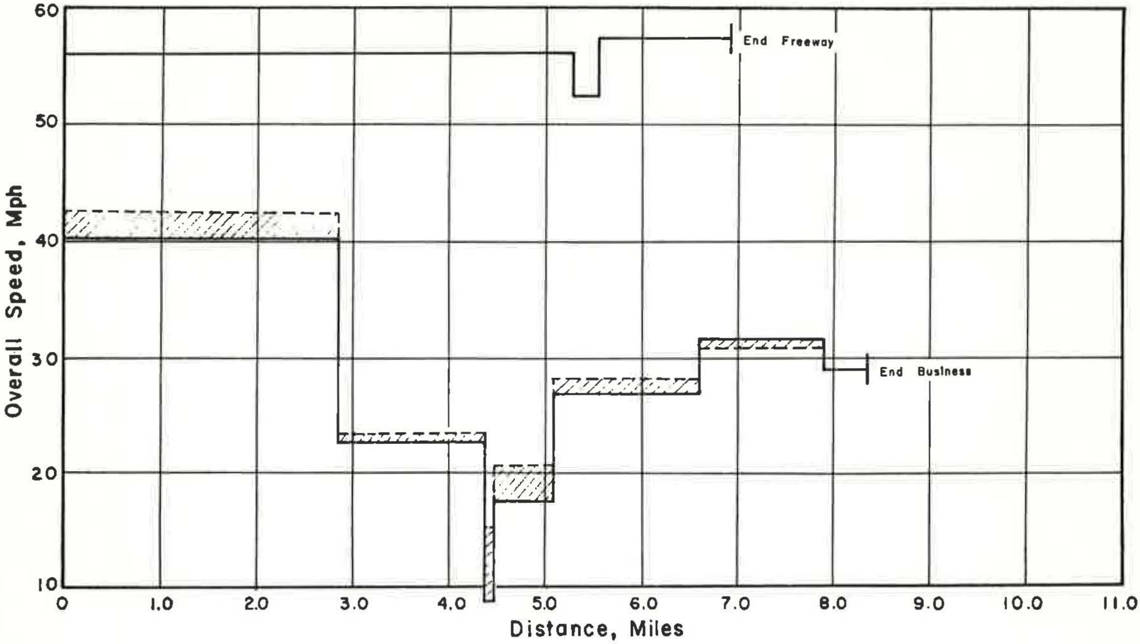


Figure 12. Overall speed and speed differential as level of service in 1963 by business and freeway routes for US 99, Olympia (E-S).

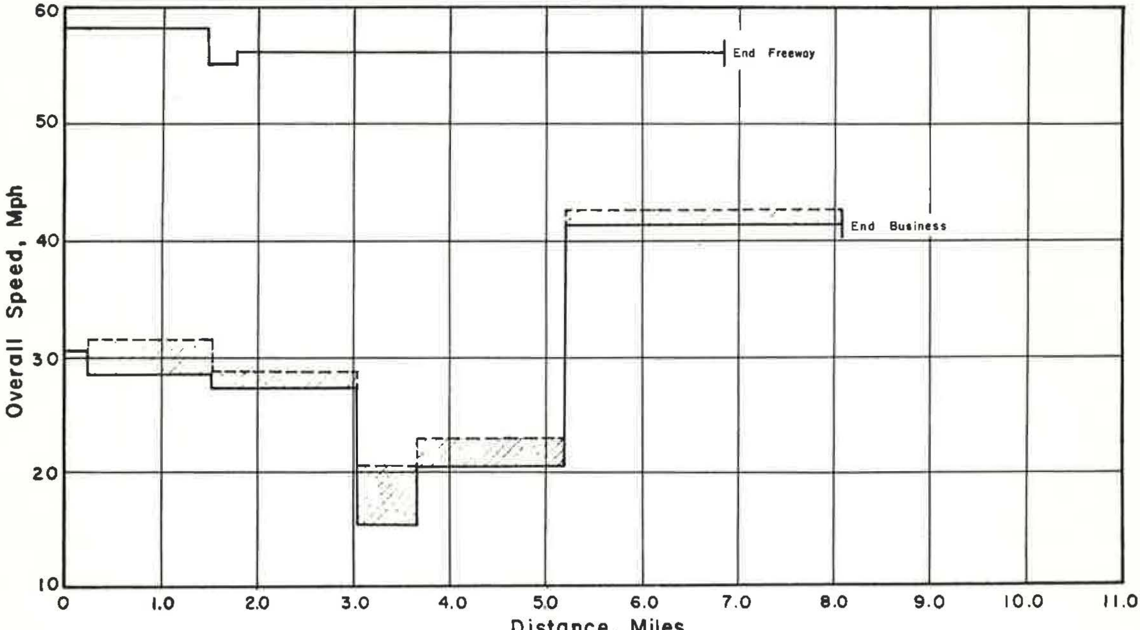


Figure 13. Overall speed and speed differential as level of service in 1963 by business and freeway routes for US 99, Olympia (S-E).

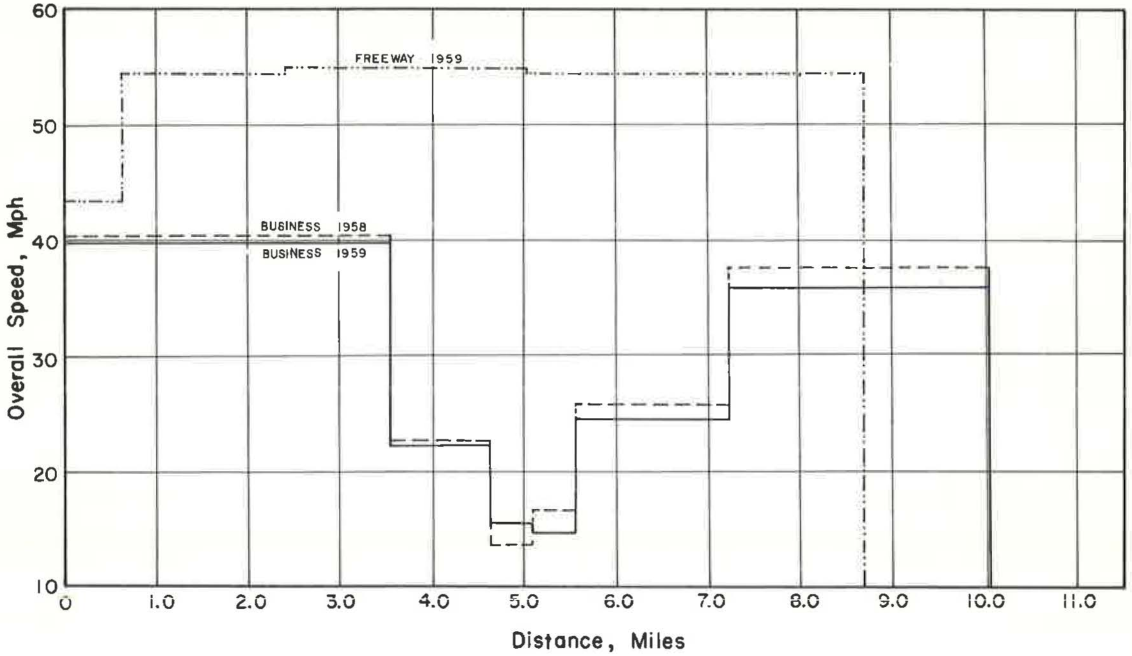


Figure 14. Overall speed as a level of service in 1958 (before) and 1959 (after) on business and freeway routes for US 99, Olympia (E-S).

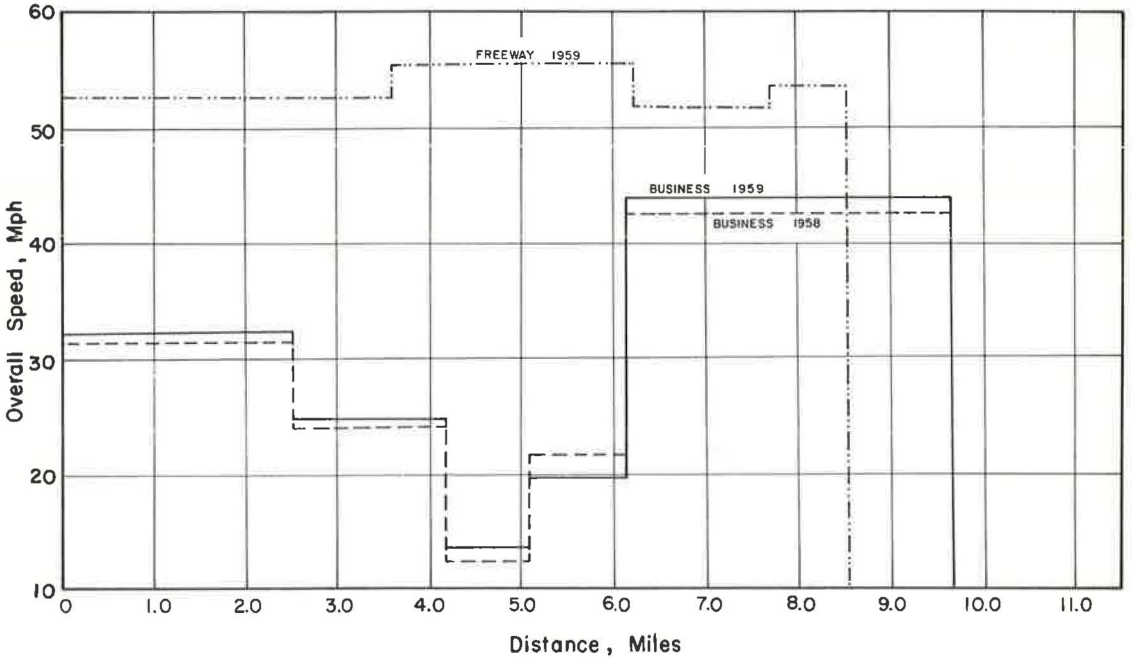


Figure 15. Overall speed as a level of service in 1958 (before) and 1959 (after) on business and freeway routes for US 99, Olympia (S-E).

vice. In some instances the reduction in the overall speed is still an acceptable level of service, particularly when traversing from a suburban area to a downtown area. Of more significance in the downtown area is the amount of delay or the difference between the overall and the running speeds.

The magnitude of the spread between the overall and the running speeds indicates to some individuals the potential of a deficiency in level of service. However, to others it is more desirable to indicate the location of delay, the number of delays and the length of time of each. Figures 12 and 13 show a difference between operating and running speeds which could be the result of one long delay or numerous short delays. Both are a source of irritation to the driving public; however, the number of stops causes more reaction from the driver and at the same time creates additional wear on the vehicle which is not immediately apparent to the driver. Figure 16 shows the number of delays, total delay and average delay for a typical test run on the Olympia US 99 business route in 1963.

Traffic Volume Relief

Many studies have utilized traffic volume counts for an evaluation of one route vs another or for before and after studies. It is logical to assume that the level of service could be raised if traffic volume is decreased. Until such time as a definite relationship can be established between traffic volume and road user costs, traffic volume should only be considered as an indicator of relief. For the Olympia bypass study, directional daily volume variations are indicated for 1958, 1959 and 1963 at comparable locations (Figs. 17 through 20). The hourly volume variation for these same years on the business route is shown in Figures 21 through 24 and the daily volume relief along the US 99 business route is indicated in Figures 25 and 26. These figures show that there has been a definite reduction in the level of traffic volume on the Olympia US 99 business route; this can be attributed to the opening of the freeway bypass. It can also be seen that during the peak hour in 1958 the level of traffic volume was considerably greater than that observed in 1963.

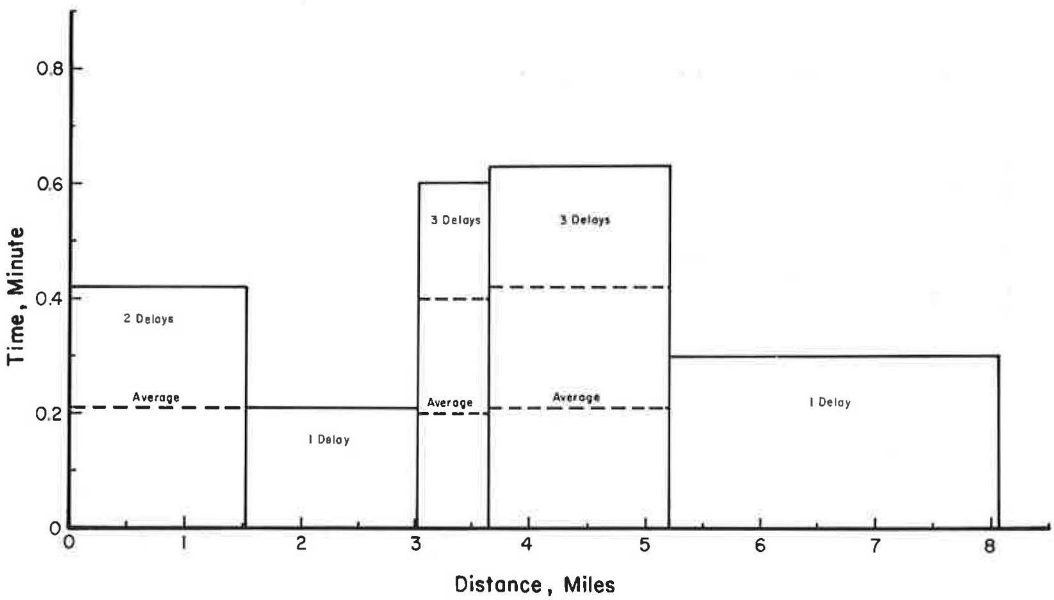


Figure 16. Example of total delay, number of delays and average delay for standard passenger car on Olympia US 99 business route (S-E) in 1963.

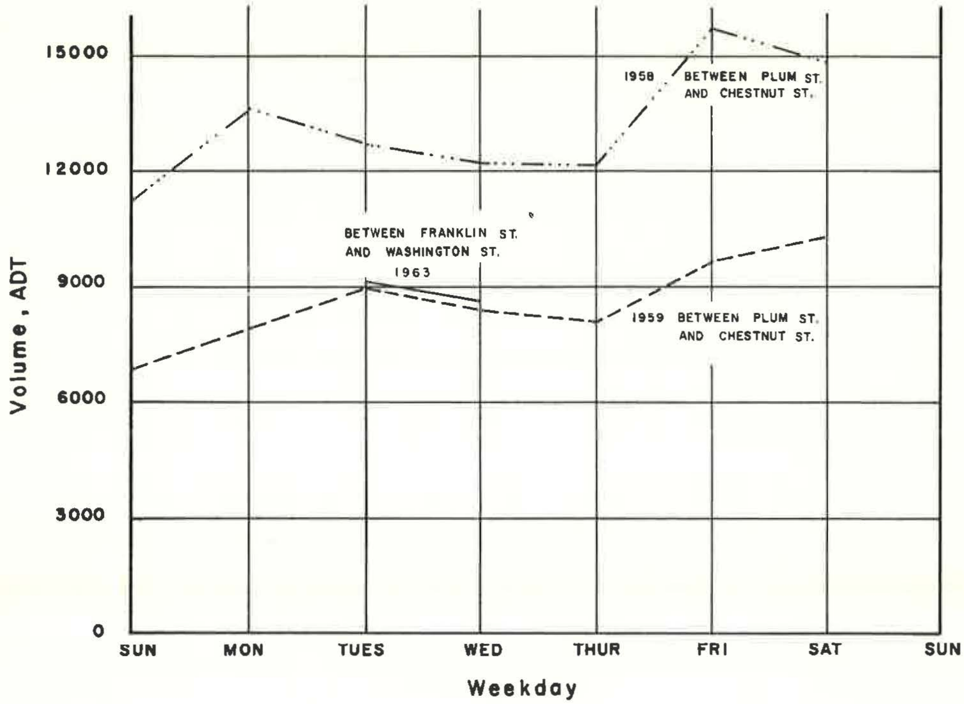


Figure 17. Daily volume variation on US 99 business (E-S) and US 410 business (E-W) for 1958 (before), 1959 and 1963 (after) on State Ave. E. in Olympia.

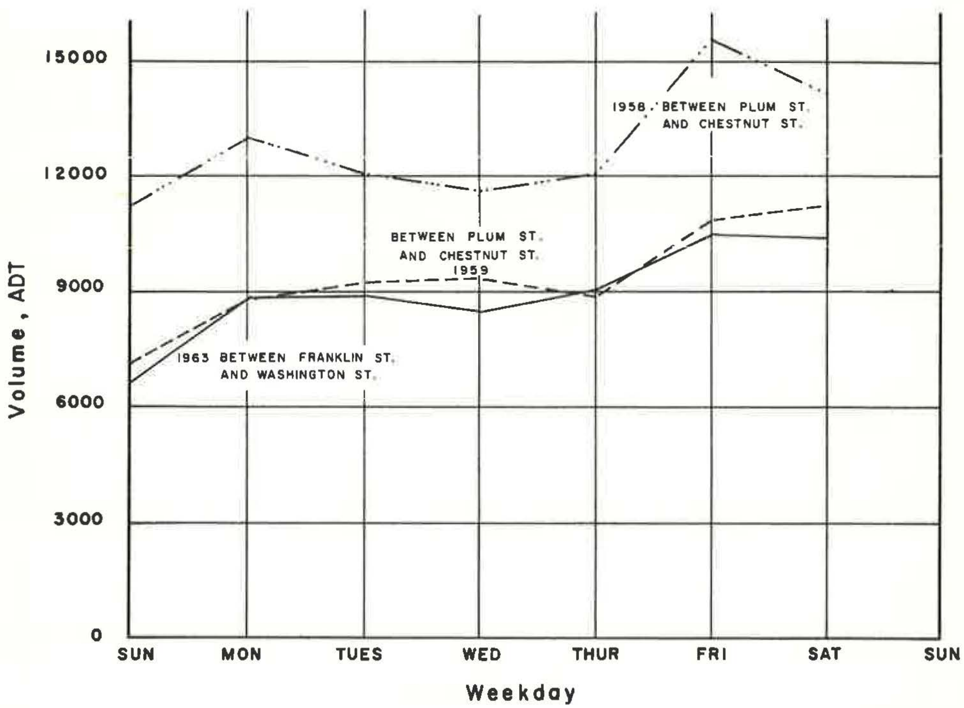


Figure 18. Daily volume variation on US 99 business (S-E) and US 410 (W-E) for 1958 (before), 1959 and 1963 (after) on the 4th Ave. E. in Olympia.

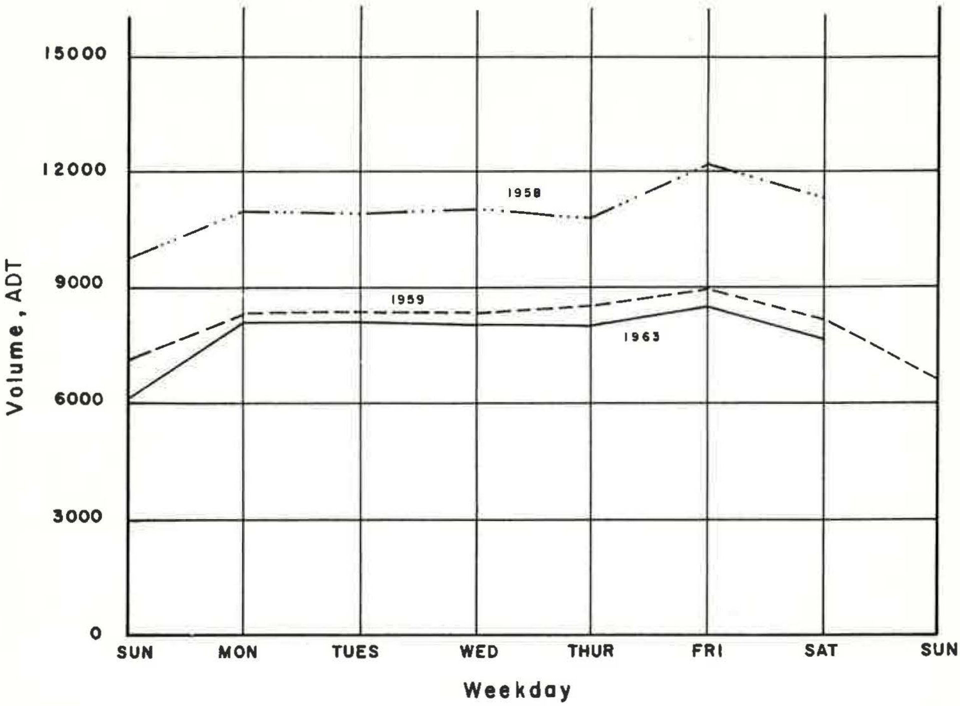


Figure 19. Daily volume variation on US 99 business (E-S) and US 101 business (W-S) for 1958 (before), 1959 and 1963 (after) on Capital Way at freeway overpass in Olympia.

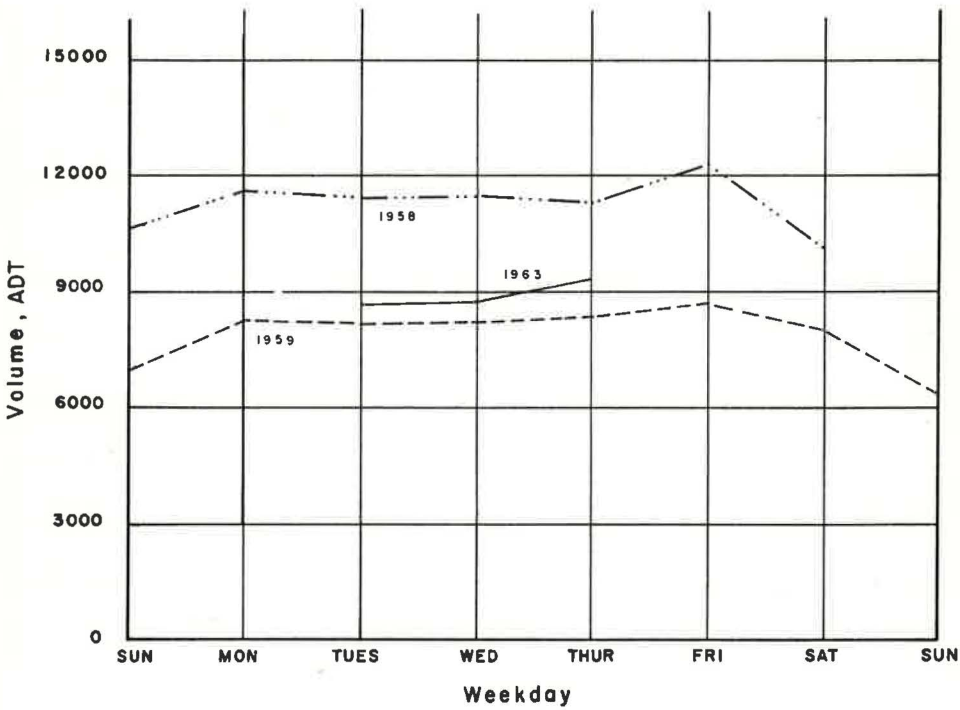


Figure 20. Daily volume variation on US 99 business (S-E) and US 101 business (S-W) for 1958 (before), 1959 and 1963 (after) on Capital Way at freeway overpass in Olympia.

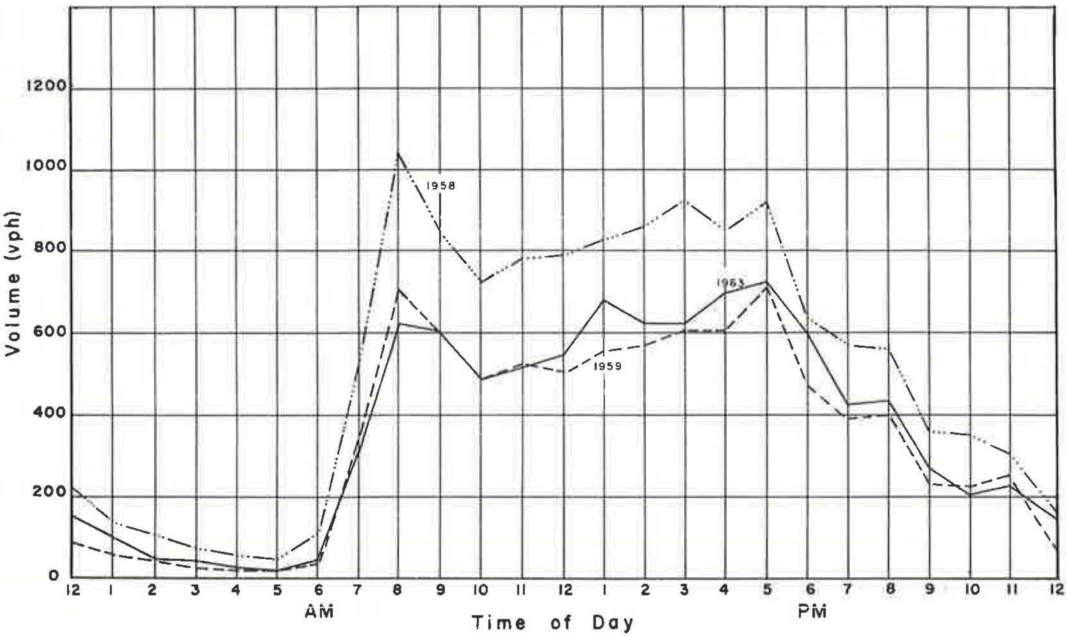


Figure 21. Hourly volume variation on US 99 business for 1958 (before), 1959 (after) on State Ave. between Plum and Chestnut Sts. and for 1963 (after) on State Ave. between Washington and Franklin Sts. in Olympia.

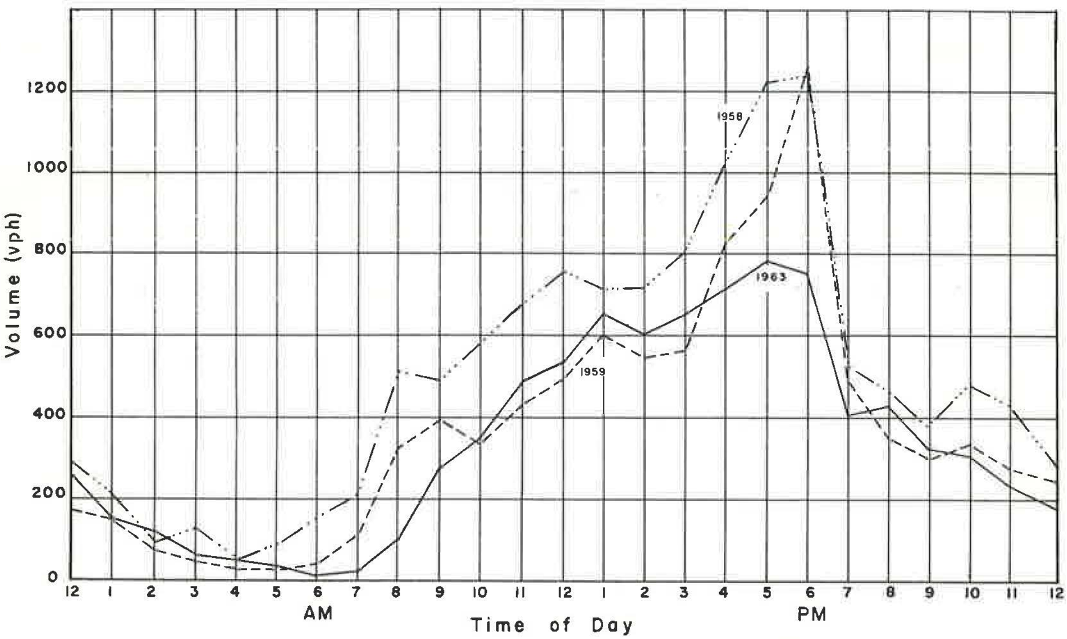


Figure 22. Hourly volume variation on US 99 business (S-E) for 1958 (before), 1959 and 1963 (after) on 4th Ave. between Washington and Franklin Sts. in Olympia.

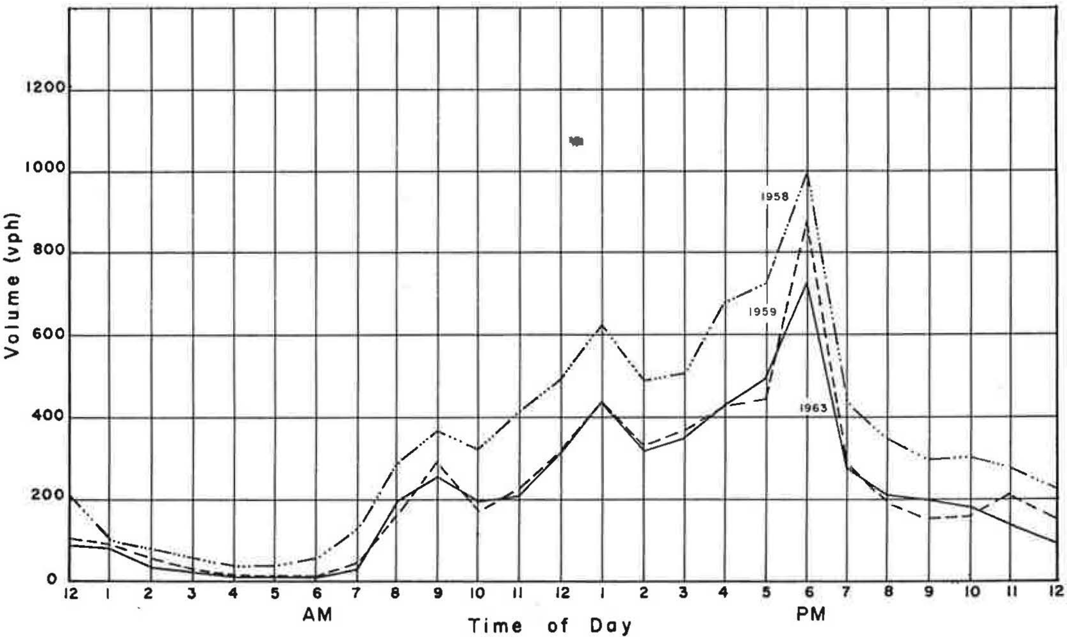


Figure 23. Hourly volume variation on US 99 business (E-S) and US 101 business (W-S) for 1958 (before), 1959 and 1963 (after) on Capital Way in Olympia.

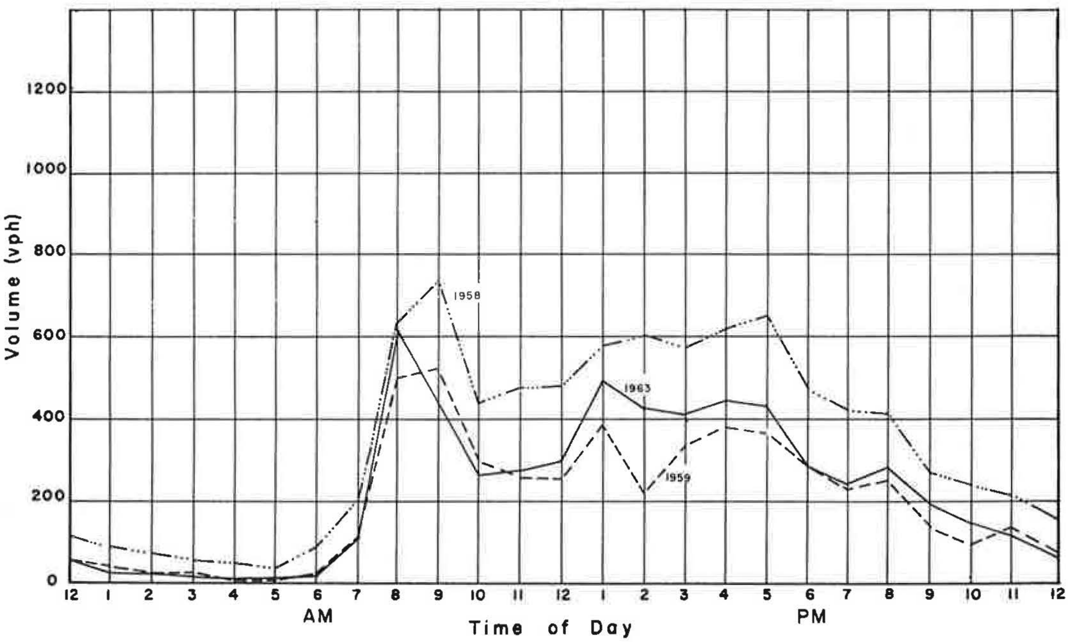


Figure 24. Hourly volume variation on US 99 business (S-E) and US 101 (S-W) for 1958 (before), 1959 and 1963 (after) on Capital Way in Olympia.

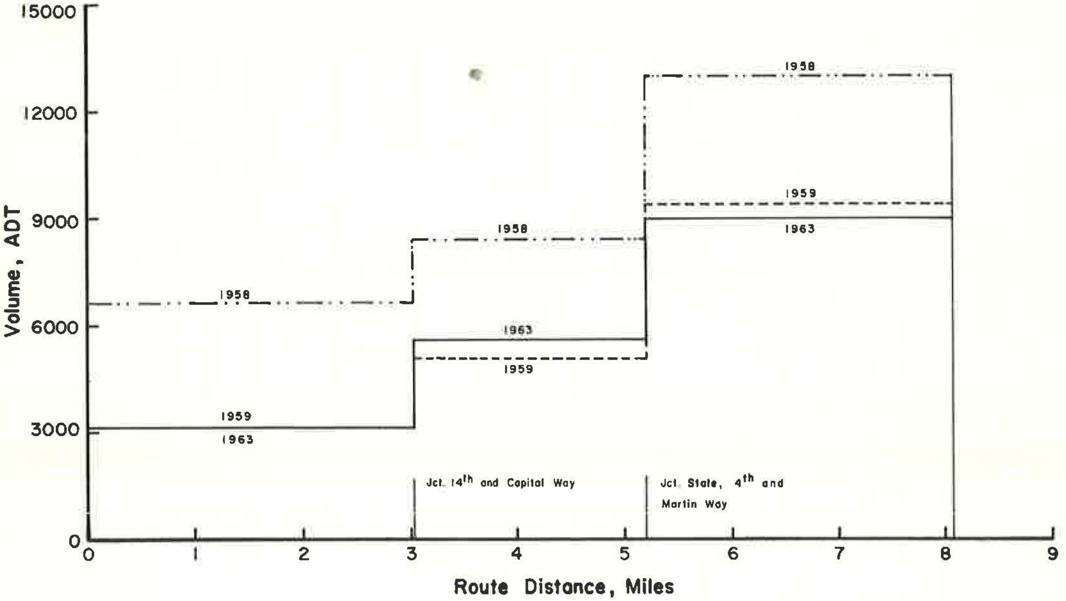


Figure 25. Relief in traffic volume along US 99 business route (S-E) in Olympia.

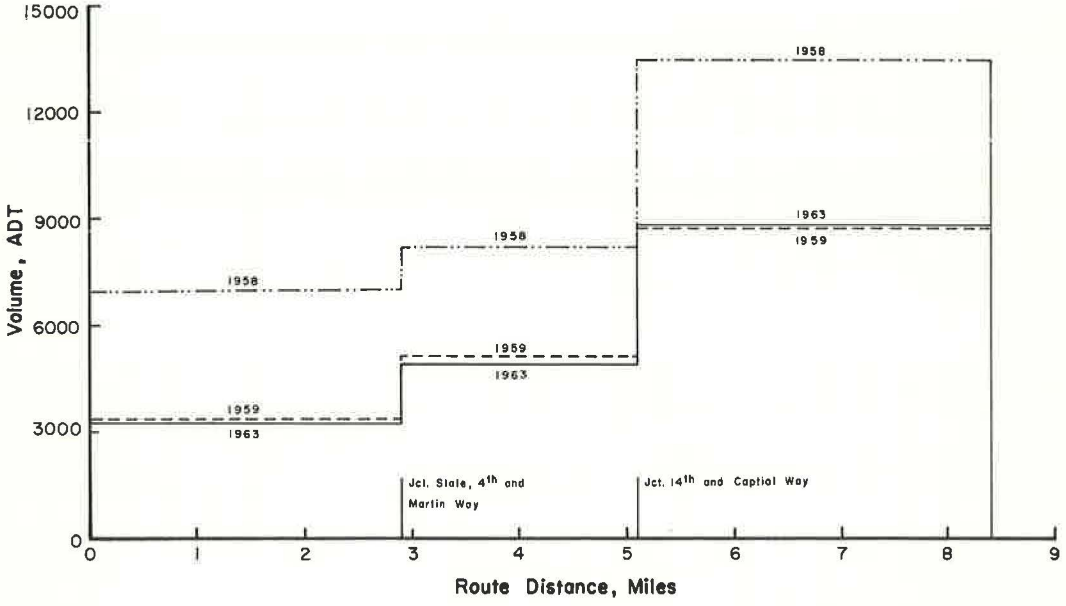


Figure 26. Relief in traffic volume along US 99 business route (E-S) in Olympia.

Appendix B

OVERALL SPEED SUMMARY BY VEHICLE TYPE ON THE SEATTLE-TACOMA FREEWAY ROUTE[a]

Vehicle Type	Travel Time[b]				Spot Speed[c]			
	No. Veh		Speed (mph)		No. Veh		Speed (mph)	
	S. B.	N. B.	S. B.	N. B.	S. B.	N. B.	S. B.	N. B.
Compacts	31	23	57.805	60.499	37	-	54.892	-
Std. pass. car	102	83	59.654	60.122	174	200	56.425	55.460
Pickups and panels	17	20	56.733	55.056	8	25	56.750	56.240
SU	4	19	53.071	51.868	14	14	52.286	45.256
Truck and trailer	-	-	-	-	21	15	48.381	46.267
Diesel	16	9	47.544	46.209	-	-	-	-
Gas	4	14	48.100	47.607	-	-	-	-

[a]Weather conditions on Jan. 16, 1963, PM, overcast; May 14, 1963, AM, clear; and June 12, 1963, AM, overcast and PM, broken clouds.

[b]Travel time observations were made on June 12, 1963: S.B. observations between 10:30 AM and 12:00 noon, and N.B. observations between 12:45 and 2:15 PM by the University of Washington using the license plate method.

[c]Spot speed observations were made by the State District Traffic Engineer; S.B. observations on May 14, 1963 between 10:10 and 11:10 AM, and N.B. observations on Jan. 16, 1963 between 1:15 and 2:10 PM. A radar speed meter was utilized to obtain the data.

Appendix A

TEST VEHICLE DESCRIPTIVE DATA

Test Unit No.	Axle Classif.	Year of Manuf.	Body Type	Wheel Base (in.)	Overall Length (in.)	Fuel Type	Cylinders		Mfrs. Net Hp	Rpm	Rear Axle Gear Ratio	Transmission								Tire Size	Gross Veh. Wt.[a] (lb)
							No.	Displacement (cu in.)				Type	Ratio				Range				
													1st	2nd	3rd	4th	Low	Hi	Reverse		
1	2	1962	4-door	109.5	181.1	gasoline	6	170	101	4,400	3.50:1	Auto	-	-	-	-	1.75:1	1:1	1.5:1	6.00 × 13	3,100
2	2	1962	4-door	119	209.6	gasoline	V-8	283	170	4,200	3.36:1	Auto	-	-	-	-	4.55:1-1.32:1	4.55:1-1:1	4.55:1-1.32:1	7.50 × 14	4,180
3	2	1962	pickup	127	206	gasoline	6	235	110	3,600	3.90:1	Std	2.94	1.68	1.00	-	-	-	-	6.70 × 15	4,500
4	3-S2	1955	tractor and flatbed	514	544	diesel	6	743	275	2,100	-[b]	Std	-[b]	-[b]	-[b]	-[b]	-	-	-	10.0 × 20	48,175
5	SU2D	1962	cab over 18-ft-van	153	24.75	gasoline	V-8	327	160	4,000	-[b]	Std	7.06	3.58	1.71	1.00	-	-	-	10.0 × 22	13,000 part 25,360 full
6[b]	2	1963	4-door hard top	119	210.4	gasoline	V-8	283	195	4,800	3.36:1	Auto	-	-	-	-	3.82:1-1.82:1	3.82:1-1:1	3.82:1-1.82:1	7.50 × 14	3,740

[a]Includes wt. of driver, observer and test equipment with units 3, 4 and 5 loaded with additional wt. to obtain avg. gross veh. wt. recorded by Highway Planning Division in Seattle-Tacoma test site.
[b]Not available.

Figure 27 illustrates preliminary results of fuel consumption in gallons per mile with corresponding overall speed as observed on the old route of US 99 between Seattle and Tacoma. Both before and after fuel rates are shown, as well as the rate at constant speed on the freeway section, for a standard passenger car. A wider variation may be expected in the fuel rates for the old section with traffic volume and signals interfering with the traffic flow along the route. A manual fit of curves to the data shows curves on the old section similar to the freeway data at constant speed. This figure also shows the range of speed observed with the after values within a narrower speed range.

Traffic Volume vs Fuel and Overall Speed

Ideally the economic variables of fuel and travel time (overall speed) should be related to traffic volume. A more comprehensive analysis might develop a family of curves for highway types. A sample of this relationship for a standard V8 passenger car operating on the old US 99 route between Tacoma and Midway, a 4-lane undivided suburban major arterial, is represented in Figure 28. From such a relationship it would be relatively simple to calculate the total fuel and travel time on an existing facility if the traffic volume is known. Additional research is necessary to determine to what degree traffic volume data is required on existing facilities and also planned facilities. If 15-min or hourly volumes are necessary, the problem of predicting fuel and travel time on planned facilities is more complex. The data presented in Figure 28 are actually the best data collected in this study. Greater control of the accuracy of volume counting is considered necessary to develop more reliable curve data.

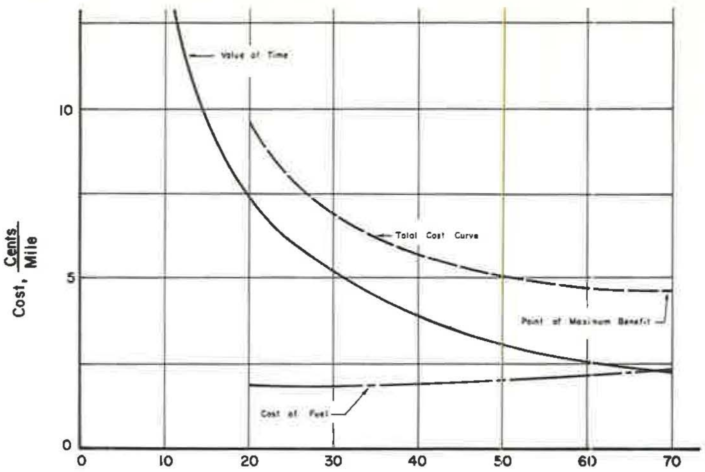
Cost of Operation at Various Speeds

Accident costs are a major portion of the total cost of operation; however, it was not within the scope of this research to collect and analyze such data. Only total time and fuel costs (1) are presented in this report. The US 99 freeway between Tacoma and Midway is utilized in this example for test vehicles ranging from a standard passenger car to a tractor and trailer unit. Time cost is an asymptotic function of speed and fuel costs are obtained from Figure 9. The total cost curve is the addition of the fuel and time curves. Figure 29 shows an optimum speed (maximum benefit) of 70 mph for the standard passenger car and 55 mph for the tractor and trailer, but with a much narrower range of operation.

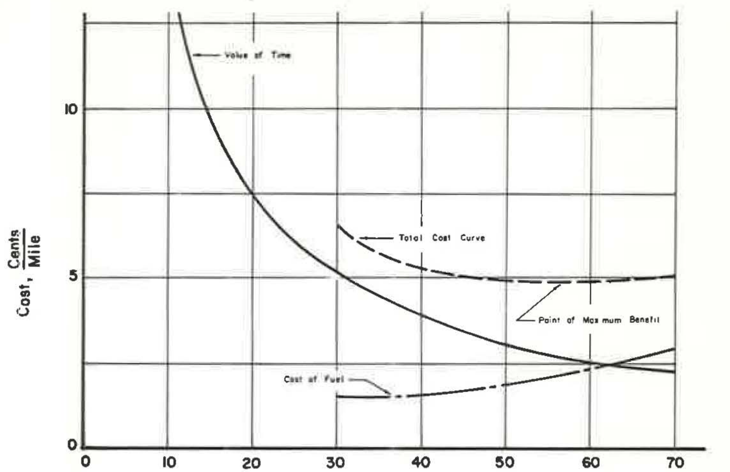
Additional research on other types of facilities may provide a realistic means of evaluating the proper speed level.

REFERENCES

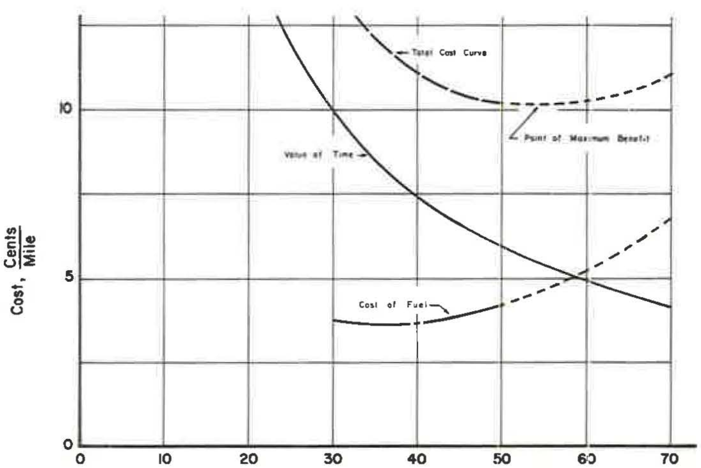
1. "Road User Benefit Analysis for Highway Improvements." Amer. Assoc. of State Highway Officials, 152 pp. rev. (1960).
2. Kieling, W. C., "Fuel Meter Model FM 200." HRB Bull. 334, pp. 47-53 (1962).
3. Sawhill, R. B., and Firey, J. C., "Motor Transport Fuel Consumption Rates and Travel Time." HRB Bull. 276, pp. 35-91 (1960).



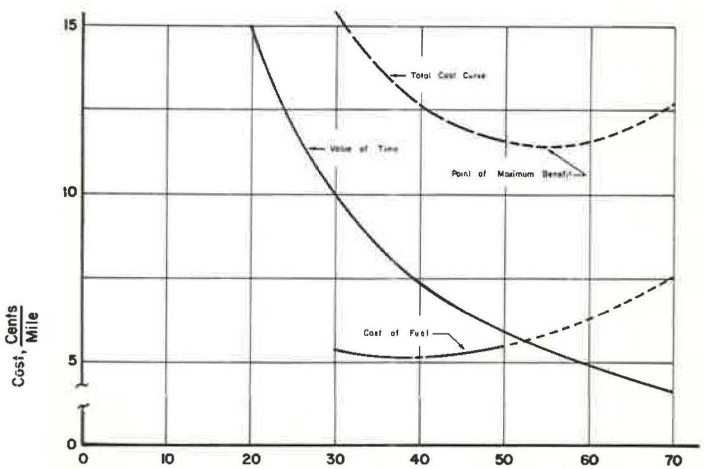
Relationship of Value of Time to Cost of Fuel, for a standard 4-door V-8 sedan.



Relationship of Value of Time to Cost of Fuel, for a half ton pick-up.



Relationship of Value of Time to Cost of Fuel, for a single unit truck.



Relationship of Value of Time to Cost of Fuel, for a diesel tractor and trailer.

Figure 29. Example of optimum speed for various types of vehicles on typical freeway.

ANALYSIS OF SPEED, FUEL, TRAVEL TIME, VOLUME RELATIONSHIPS AND COSTS

This research study has substantiated previous results and has presented new relationships as outlined in the following:

Fuel in Gallons per Mile vs Speed

From previous studies by the research group it has been definitely determined that there can be consistent relationships developed between fuel consumption and constant speed of operation. For the Seattle-Tacoma freeway study, the new section was utilized for running constant speeds and observing the corresponding fuel consumption (Figs. 8, 9). The section was basically an uphill route in a northbound direction and downhill in the southbound direction. The total amount of rise is 606.84 ft northbound and 318.68 ft southbound. It should be kept in mind that this overall speed is constant and the vehicles are operated on an open section of freeway where traffic volume and control devices are not a factor. The general shape of these curves indicates that as the vehicle size increases, the curve becomes more U-shaped, with the optimum fuel consumption at a speed of about 40 mph for the larger truck. However, the passenger cars operate most efficiently at low speeds. These characteristics should be recognized in evaluating the difference in fuel consumption on a freeway at speeds of 60 to 70 mph vs speeds of 20 to 25 mph on a business route. In such cases the savings in time may be considerable, whereas the savings in fuel would be a negative quantity, as previously illustrated for the Seattle-Tacoma study.

Fuel in Gallons per Mile vs Observed Overall Speed

Vehicles on highway facilities, except freeways or highways without impeding traffic control devices and with low traffic volume do not all travel at uniform rates of speed.

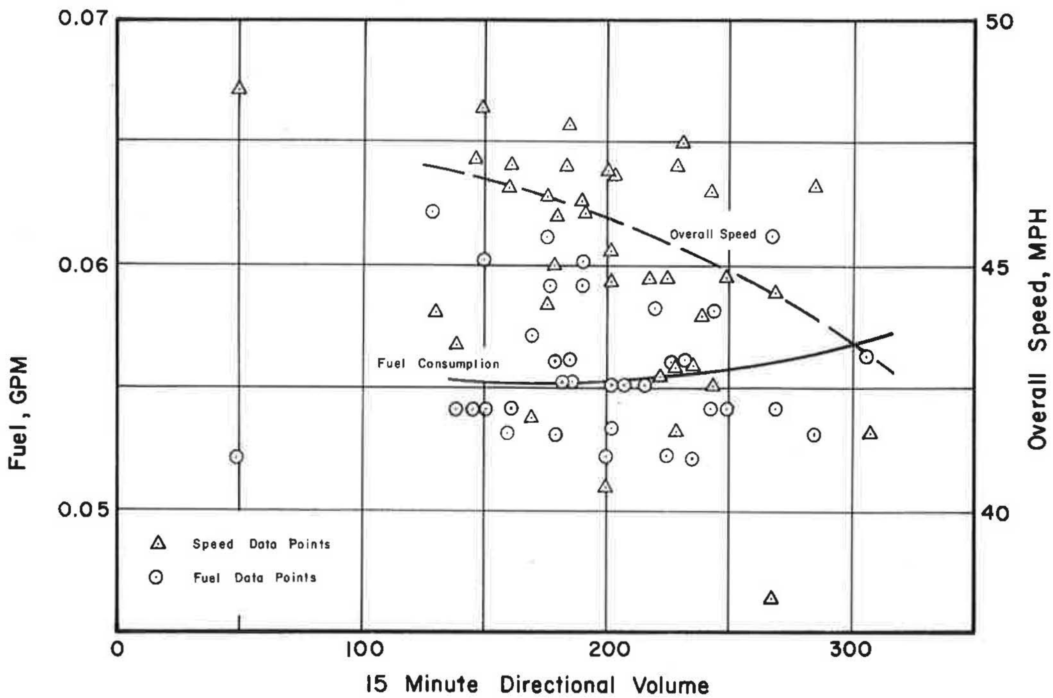


Figure 28. Relationship of fuel and travel time to traffic volume for standard passenger car on 4-lane suburban major arterial, US 99 old route (S-N).

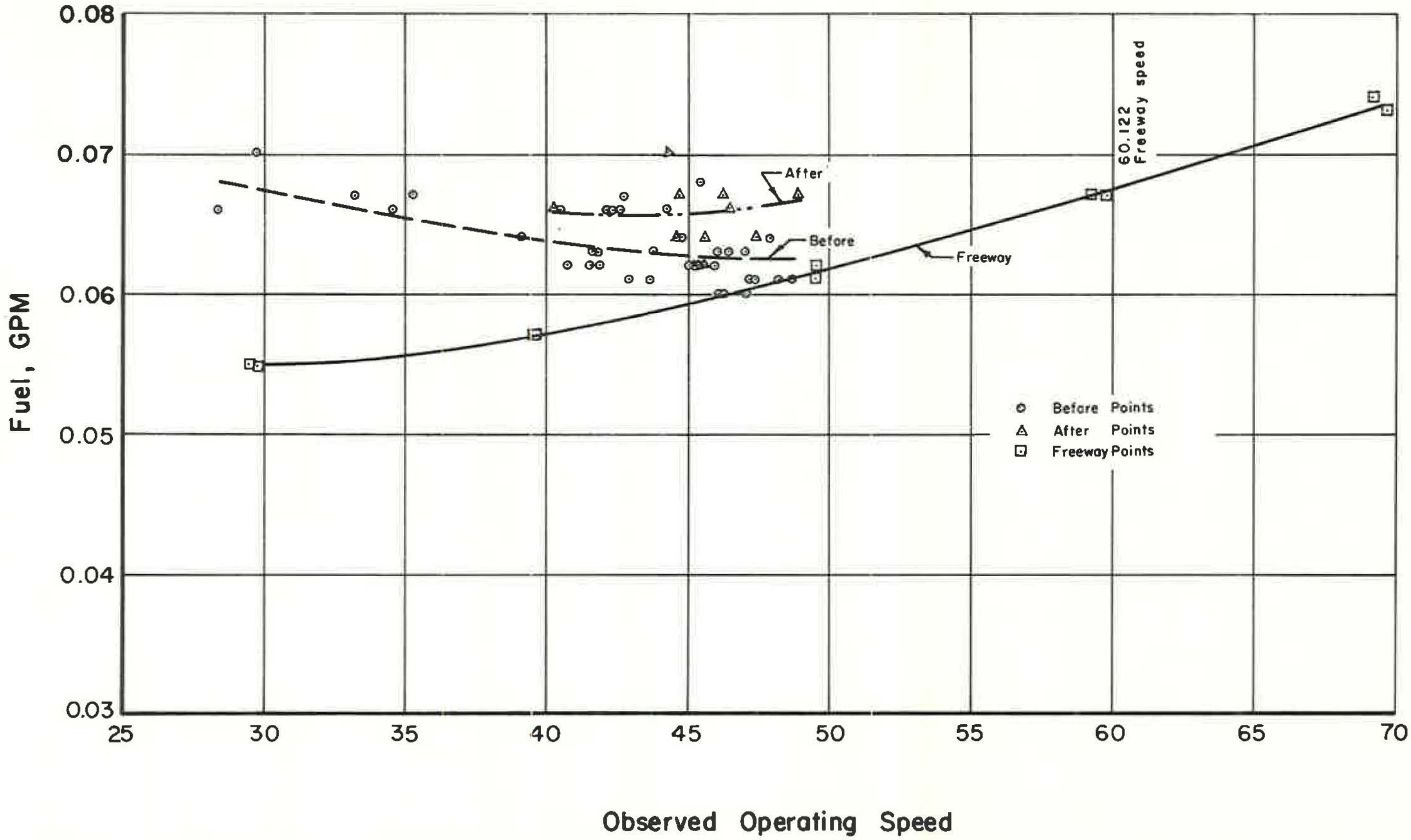


Figure 27. Relationship of fuel to overall speed for a standard passenger car on US 99 Seattle-Tacoma routes (S-N).

Appendix C

FUEL METERING DEVICES

The FM 200 (2) is a volumetric measuring device that separates out vapor so that only liquid is measured. During calibration at 71 F and measuring a quantity of 500 ml, the meter maintained a constant number of counts over a very large fuel flow range. The calibration was performed in a fuel lab using a 500 ml burette to obtain the calibration constant. The flow rate was controlled by a valve and fuel pressure was provided by an electric fuel pump. By means of the calibration constant counts could be converted to gallons by the following equations:

$$F_c = F_o - (68 - T_o)(C_e)(F_o) \qquad (1)$$

$$C_c = \frac{N_o}{F_c} \times 3785.4 \qquad (2)$$

in which

F_c = corrected fuel, ml;

F_o = observed fuel, ml;

T_o = observed fuel temperature, F;

C_e = fuel expansion coefficient at 1 F;

C_c = calibration constant, counts/gal; and

N_o = number of counts observed.

Eq. 1 is used to correct the fuel volume measured to a standard temperature of 68 F; using this corrected volume, Eq. 2 converts counts per milliliters to counts per gallon. A component part of the FM 200 is a digital counter operating from the vehicle electrical power supply. The counter can be brought to zero or turned on and off at will. For electrical continuity the FM 200 meter and the counter must be connected as they are wired in series. This is essential because the FM 200 requires electrical power to operate solenoids which position the metering valve.

The burette board (3) is a volumetric measuring device that reads directly in milliliters. This device requires no calibration but presents a problem of accurate readings while the vehicle is accelerating because the level of the liquid surface in the burette and its corresponding reading in the calibrated tubes (Fig. 4) vary. To obtain accurate readings the drivers must avoid accelerations and decelerations while passing fuel recording points. Although this was a restriction to the drivers' normal habits the distances concerned were insignificant in comparison to the route distances. Because vapor is vented to the atmosphere, only liquid is measured. The conversion constant of 3785.4 ml per gal was used to convert the fuel consumed after it had been adjusted for temperature.

Operation of the burette board requires the use of both hands because two valves are operated in rapid succession. First the burette that is being drawn from is turned off, then the next burette is turned on. The operation of the burette board type of meter requires very close attention, because it is possible to obtain false readings if the valves are not manipulated correctly.

The temperature gage used with the meters was a dial reading immersion type that read in degrees F. Temperature was recorded at the outlet on the engine side of the meters. Care was used to mount the temperature gage clear of the engine radiator when using the FM 200 to avoid errors caused by the heated air flow from the radiator. With the FM 200, the fuel temperature was recorded only at the beginning and end of the test runs. The installation with the burette board was the same as shown in Figure 4 and allowed fuel readings throughout the entire run.

Appendix D

TRAVEL TIME COLLECTION

The stopwatches as used with the FM 200 (Fig. 5) were mounted on the data recording board with the section time watches side by side so that they could be started and stopped simultaneously. The delay watch was mounted on the opposite side of the board to decrease the chance of observer error during recording. The driver's watch was mounted in the center of the steering wheel with the start button up for ease of operation.

When using the burette meters the driver's watch was mounted as with the FM 200, whereas the observer had only the delay watch mounted on the data recording board. The accumulative time watch was mounted on the burette board in a position for easy observation while operating the burette valves. At the beginning of the run the valve to the engine was closed at the same instant that the watch was started; then the valve to the first burette was opened. The slight delay in opening the burette valve made no difference in the fuel used during the first section. The time was then allowed to accumulate as the valves were operated at each additional checkpoint.

Appendix E

VOLUME WEIGHTING PROCEDURES

The following tables based on the 1963 study of US 99 business route (E-S) in Olympia, show the procedure used to adjust the sample size weighted peak averages, using the actual percent volume to obtain the volume weighted average.

Count Location[a]	Section Affected
No. 27	1 to 2
State St. between Washington and Franklin	2 to 3
	3 to 4
No. 7	4 to 5
	5 to 6
	6 to 7

[a]See Figure 1.

PERCENTAGE OF ADT AT VARIOUS COUNT LOCATIONS

Location	AM Peak	PM Peak	Off-Peak
No. 27	6.97	9.34	83.69
State	8.33	7.44	84.23
No. 7	7.50	15.23	77.27

FUEL WEIGHTING

Sect.	AM Peak			PM Peak			Off-Peak		
	Avg. Fuel	Vol. Factor	Product	Avg. Fuel	Vol. Factor	Product	Avg. Fuel	Vol. Factor	Product
1-2	0.1802	0.0697	0.0126	0.1752	0.0934	0.0164	0.1714	0.8369	0.1434
2-3	0.0666	0.0833	0.0055	0.0644	0.0744	0.0048	0.0633	0.8423	0.0533
3-4	0.0048	0.0833	0.0004	0.0100	0.0744	0.0007	0.0077	0.8423	0.0065
4-5	0.0614	0.0750	0.0046	0.0706	0.1523	0.0108	0.0634	0.7727	0.0490
5-6	0.1026	0.0750	0.0077	0.1004	0.1523	0.0153	0.0926	0.7727	0.0716
6-7	0.1208	0.0750	0.0091	0.1128	0.1523	0.0172	0.1143	0.7727	0.0883

SUMMATION OF WEIGHTED VALUES

Section	AM Peak	PM Peak	Off-Peak	Total
1-2	0.0126	0.0164	0.1434	0.1724
2-3	0.0055	0.0048	0.0533	0.0636
3-4	0.0004	0.007	0.0065	0.0076
4-5	0.0046	0.0108	0.0490	0.0644
5-6	0.0077	0.0153	0.0716	0.946
6-7	0.0091	0.0172	0.0883	0.1146
Total				0.5172

Road Resistance of Large Transport Trucks

J. W. ANDERSON, J. C. FIREY, and W. C. KIELING
Respectively, Assistant Professor, Professor, and Assistant Professor of Mechanical Engineering, University of Washington

A road test procedure, using coastdown tests at various loads, has been developed for measuring separately the air drag and tire drag force of a truck. The rayon cord tires tested showed a drag force of 8.2 lb per 1,000-lb load, as measured by these coastdown tests, and a drag force of 8.25 lb per 1,000-lb load as measured by tire dolly towing tests, indicating that the coastdown procedure yields essentially correct results. Air drag coefficients measured by the coastdown test procedure for the four different truck-trailer combinations tested varied between 0.8 and 1.2, being relatively constant for any one truck and trailer. These drag coefficients are of the same order or magnitude as those measured for truck models in wind tunnel tests by Flynn and Kyropoulos (1).

Because the air drag coefficients of full-scale trucks have not previously been measured, other comparisons of the foregoing results cannot be made. To fill in this gap, tests are currently under way to measure the air drag coefficients of full-scale trucks using a railway flat car train as a moving test bed.

•THE DEVELOPMENT of road test procedures for measuring the components of truck rolling resistance and some data obtained by using these procedures are reported. These studies were undertaken to improve the truck fuel consumption and travel time estimation procedures developed by Sawhill and Firey (2) As these estimation procedures were being developed it became evident that a significant source of uncertainty and error in the fuel consumption and travel time estimation method was the lack of general knowledge about the rolling resistance properties of trucks. Sawhill and Firey used an essentially empirical relation for truck rolling resistance which could not be used with confidence for trucks other than those tested. Hence, these studies were undertaken to develop a general relation for truck rolling resistance that could be used to estimate the fuel consumption and travel time for hypothetical future trucks and highways as well as present trucks and highways. In this manner the fuel consumption and travel time procedures may become more useful for highway economic analyses and designs.

The studies are incomplete because independent, non-road test methods of measuring air drag force and bearing and gear drag force are to be developed and compared with the road test results. These further studies will be reported subsequently.

NOMENCLATURE

The following abbreviated nomenclature will be used herein. Indicated in parentheses is the corresponding nomenclature as used by SAE (10).

G = Percent grade of a highway = 100 (Rise)/Distance, (GP);
GVW = Gross vehicle weight in lb, (GW);

Paper sponsored by Committee on Vehicle Characteristics.

A_F = Vehicle frontal area in sq ft, (A);
F = Force in lb;
F_{AD} = Air drag force of tire dolly in lb;
F_M = Measured force in lb;
ρ_a = Air density in pcf;
V = Vehicle speed in fps;
mph = Vehicle speed in miles per hour, (MPH);
g = Gravitational constant in ft/sec/sec;
C_D = Air drag coefficient;
F_T = Tire drag force in lb;
KE = Vehicle kinetic energy in ft-lb;
t = Time in sec;
T = Time in hr;
F_D = Total vehicle drag force in lb;
W = Number of wheels on vehicle;
RHP = Road horsepower, the power required to overcome total drag force, F_D, (Resistance Power);
f_t = Specific tire drag force = 1,000 F_T/GVW;
F_A = Air drag force in lb;
I = Wheel polar moment of inertia in lb-in.-sq sec;
p = Torsional oscillation period in sec;
K = Torsional spring constant in lb-in. per radian;
Θ = Torsional oscillation amplitude in radians;
Φ = Angle of tire tread deflection in radians;
l = Length of tire tread flattened portion in ft;
r = Tire tread radius in ft;
c = Tire tread elastic constant in psf per radian;
b = Tire tread width in ft;
d = Tire tread thickness in ft;
h = Tire tread hysteresis coefficient, the fraction of tread deflection work lost to internal hysteresis;
A_t = Area of the flattened portion of the tire tread in sq ft;
P_i = Tire inflation pressure in psf;
a = Ratio of tire tread flattened area to product of length and width of the flattened portion;
dQ/dt = Rate of heat transfer from the external tire area;
A_X = External tire area in sq ft;
T_t = Tire air temperature in °F;
T_a = Ambient air temperature in °F;
ΔT_t = $T_t - T_a$ in °F = Tire air temperature rise above ambient air temperature;
U = Over-all heat transfer coefficient from tire external area to ambient air temperature, ft-lb/°F/sec/sq ft; and
R = Damping coefficient.

EXPERIMENTAL PROCEDURE

Four types of experiments were carried out: (a) tire dolly towing tests to measure tire drag force; (b) truck towing tests to measure total rolling resistance force; (c) coastdown tests at various loads to measure total rolling resistance force, tire drag force and air drag force; and (d) wheel tests with a torsional spring to measure wheel kinetic energy and improve the accuracy of the coastdown tests.

Road tests were run on a 4-lane portion of US 99 extending about 10 mi north from the town of Marysville, Wash. Four northbound and four southbound test sections of uniform grade were selected and marked with stakes. The grades of these eight sections were then measured with a surveying level and chain (Table 1).

Two trucks and two semitrailers were used in combinations to give four different test vehicles: (a) truck-tractor with gasoline engine, 12 ft wheelbase, single drive axles; (b) truck-tractor with diesel engine, 10 ft 3 in. wheelbase, dual drive axles;

TABLE 1

GRADE OF TEST SECTIONS

Test Section	Direction	Grade (%)
1N	Northbound	0.314[a]
2N	Northbound	0.281[a]
3N	Northbound	0.325[a]
4N	Northbound	0.438[a]
1S	Southbound	0.502[b]
2S	Southbound	0.255[b]
3S	Southbound	0.312[b]
4S	Southbound	0.283[b]

[a]Up. [b]Down.

(c) flatbed semitrailer, 35 ft length, dual rear axles; and (d) van semitrailer, 35 ft length, dual rear axles.

The gasoline-engine truck assembled with the flatbed trailer is shown in Figure 1 and the diesel-engine truck assembled with the van trailer is shown in Figure 2. The diesel-powered truck-trailer combinations have 18 tires of 10:00-20 size. The gasoline-powered truck-trailer combinations have 14 tires of 10:00-20 size. The frontal areas of the four truck-trailer combinations are: gasoline tractor-flatbed trailer, 49.7 sq ft; gasoline tractor-van trailer, 92.9 sq ft; diesel flatbed trailer, 61.4 sq ft; diesel tractor-van trailer, 90.5 sq ft.

Variations in gross vehicle weight were obtained by loading on leased lead pigs securely fastened to the trailer bed. The lead pigs were so small that very little change of frontal area occurred for the flatbed trailer. A Washington State Highway Patrol truck weighing station was used to measure the weights of the test vehicles.

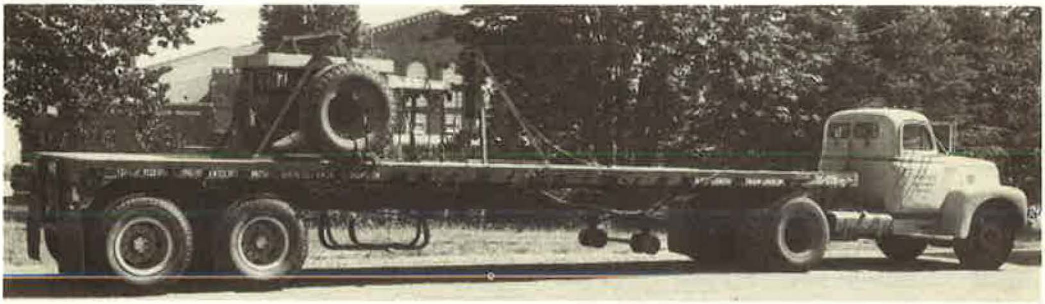


Figure 1. Gasoline tractor with flatbed trailer.

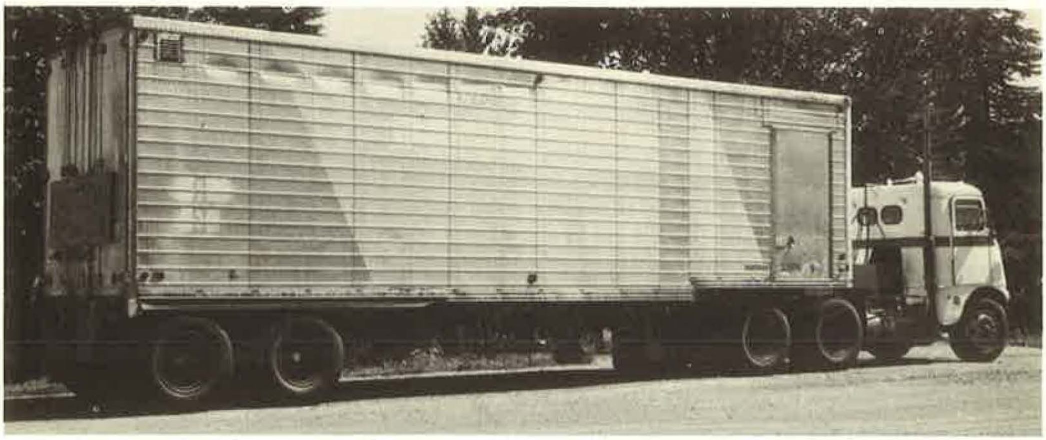


Figure 2. Diesel tractor with van trailer.

Tire Dolly Towing Tests

A separate trailer dolly was used as the tire dolly with two identical test tires mounted in the outer two of the four rims. The dolly was connected to the towing truck by the bracket shown in Figures 3 and 4, which insured that only the force component parallel to road surface was measured by the strain gage towbar. The bracket also provided a safety catch to retain the dolly if the rather delicate strain gage towbar broke. The dolly was fitted with steel brackets and clamps to hold the steel plates, which constituted the tire loading, centrally between the tires. Various numbers and thicknesses of steel plates were used to obtain five different tire loads up to the rated maximum load of the test tires (Table 2). The assembled tire dolly and towing truck are shown in Figure 5.

The valve core was removed from the tire tube valve stem and the stem was fitted with a special tee (Fig. 6). Through this tee an iron-constantan thermocouple was inserted into the tire air space and the side connection was made to a calibrated pressure gage at the wheel hub. The thermocouple leads were connected to a quick-connect fitting. In this manner the prevailing tire air pressure and temperature could be quickly measured after stopping. Tire tread temperatures were measured with a surface pyrometer fitted with a needle thermocouple.

During the tire dolly towing tests particular care was taken to hold a steady speed without acceleration or deceleration by keeping both the speedometer and engine tachom-

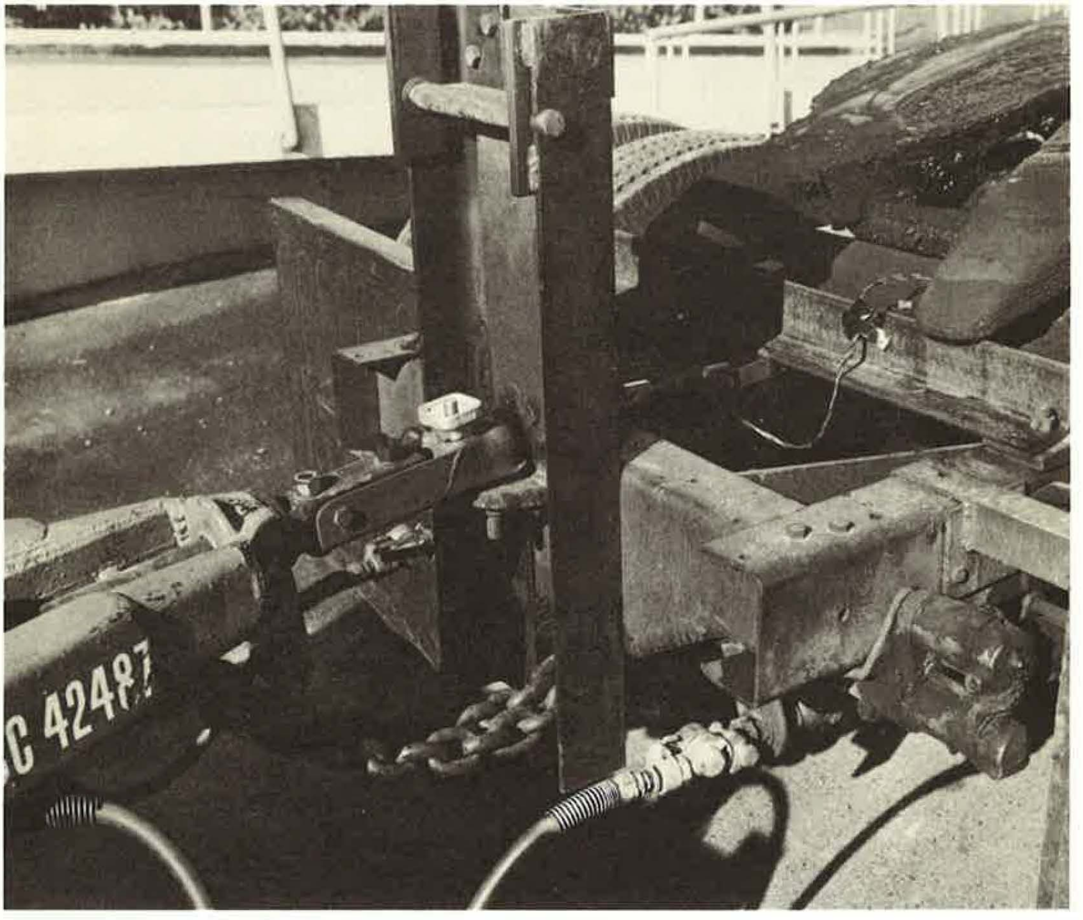


Figure 3. Towing test setup.

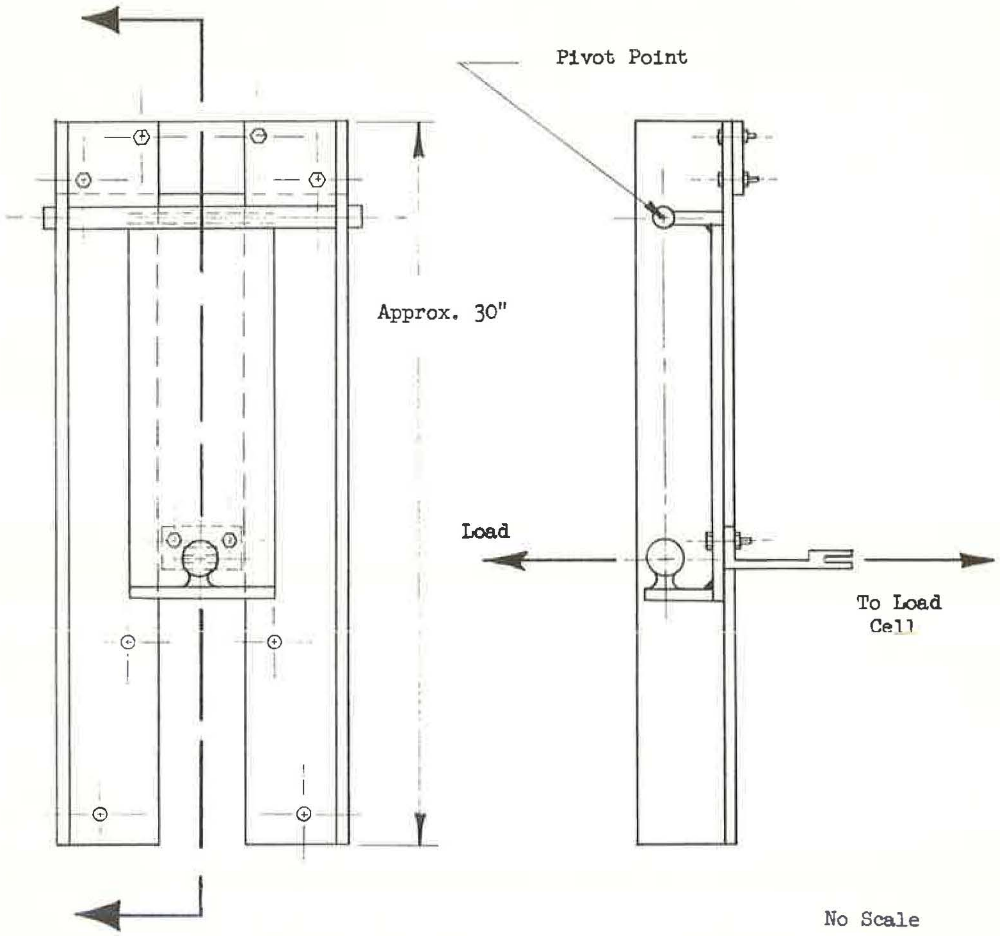


Figure 4. Towing bracket.

TABLE 2

TIRE DOLLY TOWING TEST LOADS

Load	Total Dolly Weight (lb)	Load per Tire (lb)
Empty	2,600	1,300
Quarter	4,100	2,050
Half	5,615	2,807
Three-quarter	7,500	3,750
Full	9,200	4,600

eter at constant readings. For each test condition of load and speed, tire air pressures, tire air temperatures, tread temperatures, and ambient air temperature were recorded before and after each northbound and southbound run. The towbar readings were taken only on the staked test sections of known grade.

Two types of 10:00-20 truck tires were tested, a 12-ply, rayon cord tire and a composite cord tire (steel cords in the carcass and nylon cords in the tread). Only the rayon cord tires were used in the coasting tests.

It was originally hoped that the air drag force of the dolly would be so small that it could be neglected in calculating the tire drag force. However, because this was not the case, it became necessary to measure separately the air drag force of the tire dolly. This was done by suspending the tire dolly from long vertical cables above a flatbed truck and using the towing bracket and strain gage towbar to measure the force between the tire dolly and the flatbed truck as the latter was driven at various steady speeds. This arrangement is shown in Figure 1.

Four resistance wire strain gages were mounted on the towbar made of type 6061-T6 aluminum alloy (Fig. 7). A bridge type strain gage indicating unit was connected to these four

Figure 5. Tire dolly test rig.

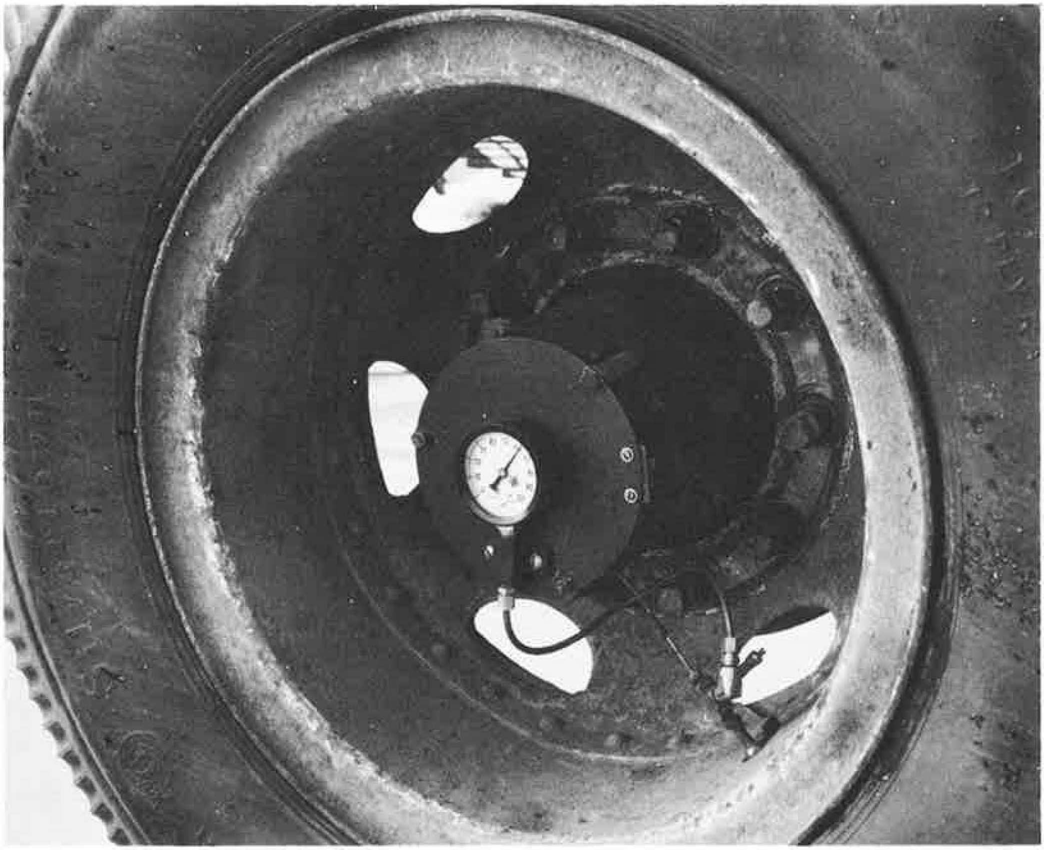


Figure 6. Tire instrumentation.

strain gages (Fig. 8). High sensitivity to strain was obtained and correction was made for thermal expansion of the towbar.

The strain gage towbar was calibrated by suspending known weights by the towbar and recording the resulting change in the strain indicator reading. This change increased linearly with applied weight and the towbar force was calculated as the product of the change in strain

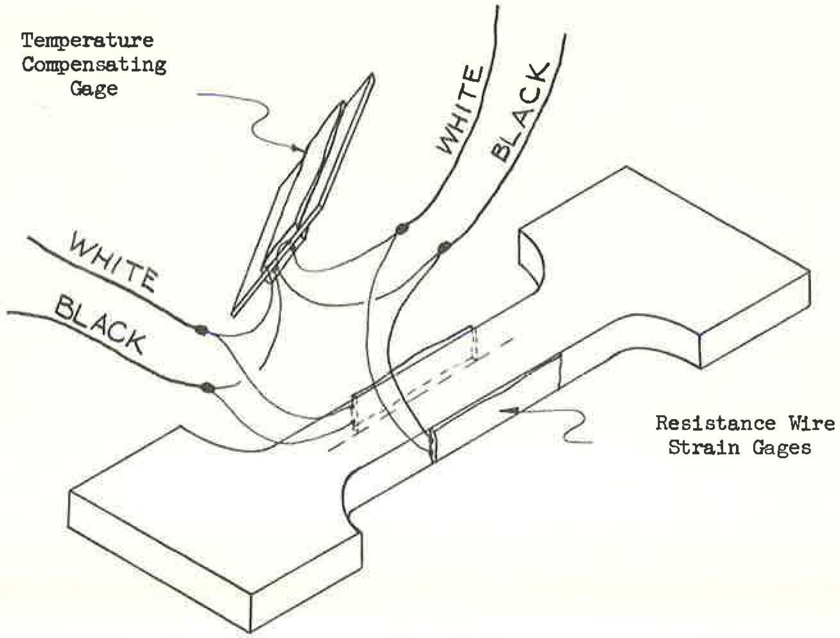


Figure 7. Towing test strain gage towbar.

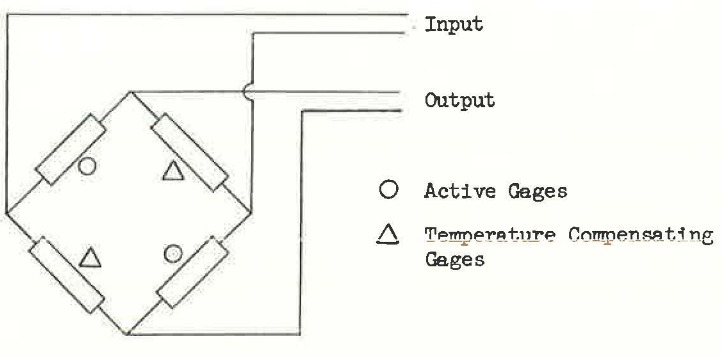


Figure 8. Schematic strain gage wiring.

indicator reading and the towbar calibration coefficient. The zero load strain indicator reading was read at the start and end of each separate experiment and was found to drift only slightly from day to day. The towbar was recalibrated periodically during the experiments but the calibration coefficient did not change measurably.

Tire dolly air drag force was calculated by subtracting the product of dolly weight and test section slope from the towbar force measured during the flatbed tests:

$$F_{AD} = F_M - (GVW)\left(\frac{G}{100}\right) \tag{1}$$

The results of these calculations (Fig. 9) show the variation of tire dolly air drag force, F_{AD}, with speed. This air drag force fits the usual drag force equation very well with a calculated drag coefficient, C_D, of 0.705 which appears reasonable:

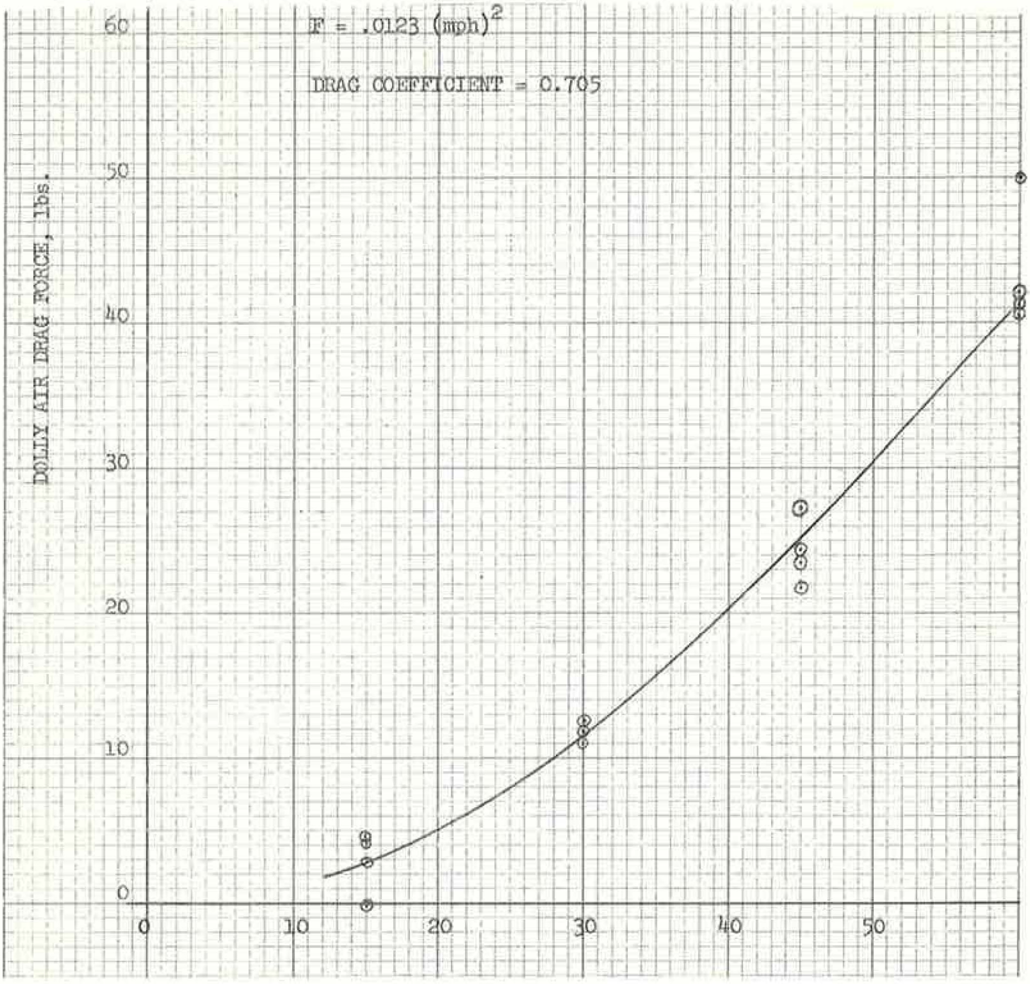


Figure 9. Tire dolly air drag force, averaged results.

$$F_{AD} = C_D A_F \rho_a \frac{V^2}{2g} = 0.0123\ (\text{mph})^2 \qquad (2)$$

Tire drag force was calculated by subtracting the air drag force, F_{AD}, and the slope force (product of dolly weight and test section slope) from the towbar force measured during the tire dolly towing tests:

$$2F_T = F_M - F_{AD} - (\text{GVW})\left(\frac{G}{100}\right) \qquad (3)$$

The results of these calculations (Figs. 10 through 13) show the variation of tire drag force per tire, F_T, with tire load speed.

The observed tire air temperature rise above ambient is plotted against tire load at various speeds in Figures 14 through 17. The observed tire tread temperature rise over ambient varied in essentially the same manner as for the tire air temperature rise but the tread data were more scattered.

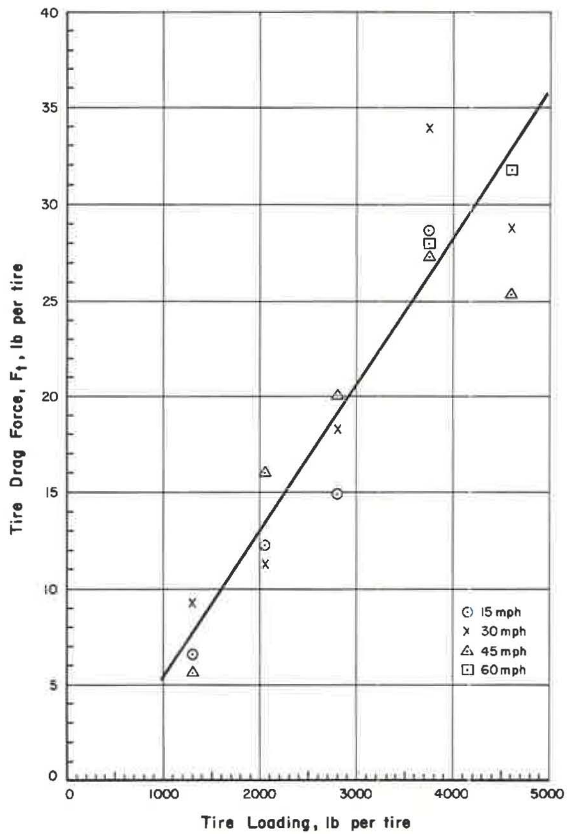


Figure 10. Tire drag force data, steel cord tires at 60 psig, 10:00-20.

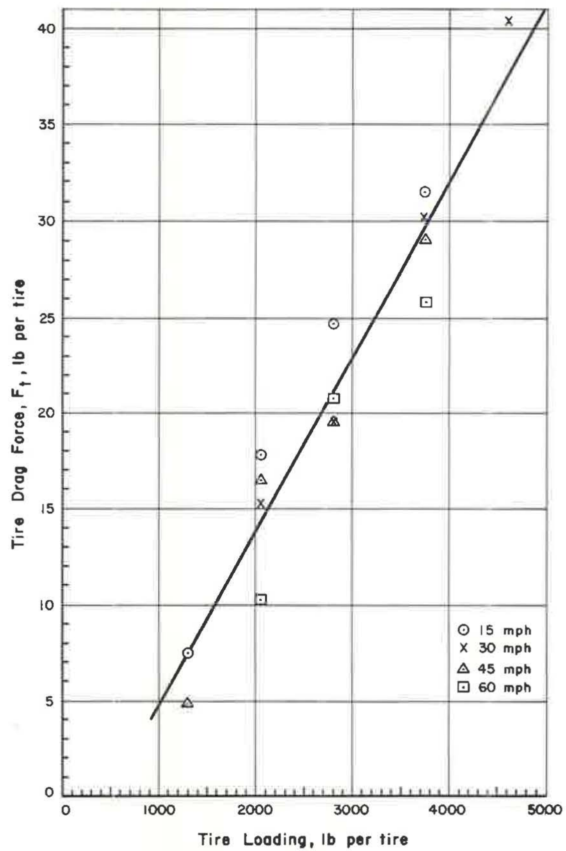


Figure 11. Tire drag force data, steel cord tires at 75 psig, 10:00-20.

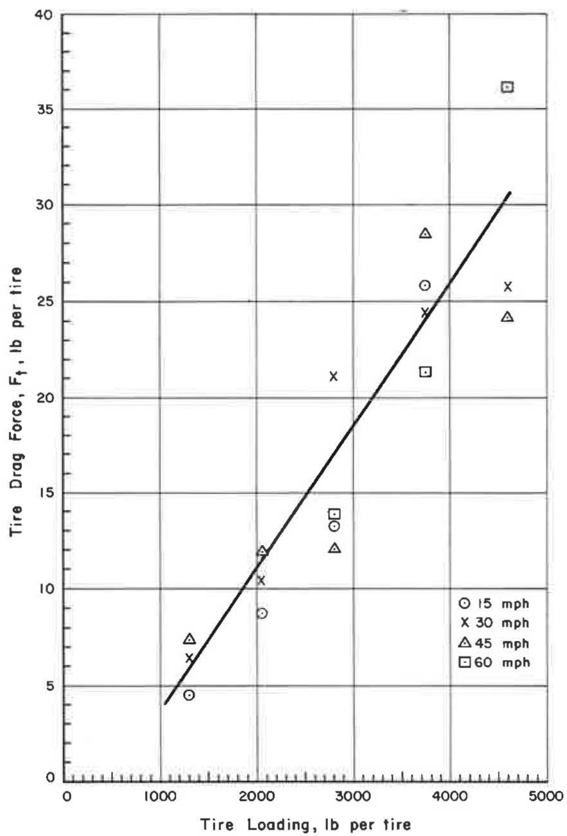


Figure 12. Tire drag force data, steel cord tires at 90 psig, 10:00-20.

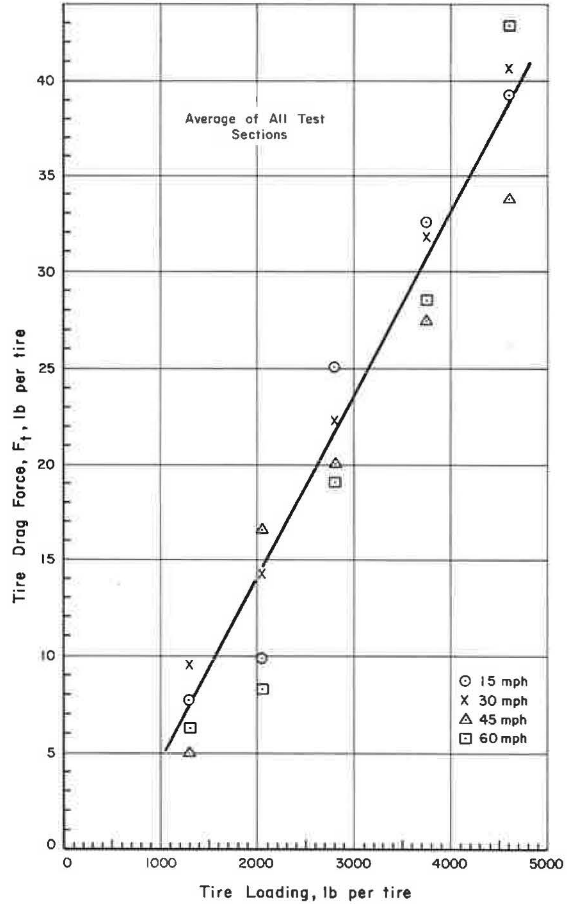


Figure 13. Tire drag force data, rayon cord tires at 75 psig, 10:00-20.

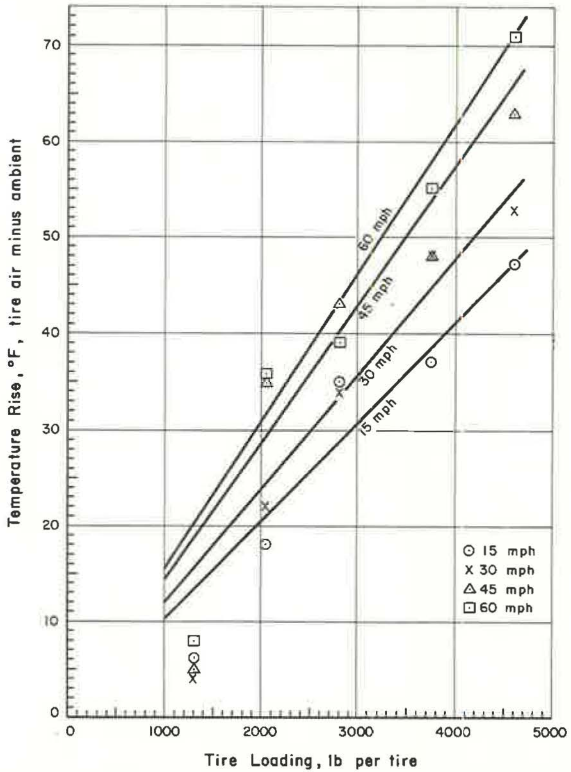


Figure 14. Tire temperature rise data, steel cord tires at 60 psig, 10:00-20.

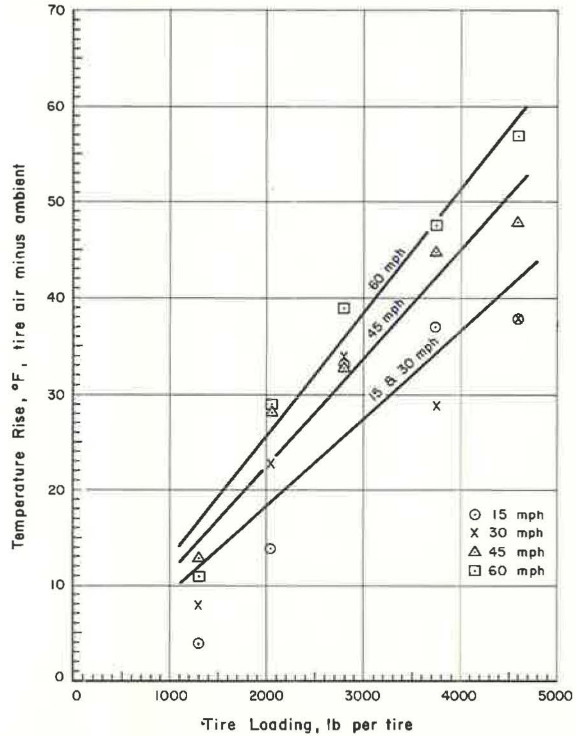


Figure 15. Tire temperature data, steel cord tires at 75 psig, 10:00-20.

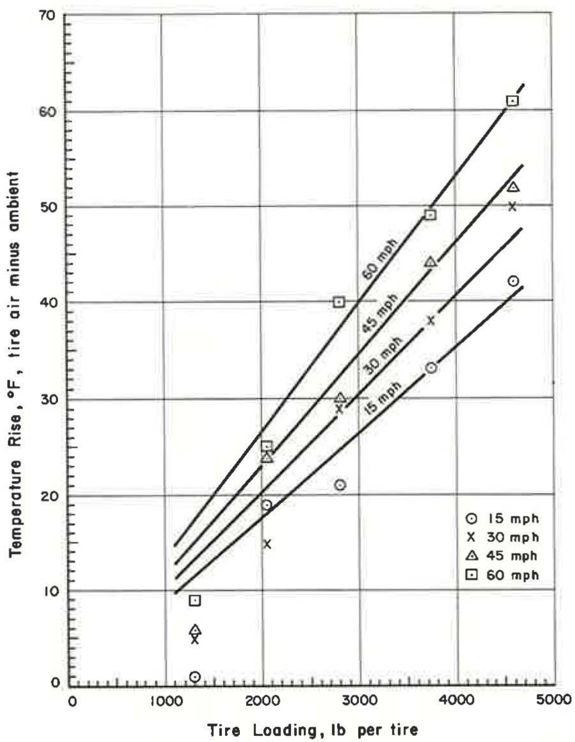


Figure 16. Tire temperature data, steel cord tires at 90 psig, 10:00-20.

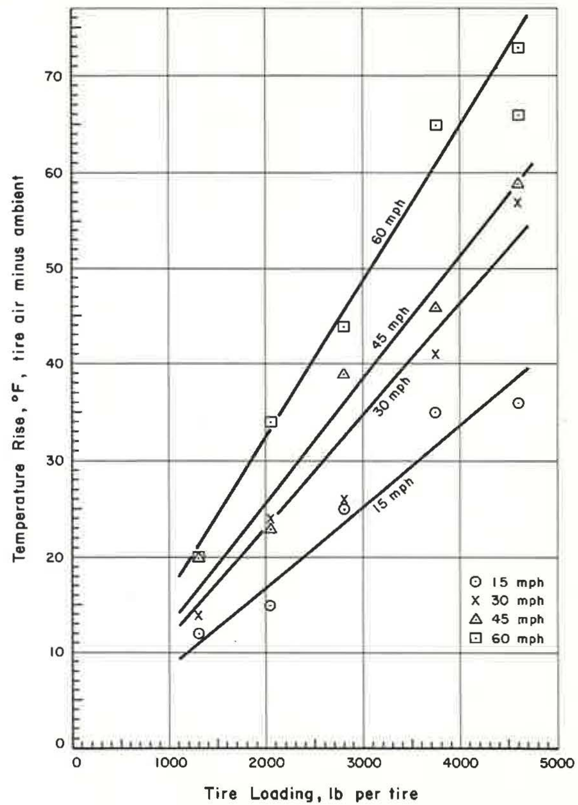


Figure 17. Tire temperature data, rayon cord tires at 75 psig, 10:00-20.

Truck Towing Tests

One of the test trucks was towed via the same bracket as used with the tire dolly towing tests. The test truck was towed at steady speeds between 30 and 45 mph and the towing force was recorded on the staked test sections. Tests were limited to this narrow range of speeds because at lower speeds steady towing force readings were difficult to obtain and higher speeds were considered unsafe. After running some of these towing tests it became apparent that towing tests on large trucks are unsafe at any speed on a public highway where unexpected sudden stops may be required. Accordingly only a limited amount of truck towing data was obtained. A comparison is shown in Figure 18 between total drag force as measured by the truck towing test and that as measured by the coastdown test. Both test methods give essentially the same answer. Further measurements of total drag force of the test trucks were made by coastdown tests only.

Coastdown Tests

The coastdown test (3, 4) consists of bringing the truck up to a speed of about 60 mph, disengaging the clutch and recording speed and time as the truck coasts to a stop. During the coastdown, the truck kinetic energy is being used to overcome the total drag force, which can be calculated from the rate of loss of kinetic energy. To obtain improved accuracy the following detailed changes were made in the test procedure of Sawhill and Firey (4):

1. A special calibrated speedometer indicating head driven by the truck speedometer cable was used instead of the usual truck speedometer or a fifth wheel. The gearbox

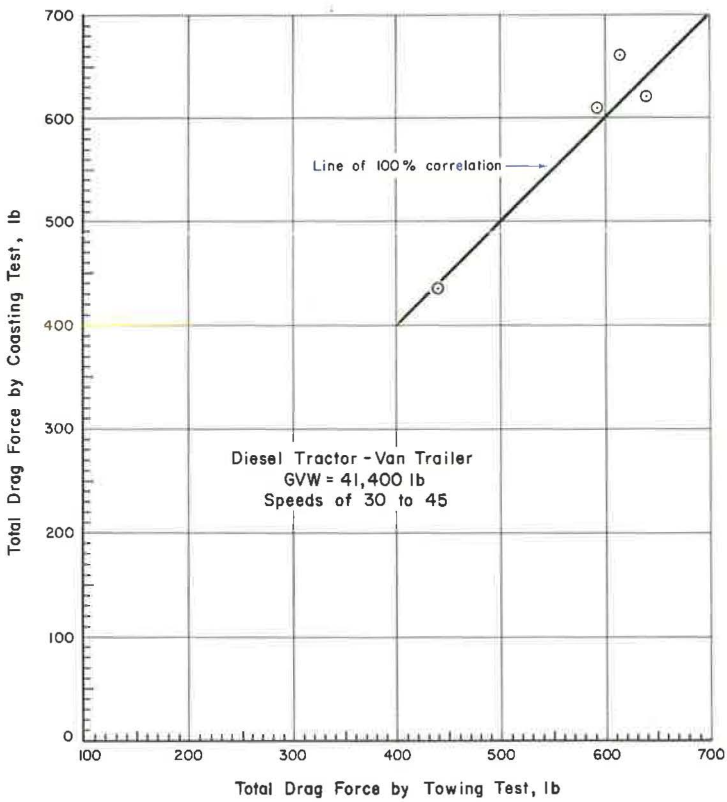


Figure 18. Comparison of total vehicle drag force as measured by coasting and towing tests.

between the speedometer cable and the indicating head was selected for each truck so that true speed was indicated. The special indicating head was more readable and more accurate than the truck speedometer. By using the truck speedometer cable the problem of fifth wheel bouncing was avoided.

2. The tests were run only on the staked test sections of known and uniform grade so that corrections for slope could be made accurately. Because these test sections were short, the full coastdown was done in two or more parts. In previous coastdown tests, the full coastdown was run at once and invariably a change of grade occurred somewhere during the test and only an approximate correction could be made for slope. Even a small error in grade correction can introduce a large error in total drag force.

3. Mr. Carl Saal suggested the use of a half-silvered glass with one edge straight for determining the position of the line tangent to the graph of truck speed vs time. The slope of this tangent line gives the truck deceleration and, hence, the total drag force. A half-silvered glass was not only a more accurate but also a more convenient means for determining the tangent line than the previous straightedge method.

When coasting on a grade, truck kinetic energy is utilized to overcome total drag force and to increase truck potential energy:

$$\frac{d(KE)}{dt} = F_D V + (GVW)\left(\frac{G}{100}\right) V \tag{4}$$

After introducing the kinetic energy equations and rearranging the units this relation becomes (4)

$$\frac{GVW}{29.6 \times 10^6}\left[1 + 128\,\frac{(W + 1)}{GVW}\right](mph)\left(\frac{\Delta mph}{\Delta T}\right) = \frac{F_D}{375}\,(mph) + \left(\frac{GVW}{375}\right)\frac{G}{100}\,(mph) \tag{5}$$

Solving for the total drag force, F_D, yields

$$\frac{F_D}{375} = \frac{RHP}{mph} = \frac{GVW}{29.6 \times 10^6}\left(\frac{\Delta mph}{\Delta T}\right) + \frac{128\,(W + 1)}{29.6 \times 10^6}\left(\frac{\Delta mph}{\Delta T}\right) - \frac{GVW\ (G)}{37,500} \tag{6}$$

On the right side of Eq. 6 the first term is the truck deceleration force, the second term is the wheel deceleration force, and the third term is the force due to grade. The truck deceleration was measured from the slope of the speed vs time graph during coastdown. Values of the total drag force, calculated in this manner, are plotted as the total drag force vs gross vehicle weight at constant speeds (Figs. 19 through 22) and as the total drag force vs the square of truck speed at constant gross vehicle weights (Figs. 23 through 26).

The linear increase of total drag force with increasing weight at constant speed can only be due to change of tire drag force because the air drag properties of the truck are not changed by weight. By assuming tire drag to be zero at zero load, as indicated by the tire dolly towing test results, the tire drag force can be calculated at each speed from the slope of the total drag force vs vehicle weight graphs:

$$F_T = 375\ (GVW)\ (slope),\ lb \tag{7}$$

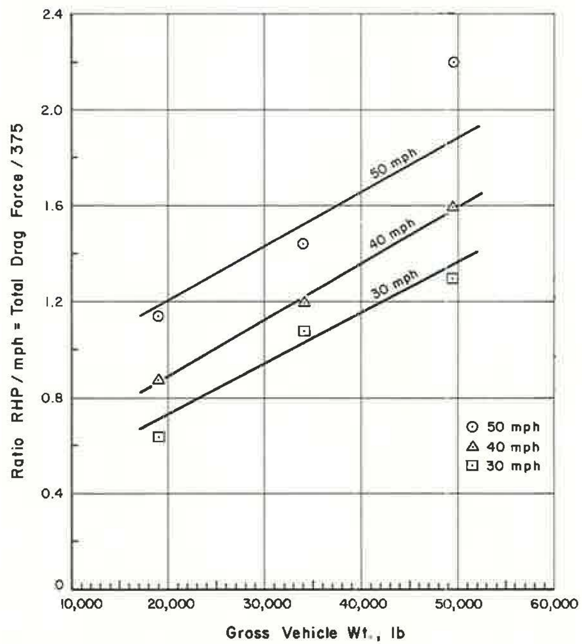


Figure 19. Total vehicle drag force by coasting test vs vehicle weight, gasoline tractor with flat-bed trailer.

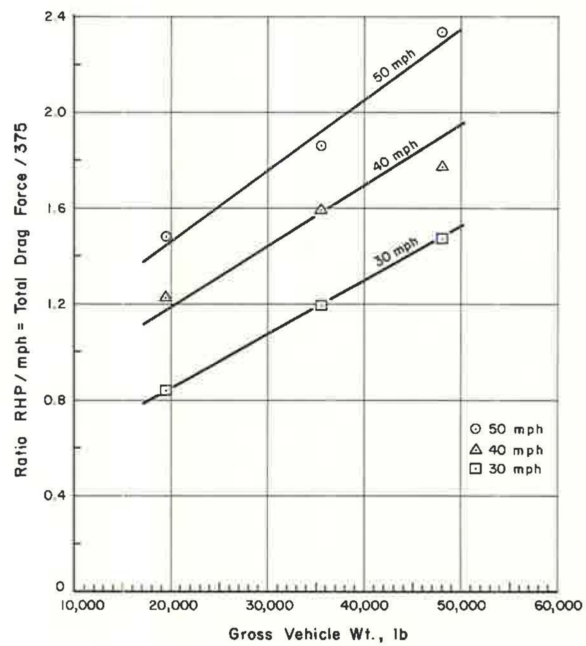


Figure 20. Total vehicle drag force by coasting test vs vehicle weight, gasoline tractor with van trailer.

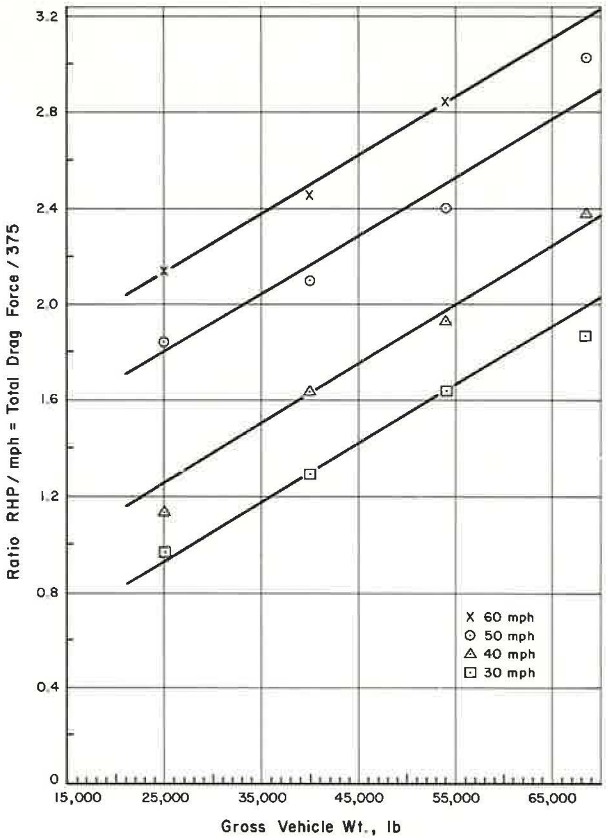


Figure 21. Total vehicle drag force by coasting test vs vehicle weight, diesel tractor with flat-bed trailer.

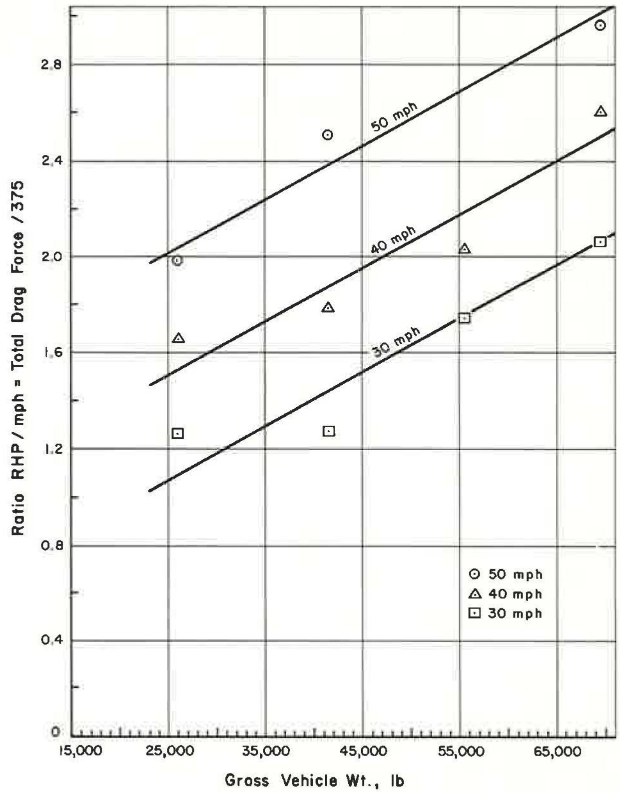


Figure 22. Total vehicle drag force by coasting test vs vehicle weight, diesel tractor with van trailer.

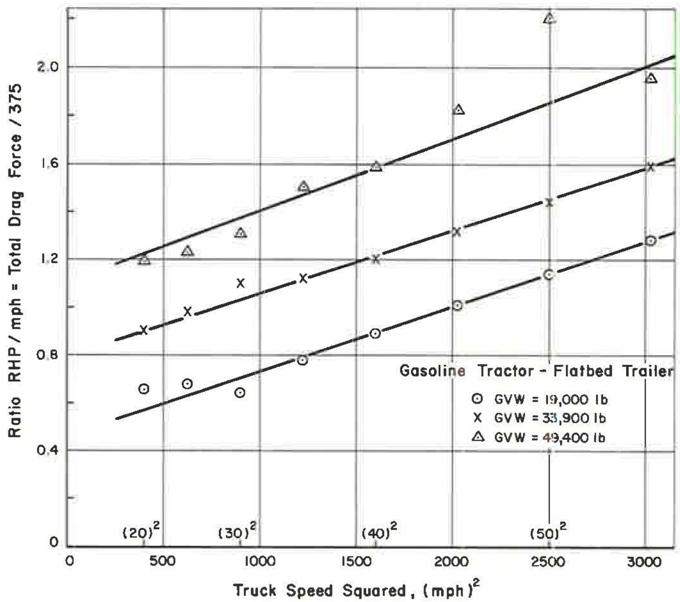


Figure 23. Total drag force vs speed squared.

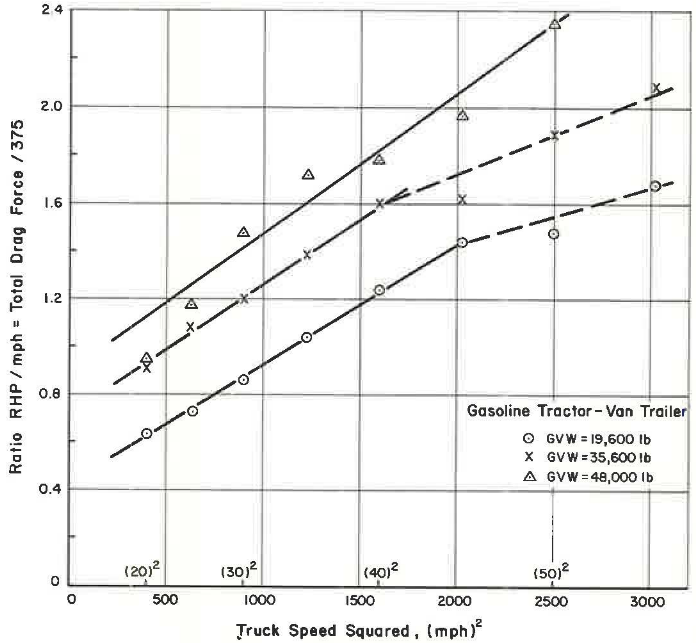


Figure 24. Total drag force vs speed squared.

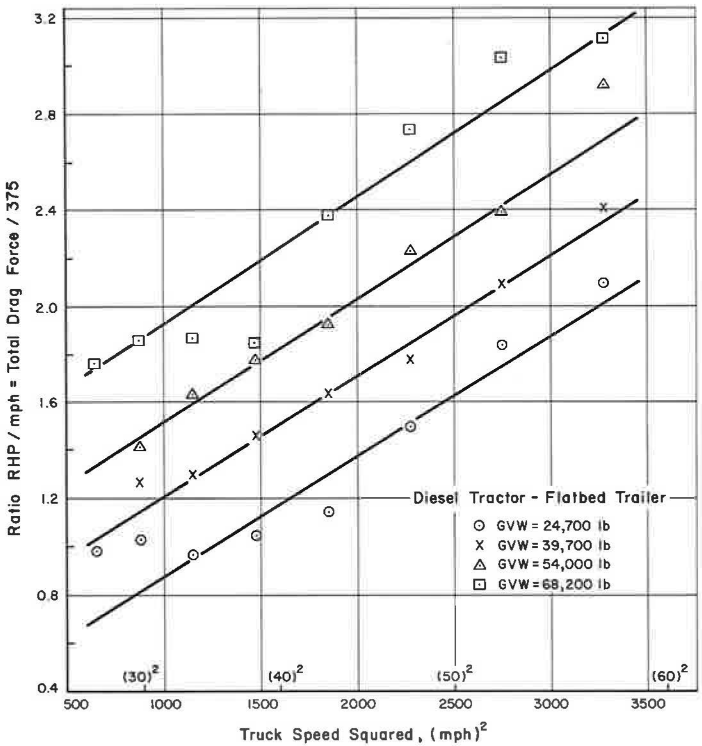


Figure 25. Total drag force vs speed squared.

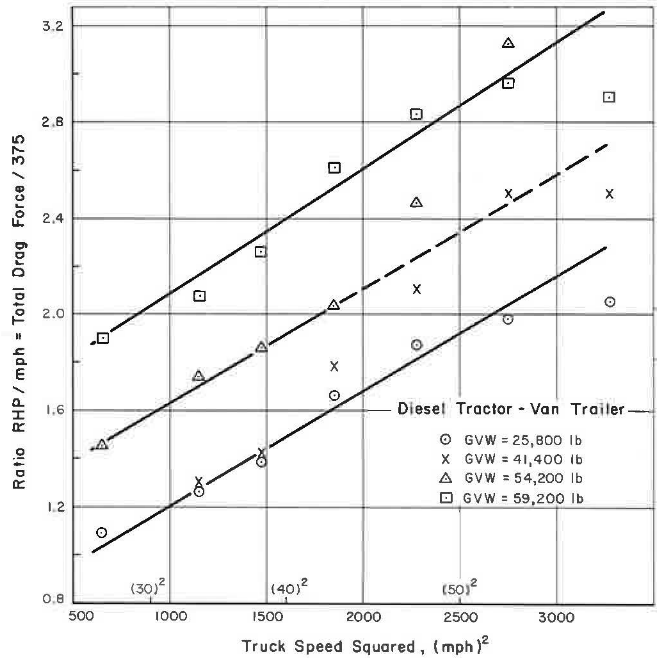


Figure 26. Total drag force vs speed squared.

in which the slope is in units of RHP/[(mph) (GVW)]. Tire drag force is also calculated in units of specific tire drag force, f_t, in pounds drag per 1,000-lb load. Tire drag force of the rayon cord test tires, calculated in this manner, was essentially independent of truck speed (Fig. 27). This result is in aggrement with the data obtained by the tire dolly towing tests shown in Figure 13. For the rayon cord tires tested, the coastdown tests gave an average specific tire drag force of 8.20 lb per 1,000 load, which agrees closely with the average specific tire drag force of 8.25 lb per 1,000 lb load obtained from the tire dolly towing tests.

The increase of total drag force with speed at constant load was presumed to result from the change of air drag force. This is equivalent to assuming tire drag force to be independent of speed as is shown in Figures 10 through 13. Air drag force was then calculated from the slope of the total drag force vs speed squared graphs:

$$F_A = 375\ (\text{mph})^2\ (\text{slope}),\ \text{lb} \tag{8}$$

in which the slope is in units of RPH/(mph)^3.

The air drag coefficient, C_D, was calculated for each vehicle by fitting the air drag force to an air drag equation:

$$F_A = C_D A_F \frac{\rho_a V^2}{2g} \tag{9a}$$

$$C_D = \frac{2gF_A}{A_F \rho_a V^2} \tag{9b}$$

The truck air drag coefficients, calculated in this manner, are given in Table 3.

This method for separately determining tire drag force and air drag force from coastdown tests is essentially that described by Beck (5) and is equivalent to assuming

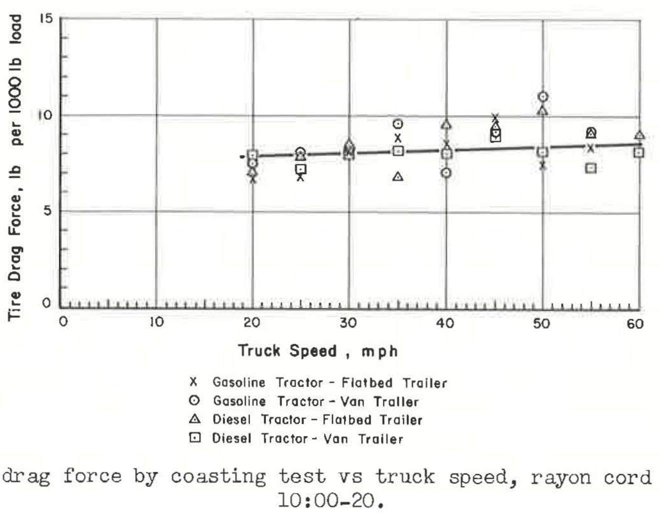


Figure 27. Tire drag force by coasting test vs truck speed, rayon cord tires at 75 psig, 10:00-20.

TABLE 3

TEST VEHICLE AIR DRAG COEFFICIENTS

Vehicle	C_D
Gasoline tractor, flatbed trailer	0.835
Gasoline tractor, van trailer	0.873
Diesel tractor, flatbed trailer	1.240
Diesel tractor, van trailer	0.813

total drag force consists of a weight dependent term, the tire drag force, and a speed squared dependent term, the air drag force:

$$F_D = F_T + F_A = GVW \frac{f_t}{1,000} + C_D A_F \left(\frac{\rho_a V^2}{2g}\right) \tag{10}$$

The following assumptions are implied in using this calculation procedure: (a) the tire drag force is zero at zero load; (b) it is independent of speed; (c) it increases linearly with load; and (d) gear and bearing friction in the drive train and wheels is negligible. Assumptions (a), (b), and (c) are approximately verified for the rayon cord tires by the tire dolly towing test results shown in Figure 13. By the foregoing calculation method the gear and bearing friction force is split into two parts, the load dependent part being included with the tire drag force and the speed dependent part being included with the air drag force. Future experiments are planned to measure separately the gear and bearing friction force.

Wheel Tests with a Torsional Spring

Part of the truck kinetic energy, calculated in the coastdown tests, is translational and part is rotational. The translational kinetic energy is calculated from the measured speed and weight. The wheel polar moment of inertia is needed to calculate the rotational kinetic energy. In previous studies (4) an estimated value of wheel moment of inertia was used. In this work the wheel moment of inertia is directly measured by coupling the wheel to a torsional spring and measuring the natural frequency of torsional vibration of the system. Details of this torsional spring unit are shown in Figure 28. The spring was a ½-in. steel bar of 24-in. length having a spring constant of 3,070 lb-in. torque per radian. To measure the oscillations of the wheel, four strain gages were mounted on the spring at 45° to the axis. By mounting two gages 180° apart on a right-hand helix and two gages similarly on a left-hand helix, the effects of bending, axial loads and thermal expansion were minimized. The test wheel was jacked up, connected to the spring and given an initial rotation of 25°. The decaying wheel oscillations were sensed by the strain gages and the imbalance of the gage bridge circuit was recorded on a strip chart recorder. From the strip chart record the natural frequency, or vibration cycle period, of the wheel and spring system was measured. The wheel polar moment of inertia was then calculated by use of the usual torsional vibrational relation:

$$I = \frac{p^2 K}{4\pi^2} \tag{11}$$

If the effect of damping (assumed viscous) is included the more accurate relation becomes

$$I = \frac{P^2 K}{4\pi^2 + R^2} \tag{12}$$

Damping was calculated by measuring the decrease in oscillation amplitude from one cycle to the next:

$$R = \log\left(\frac{\Theta_1}{\Theta_2}\right) \tag{13}$$

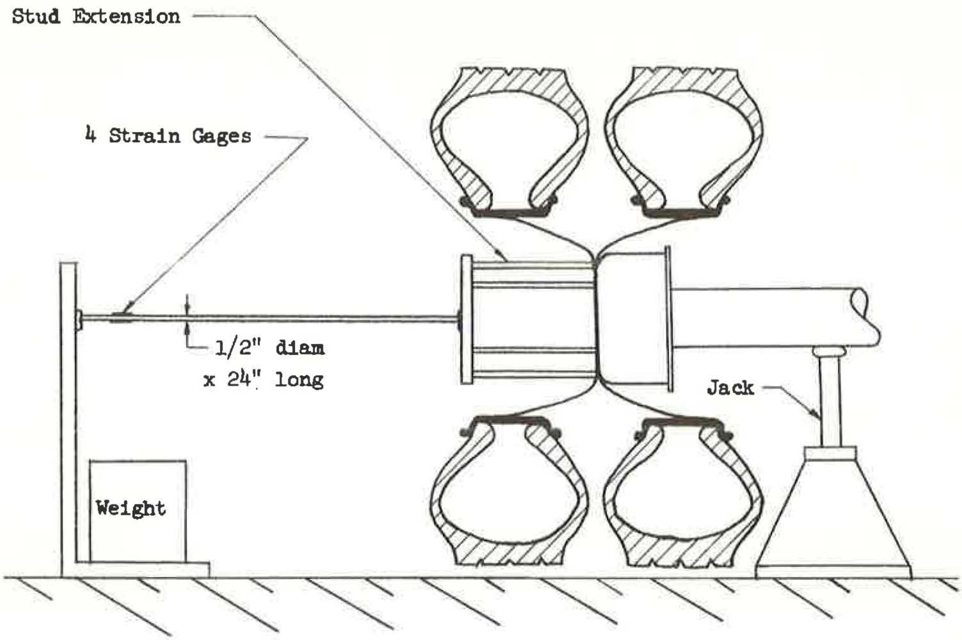


Figure 28. Wheel torsional spring unit.

Several sets of dual wheels were tested with the following average results: period = P = 1.81 ± 0.02 secs; damping = R = 0.40; and Polar moment of inertia = I 258 lb-in.-sec^2 for two wheels and rims with 10:00-20 tires and a brake drum. Extreme accuracy is not ordinarily needed in determining the wheel polar moment of inertia because rotational kinetic energy of a large truck is only between 5 and 10 percent of the total kinetic energy.

DISCUSSION

Tire Drag Force

Tire drag force appears to originate largely from hysteresis of the tire rubber. The rubber in the tire tread is bent through an angle, Φ, as the tread enters the flattened part on the road. This deflection produces stresses in the rubber and work is done on the rubber. Due to the nature of rubber only a part of this work is recovered when the tread rubber unbends on leaving the flattened part. The loss in work produces both the tire drag force and internal heating and temperature rise within the tire.

The following rather crude analysis of tire deflections may serve to indicate qualitatively some of the factors which influence tire drag force. The angle, Φ, through which the tread rubber is bent is related to the length, l, of the flattened portion as

$$\Phi = \frac{l}{2r} \tag{14}$$

The total work done in thus bending the tread rubber varies directly with Φ and the volume of rubber bent:

$$\text{Work} = c\,\Phi b d \text{ (velocity) (time)} \tag{15}$$

$$\text{Volume} = b\,d\,(\text{velocity})\,(\text{time}) \tag{16}$$

A constant fraction, h, of this work is lost internally to hysteresis:

$$\text{Lost Work} = h\,c\,\Phi b\,d\,(\text{velocity})\,(\text{time}) \tag{17}$$

This lost work should equal the product of tire drag force, F_t, truck velocity and time:

$$\text{Lost Work} = F_t\,(\text{velocity})\,(\text{time}) \tag{18}$$

$$F_t = h\,c\,\Phi b\,d = \frac{h\,c\,l\,b\,d}{2\,r} \tag{19}$$

The area, A_t, of the flattened portion is approximately equal to the ratio of load, W, to inflation pressure, p_i:

$$A_t = \frac{W}{p_i} \tag{20}$$

This area is also equal to a constant, a, times the length, l, and width, b, of the flattened portion:

$$A_t = a\,l\,b = \frac{W}{p_i} \tag{21a}$$

$$lb = \frac{W}{a\,p_i} \tag{21b}$$

$$F_t = \frac{W\,h\,c\,d}{2\,a\,r\,p_i} \tag{22}$$

An approximate experimental verification of this relation is found in the following observations:

1. Tire drag force increases very nearly linearly with tire load (Figs. 10 through 13).
2. Tire drag force usually decreased at higher inflation pressure (Figs. 10 through 12), although the effect was not as large as indicated by Eq. 22.
3. Measurements were made of the area of the flattened portion for the rayon cord tires at 75 psig inflation pressure. The product of inflation pressure and flattened area equaled applied load within about 20 percent.
4. Previously reported experiments (6) suggested that worn tires, having a smaller tread thickness, d, showed lower values of tire drag force.

Eq. 22 can only be very approximate because many complicating factors, such as deflections of the sidewall rubber and variations of a with load, were not considered. Nevertheless, this analysis appears to provide a qualitatively correct picture of the origin of the tire drag force.

The portion of the tire tread bending work lost to hysteresis reappears as increased tire and tire air temperature. Presumably at equilibrium tire temperature, the rate of hysteresis work equals the power required to overcome tire drag force and also equals the rate of heat transfer from the tire to the ambient air:

$$F_T \text{ (velocity)} = \text{hysteresis power} = \frac{dQ}{dt} \tag{23}$$

$$F_T \text{ (velocity)} = UA_x \, (T_t - T_a) = UA_x \Delta T_t \tag{24}$$

Because ΔT_t is the tire temperature rise above ambient by definition:

$$\Delta T_t = \frac{F_T \text{ (velocity)}}{UA_x} = \frac{Whcd \text{ (velocity)}}{2UA_x \, arp_i} \tag{25}$$

This relation predicts that tire temperature rise increases linearly with load as was observed experimentally (Figs. 14 through 17). The observed effects of tire inflation pressure, p_i, and truck speed on tire temperature rise Figs. 14 through 16) agree qualitatively but not quantitatively with Eq. 25. The relatively small effect of velocity on tire temperature rise may result from an increase of heat transfer coefficient, U, from tire to ambient air due to increased velocity. Turbulent flow heat transfer coefficients increase with about the 0.8 power of velocity. Hence, tire air temperature rise should then increase with about the 0.2 power velocity, as was indeed observed for the data of Figures 14 through 17.

Interest in tire tread and tire air temperatures originally centered around the thought that the rubber hysteresis coefficient, h, varied with rubber temperature, being generally lower at higher temperature. Hence, at higher load, tire drag force would not be proportionately increased because tire temperatures would be greater. Because the experimental results show tire drag force to increase nearly linearly with tire load it would appear that variations of the rubber hysteresis factor with temperature are small within the range of temperatures of these experiments (70 to 140 F).

The tire drag force data reported herein do not agree very well with previous data on truck tires reported by Stiehler et al. (7). Stiehler measured tire drag force in a laboratory apparatus by running the test tires against a steel drum of about 5.5 ft diameter. Both sets of results agree on the effect of load except that Stiehler shows that an appreciable tire drag force exists at zero load. It is difficult to reconcile the two sets of results except on the possible basis that a tire deflects very differently against the steel drum than it does against a flat highway surface.

Air Drag Force

The air drag force of a truck results from skin friction drag over the entire external surface area of the truck and pressure drag due to air displacement by the frontal area of the truck. For passenger cars, pressure drag appears to be much larger than skin friction drag (8). Presumably this is also true for large transport trucks, although very little experimental data are published concerning the air drag properties of large trucks. Judging from experimental results on passenger cars and car models (8) a significant portion of motor vehicle drag results from protuberances such as rear view mirrors, and door knobs, and particularly from protuberances on the underside such as springs and axles. This protuberance drag is primarily pressure drag.

For truck speeds greater than about 30 mph the air drag coefficients, C_D, as measured by these road test methods are in reasonable agreement with air drag coefficients for truck models as measured by Flynn and Kyropoulos in wind tunnel tests (1).

Some of the uncertainties about the true air drag of full-scale trucks are expected to be resolved by the forthcoming railroad flatcar tests of truck air drag and the wheel spinning tests of bearing and gear friction. For the railroad flatcar tests, the test trucks will be suspended on flatcars and the air drag force measured in a manner similar to that used for measuring the tire dolly air drag force. To measure bearing friction a nondrive wheel is spun up and speed and time recorded as the wheel coasts to a stop. During the coastdown, wheel rotational kinetic energy is utilized to overcome bearing drag which can be estimated from the rate of loss of kinetic energy. A similar experiment on a set of drive wheels will permit estimation of the gear and drive train friction. It is hoped that the air drag results obtained by coasting test, when corrected for bearing and gear drag, will agree fairly closely with the air drag results obtained by railroad flatcar test.

Application to Fuel Consumption and Travel Time Estimation Procedures

The results reported herein indicate that truck rolling resistance can be closely approximated as being composed of a speed squared dependent term, the air drag, and a gross vehicle weight dependent term, the tire drag. In the fuel consumption analysis of Sawhill and Firey (2) truck rolling resistance was assumed to depend only on gross vehicle weight. The analysis method used cannot be readily modified to include correctly the air drag effect. This limitation applies also to their travel time analysis, because the fuel consumption calculation is an integral part of the travel time estimation procedure. Perhaps some of the discrepancies noted by Sawhill and Firey between their theoretical and empirical fuel consumption analyses resulted from the incorrect rolling resistance relation used.

The difficulty originates from the necessity of knowing the truck speed before the air drag force can be estimated. The truck speed, in turn, is to some extent dependent on the magnitude of the air drag force. An approximate way out of this dilemma appears to lie in the following method for fuel consumption and travel time estimation for a particular truck on a specified section of highway:

1. The maximum speed-distance-time history of the truck on the highway can be estimated by assuming the truck to be at legal speed limit or, where this cannot be maintained, at maximum sustained speed as calculated by the methods of Firey and Peterson (9). The travel time is now directly calculable.
2. With speeds known the various drag forces can be determined, as well as the duration of wide-open throttle running, and hence, the fuel consumption can be calculated by the methods of Sawhill and Firey (2) modified to include an air drag power term.

Although the details of this calculation procedure remain to be worked out, it appears likely that it will be a more cumbersome calculation than the relations developed earlier. However, the proposed calculation procedure can probably be used with greater confidence in predicting the effects of future truck and highway designs on fuel consumption and travel time.

ACKNOWLEDGMENTS

The writers wish to express their appreciation of the generous financial assistance given this research by the U. S. Bureau of Public Roads under Research Contract CPR 11-8016. The cooperation and assistance given by the Washington State Highway Department and the Washington State Patrol are also appreciated.

REFERENCES

1. Flynn, H., and Kyropoulos, P., "Truck Aerodynamics." SAE Preprint 284A (Jan. 1961).
2. Sawhill, R. B., and Firey, J. C., "Predicting Fuel Consumption and Travel Time of Motor Transport Vehicles." HRB Bull. 334, pp. 27-46 (1962).
3. Saal, C., "Public Roads." (May 1942).
4. Sawhill, R. B., and Firey, J. C., "Motor Transport Fuel Consumption Rates and Travel Time." HRB Bull. 276, pp. 35-91 (1960).
5. Beck, W., "An Investigation of Motor Vehicle Parasitic Power Loss by Road Test." M. S. in M. E. Thesis, Univ. of Washington (1962).
6. Firey, J. C., "Effects of the Tire Pressure and Temperature on the Rolling Resistance of Trucks." Univ. of Washington, Traffic and Operations Series, Transportation Res. Group, Res. Rep. 4 (June 1962).
7. Stiehler, R. D., Steel, M. N., Richey, G. G., Mandel, J., and Hobbs, R. H.,

"Power Loss and Operating Temperature of Tires." Jour. Res. NBS, 64C: 1 (Jan. -Mar. 1960).
8. Hoerner, S. F., "Fluid Dynamic Drag." Midland Park, N. J., pp. 12-1—12-9 (1958).
9. Firey, J. C., and Peterson, E. W., "An Analysis of Speed Changes for Large Transport Trucks." HRB Bull. 334, pp. 1-26 (1962).
10. "Truck Ability Prediction Procedure." SAE Rec. Prac. Rep. TR-82.

A Study of the Feasibility of Using Roadside Radio Communications for Traffic Control and Driver Information

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A new method of roadside communication with the driver incorporates car mounted receivers and roadside transmitter installations. The primary aims of the research project were to measure the effectiveness of this as a traffic control and driver information device, to judge its acceptability by the driving public, and to arrive at a preliminary cost for the implementation of such a system.

Half the vehicles selected were used as test group and half as control group for each of three experiments in which the test group drivers receivedradio information on accidents and typical maintenance activities. In the route information experiment no control group was used. Both test and control group drivers received similar information from signs and other signals where they were employed. Data on traffic flow were collected using time-lapse motion picture photography at locations just beyond the points of information reception. In addition, test vehicle operators were interviewed at the end of 10 mi of the test section to determine their reaction to the radio communication.

Results of the experiments showed that radio communication is effective in controlling vehicle speed in hazardous areas. The difference in the lateral placement distribution between the test and control vehicles immediately prior to the hazardous areas was significant in some of the experiments. The route information given in one of the experiments was considered by drivers to be helpful and a possible future use of the radio system. Interview data revealed that the motorists considered radio communication useful and that it should be used in a variety of situations to provide a variety of information. Driver acceptance was indicated by the amount drivers were willing to pay for a radio receiver capable of receiving roadside communication, based on the assumption that this receiver would be constructed as an integral part of the usual car radio and would operate if the car radio was on or off.

• THE PURPOSE of this research was to investigate the feasibility of roadside radio communications as a device to control traffic and inform motorists. Measurements were made by means of the behavior of the test vehicle in the traffic stream, and of the test vehicle operator's answers to a public opinion-type questionnaire. This work was

Paper sponsored by Special Committee on Electronic Research in the Highway Field.

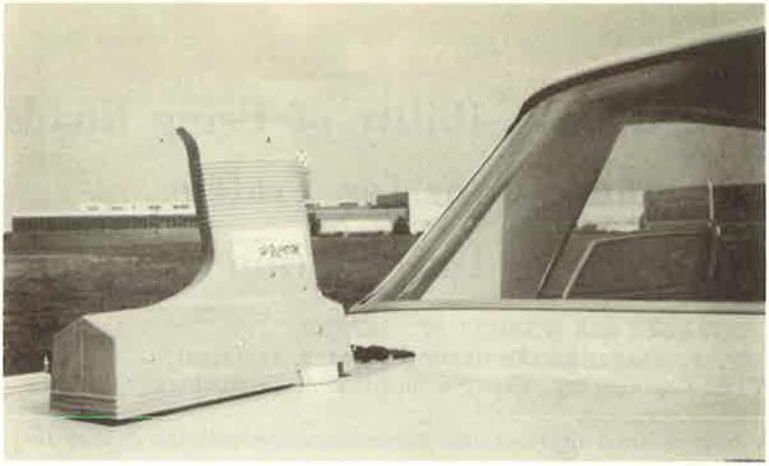

Figure 1. Radio receiver unit.

sponsored by the Bureau of Public Roads under a contract with the Engineering Experiment Station of the Georgia Institute of Technology.

The first phase of this research program was designed to measure the effectiveness of roadside radio communications as a traffic control and driver information device, to gage the drivers' acceptance of this type of roadside communications, and to obtain enough information to enable a preliminary cost estimate to be made for such a system. To accomplish these objectives a series of relatively simple but important experiments were designed and conducted on the Kentucky Toll Road from Shepherdsville to Louisville, Ky., in July and August 1963.

TEST EQUIPMENT

The radio equipment used, Delco Radio Hy-Com, is a system designed to provide communications from the roadside to the driver. It consists of a car mounted receiver and a roadside transmitter installation.

The receiver system has two components, a receiver and a speaker. The receiving equipment (Fig. 1) is incased in a fiberglass and plastic case. On the bottom of the case are three circular magnets, each covered with a phenolic disk. The receiver is mounted on the rear deck lid of a typical automobile and the rubber-coated safety hook is placed in the crack between the trunk lid and the body of the automobile. The magnets and safety hook provide a secure method of attaching the receiver for most automobiles. Receivers were also taped to the top of buses or trucks, placed on the gas tanks or steps of trucks, or put in sport cars wherever room could be found.

The receiving unit is powered by four $1\frac{1}{2}$ v penlight batteries providing approximately 100 hr of continuous operation. A cable from the receiver housing to the speaker permits the speaker to be located on the interior of the automobile. A spring clip on the rear of the speaker housing enables it to be mounted on the sunvisor or other body trim.

A transmitter (Fig. 2) was positioned just off the shoulder of the highway. The associated antennas were positioned as shown in Figure 3. When a test vehicle approached from the south, the receiver mounted on it first encountered the magnetic field associated with the trigger antenna. This field was of a sufficient strength to turn the receiver on. A delay switch held the receiver in the "on" position until the test vehicle was in the area of influence of the information antenna's magnetic field, a 1,000-ft section along which the operator received the message previously inserted on the magnetic drum repeater in the transmitter. A more detailed description of the operation of the transmitter is given in the Appendix.

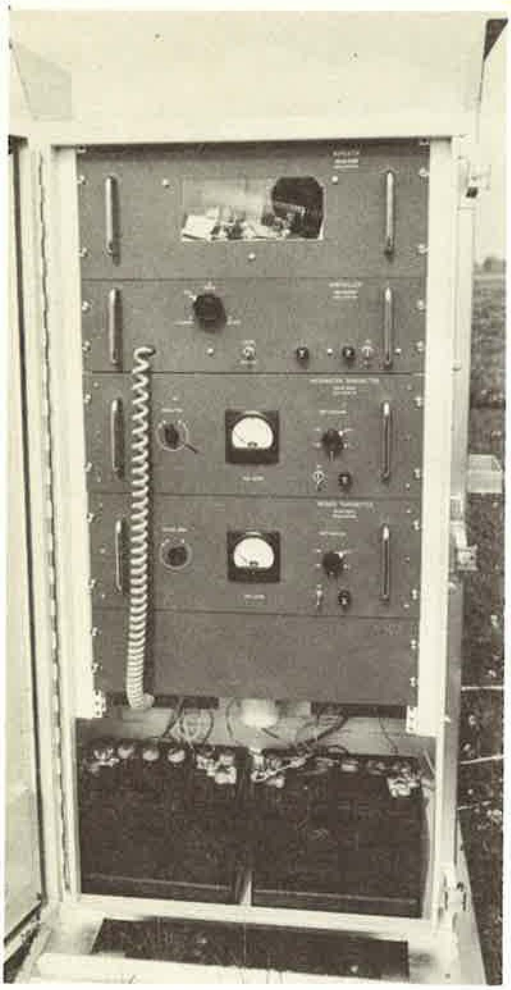

Figure 2. Inside view of transmitter cabinet.

DATA COLLECTION

Site Selection

Several conditions were required in the selection of a test site. It had to be a controlled access facility so that the driver could not leave the turnpike before he came to the interview area where the test radio could be recovered. Also the site had to offer good locations for time-lapse motion picture camera placement. Traffic volume had to be such that one could continue to draw a systematic sample all day without either too many or not enough vehicles to obtain reliable data. Proximity to Kokomo, Ind., was also important for convenience in equipment maintenance. A very important consideration was the willingness of the particular highway department to cooperate in the experiments. Based on these conditions, the Kentucky Toll Road was selected with the full cooperation of the Kentucky Highway Department.

Study Site

The study area (Figs. 4, 5) was located on Interstate 65, Kentucky Toll Road, between the Shepherdsville, Ky., Toll Plaza and the Fern Valley Exit, just south of Louisville. This portion of the road is a divided 4-lane facility with 12-ft concrete lanes. The right-hand shoulders are 10 ft wide and are paved with asphalt. The inside shoulder is approximately 4 ft wide and is also paved with asphalt. The median is 16 ft wide and is of turf-type construction, raised approximately 1 ft. The horizontal and vertical alignments are consistent with the 70-mph speed limit.

The 10-mi test section had an average daily traffic of approximately 8,000 vehicles in the summer months and a truck composition of approximately 20 percent.

The experiments were conducted only when the pavement was dry and no rain was imminent. All experiments were conducted on weekdays between 8 AM and 5 PM.

Filming Technique

Three cameras, located at bridges No. 1, 2 and 3 (Fig. 5), enabled collection of data by time-lapse motion picture photography. Each camera exposed several rolls of film at 9-min intervals randomly throughout the test days. Filming was scheduled so that camera No. 2 started 2 min after camera No. 1, and camera No. 3 started 2 min after camera No. 2. Thus it was theoretically possible, to follow a vehicle through all three camera locations. Should any situation develop that would affect traffic flow or otherwise impair the experiment, the camera operators were advised of the situation by walkie-talkie and given a revised schedule.

To facilitate film analysis, a grid system for each camera was painted on the highway shoulders perpendicular to the centerline at 40-ft intervals for a distance of 200 ft (Fig. 6).

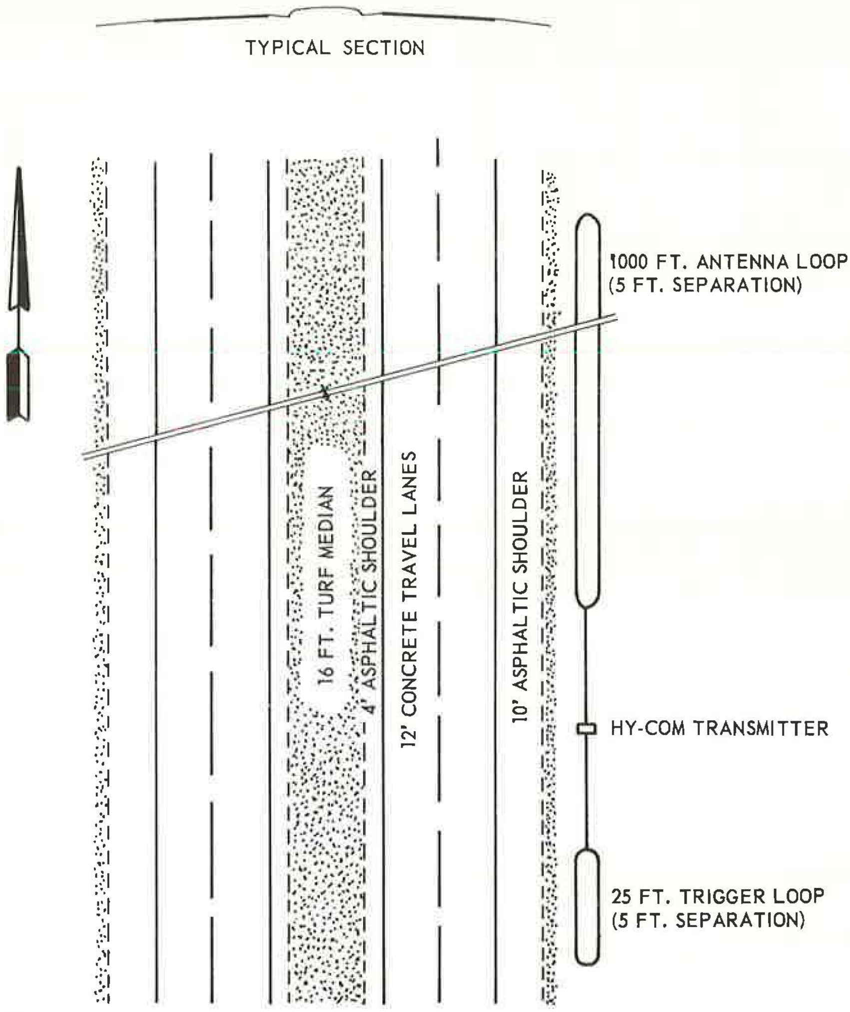

Figure 3. Typical transmitter and loop installation.

Experiment Design

To consider the psychological factor that the behavior of persons directly involved in the experiment would differ from that of nonparticipating persons, a control group of vehicles was established. This control group received essentially the same information as the test vehicles but was not given a radio receiver.

The selection of test and control vehicles was made by a systematic sampling with every other selected vehicle designated as a control vehicle or a test vehicle. The selection of the vehicles was done by a Kentucky State Police Trooper who directed every fourth northbound vehicle passing through the Shepherdsville Toll Plaza to turn into the unused section of the inside lane where the vehicle was processed.

Each driver was given a short explanation of the purpose of the project and then asked for his cooperation. If the occupants of the vehicle elected not to cooperate, the project personnel simply asked for a refusal reason and waved him on, whereupon a vehicle other than the every fourth vehicle normally selected was asked to participate in the experiment.

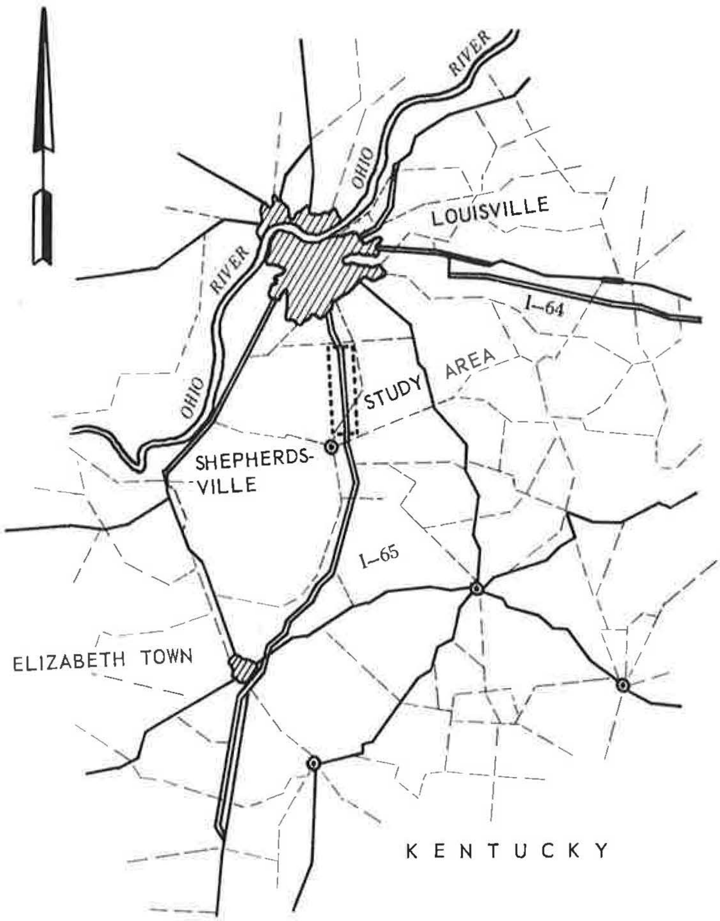

Figure 4. Location of study area, approximate scale: 1 in. = 12 mi.

If a vehicle designated as a control vehicle accepted the invitation to participate, an identifying bumper sticker was placed on the front bumper. The sticker was chartreuse so that it would be noticeable in color motion picture photography. The placement of the sticker identified the driver as being male or female. A sticker placed on the right side indicated a female driver; on the left side, a male driver. The driver of the control vehicle was then given a brochure explaining the project to read when time was available.

Test vehicles were similarly coded with bright red bumper stickers, positioned so as to identify the sex of the driver. In addition, the vehicle was outfitted with a radio unit. The motorist then drove through the test section where he was given several messages and asked to stop for an interview at the end of the test section. There he was given the information brochure.

At the end of the test section, the motorist pulled over at a well-marked area where he was subjected to an interview which took 3 to 5 min. The radio unit and identifying sticker were removed and the driver was allowed to continue after the interview was completed.

Questions were designed to evaluate the driver's acceptance of this form of communication based on his short exposure to it. Other uses were suggested and drivers were asked their opinion on its usefulness. Several questions were designed to measure the effectiveness of the radio communications. The choice of alternatives within the questions was varied from interview to interview so no position bias in the replies would be

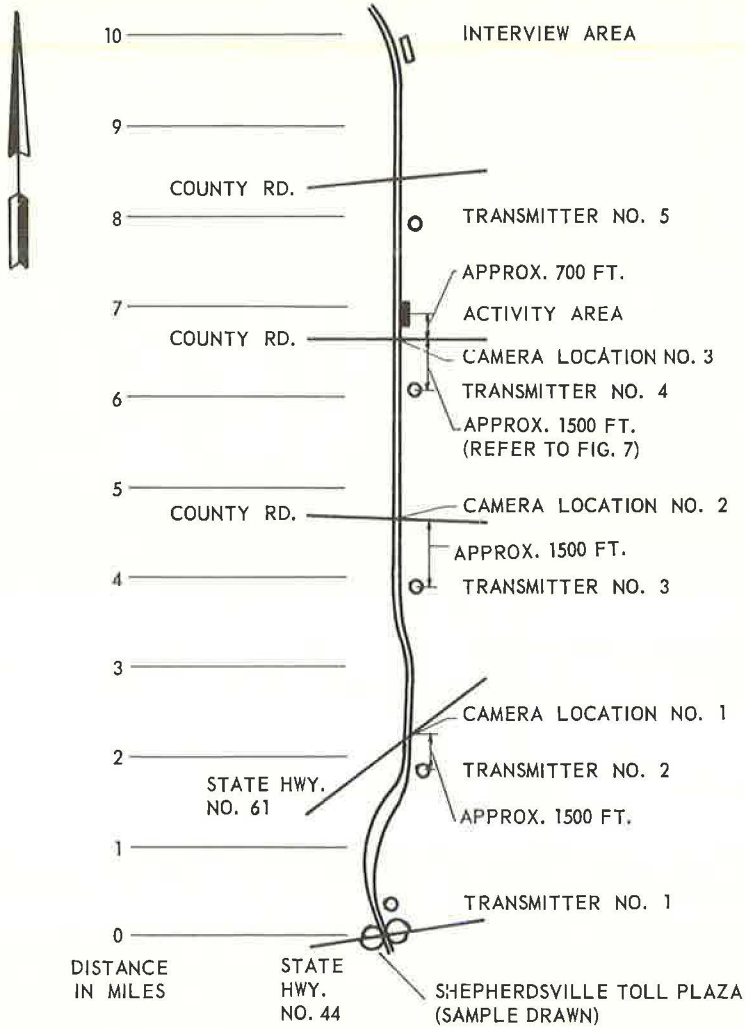

Figure 5. Study area.

created. The interviews were tabulated with respect to experiment, destination and sex of driver, and type of vehicle.

Four experiments were conducted, each dealing with a different road situation. Each experiment was repeated twice, once on each of two randomly selected days.

Experiment 1.—Experiment 1, dealing with an accident scene, was conducted on July 23 and August 1, 1963. To simulate actual conditions, a tow truck, wrecked vehicle and a State Police cruiser were positioned in the median lane of the toll road. A State Police Officer was available to direct traffic through the area should any congestion develop. The only other warning devices used were the red flashing lights on the police vehicle and the wrecker. Figure 7a shows the accident scene.

The messages given to the test vehicles were:

Transmitter No. 1—"This is Hy-Com Radio Communications. Several messages describing actual roadway conditions will be given in the next 10 mi."...repeated once in 10 sec.

Transmitter No. 2—Not used.

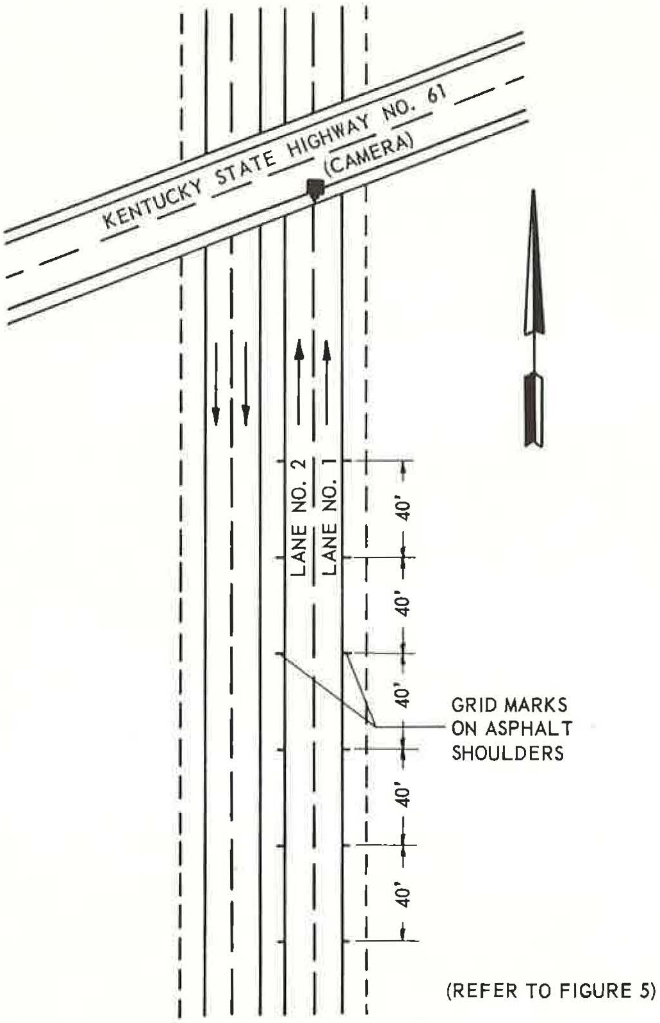

Figure 6. Typical camera and grid layout, scale: 1 in. = 60 ft.

Transmitter No. 3—"Accident ahead 2 mi."...repeated 4 times in 10 sec.
Transmitter No. 4—"Accident ahead, use right lane."...repeated 3 times in 10 sec.
Transmitter No. 5—"Drop off test radio 1 mi."...repeated 4 times in 10 sec.

Experiment 2.—Experiment 2 was conducted on July 24 and 26, 1963. In this experiment a normal maintenance activity, grass cutting, was chosen and the State Highway Department had a tractor mower working on the median. No lane blockage was necessary. A typical operation may be seen in Figure 7b. No warning signs were employed. The messages given to the test vehicles were:

Transmitter No. 1—"Messages concerning actual roadway conditions will be given in the next 10 mi."...repeated twice in 10 sec.
Transmitter No. 2—Not used.
Transmitter No. 3—"Grass cutting 2 mi ahead."...repeated 4 times in 10 sec.
Transmitter No. 4—"Grass cutting, slow to 40."...repeated 3 times in 10 sec.
Transmitter No. 5—"Drop off test radio 1 mi."...repeated 3 times in 10 sec.

Experiment 3.—Experiment 3 was conducted on July 25 and 30, 1963. In this test a typical maintenance activity, patching the shoulder, was simulated in the activity area.

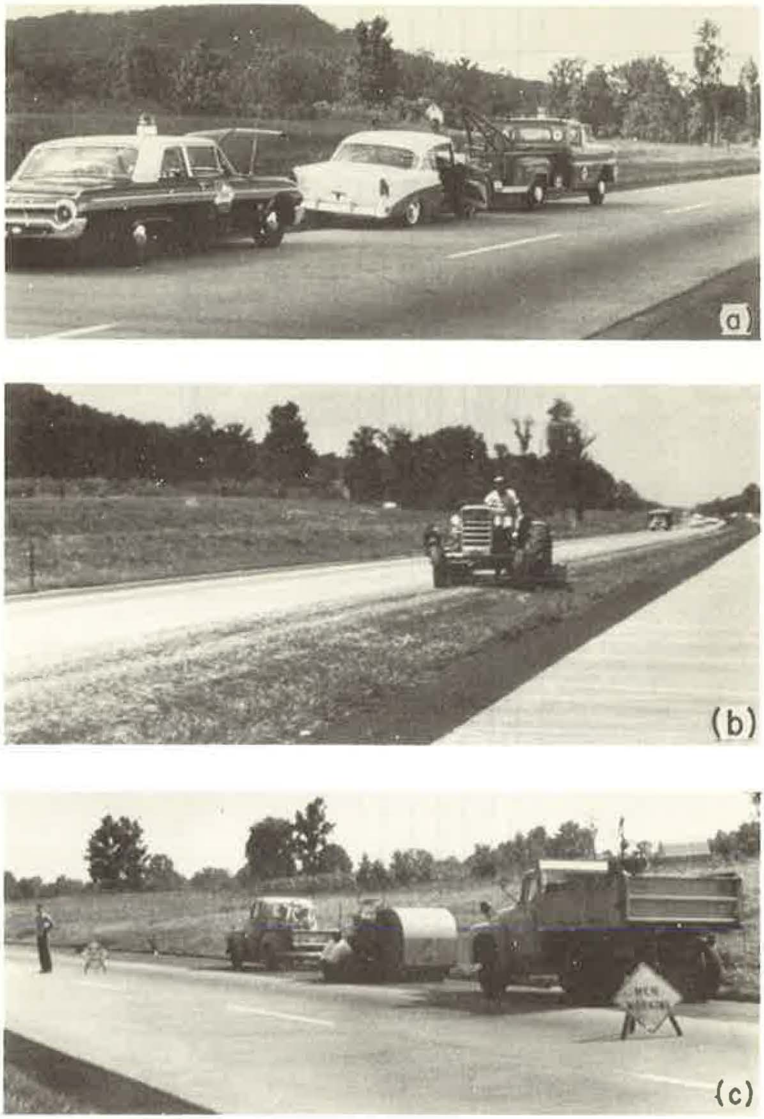

Figure 7. Activity area: (a) accident scene—Experiment 1; (b) mowing scene—Experiment 2; and (c) patching scene—Experiment 3.

The Kentucky State Highway Department supplied several trucks and the necessary personnel to realistically execute the work (Fig. 7). Half lane 1 was blocked in the activity area. A flagman and typical maintenance signing, visible to the approaching drivers while they were still in the grid of the camera at location 3, was used.

The messages given to the test vehicles were:

Transmitter No. 1—"Messages describing actual roadside conditions will be given in the next 10 mi."...repeated twice in 10 sec.
Transmitter No. 2—Not used.
Transmitter No. 3—"Men working 2 mi ahead."...repeated 4 times in 10 sec.
Transmitter No. 4—"Men working, slow to 40."...repeated 3 times in 10 sec.
Transmitter No. 5—"Drop off test radio 1 mi."...repeated 3 times in 10 sec.

Cameras were located in the same positions as in the other experiments.

Experiment 4.—This experiment, conducted on July 31 and Aug. 2, 1963, provided only route information. No roadway activity was described and, consequently, there was no reason for test vehicles to perform differently from the control vehicles in the traffic stream. For this reason no film data were taken and it was not necessary to use control vehicles. The messages given to the test vehicles were:

Transmitter No. 1—"Messages concerning route information will be given in the next 10 mi."...repeated twice in 10 sec.
Transmitter No. 2—"Louisville, home of Kentucky Derby, 15 mi."...repeated 3 times in 10 sec.
Transmitter No. 3—"Cincinnati, 135 mi on I-65."...repeated 3 times in 10 sec on recording drum.
Transmitter No. 4—"Indianapolis 125 mi on US 42."...repeated 3 times in 10 sec.
Transmitter No. 5—"Drop off test radio 1 mi."...repeated 3 times in 10 sec.

Coordination of Experiment

Coordination of the experimental activities over the 10-mi test section was achieved by using Citizens' Band radio equipment. Each camera operator was equipped with a 1-w walkie-talkie unit. In addition, several 5-w units were used in automobiles. One unit was stationed at the interview area and the others patrolled the test section checking the operation of the transmitters and the camera operators. The radio units gave the project a unifying element that could not have been otherwise attained.

Data Reduction

The film obtained in the experiments analyzed by means of a projector which allowed a frame by frame analysis of each roll of film. The film was projected onto a screen on which a grid, made to fit the grid painted on the pavement shoulders, was superimposed. Using the grid technique, it was possible to analyze the film for vehicle speeds, volume and lateral placement. Also available from the film analysis was the vehicle type and the sex of the driver which was determined from the placement of the colored bumper stickers.

ANALYSIS OF DATA

In the design of the experiment consideration was given to the fact that equal representation of all elements would not be obtained. This means that when analyzing data for differences between male and female drivers, local and nonlocal drivers, or passenger vehicles and other types of vehicles, there would not be an equal number of observations for each group. There were six vehicle types considered: passenger, panel or pickup, station wagon, single axle truck, multiple axle truck, and bus. Thus in the collection and subsequent classification of the data it was expected that some comparative analyses would not be possible.

The data collected in the film analysis were carefully considered in view of these considerations. These limitations made it necessary to pool over-all vehicle types and the sex of the driver in the statistical computations. The average speed of the test vehicles and control vehicles in Experiments 1, 2 and 3 at the three camera locations for each of the test days is presented in Table 1.

Analysis of Variance

After tabulation, the data were examined by analysis of variance methods. Because all main effects were fixed, the three factor interaction term should logically be used as the error term against which the initial tests would be made. However, this error term had only two degrees of freedom, which rendered the "F" tests rather ineffective. Therefore, a new error term, the "within cell mean error variance," was calculated, which had a greater number of degrees of freedom.

To use the analysis of variance techniques it was necessary to formulate a mathematical model in terms of the unknown parameters and the associated random variable.

TABLE 1

AVERAGE SPEED OF VEHICLES

Experiment	Day	Vehicle Type	Avg. Speed (mph) Camera No. 1	Camera No. 2	Camera No. 3
1	1	Test	58.05	59.57	42.26
		Control	57.79	61.58	50.89
	2	Test	56.93	55.45	46.69
		Control	59.69	59.27	51.64
2	1	Test	60.16	58.53	52.69
		Control	60.06	61.00	60.66
	2	Test	61.25	59.08	52.43
		Control	61.12	61.94	62.02
3	1	Test	59.08	57.85	41.69
		Control	59.73	61.09	52.94
	2	Test	59.47	57.32	41.79
		Control	59.84	61.56	51.85

TABLE 2

ANALYSIS OF VARIANCE FOR SPEED[a]

Variable	Level of Significance 5 %	10 %	20 %
Camera Location	Significant	Significant	Significant
Day	Nonsignificant	Nonsignificant	Significant
Vehicle type	Significant	Significant	Significant
Location-day	Significant	Significant	Significant
Day-vehicle	Nonsignificant	Nonsignificant	Significant
Location-vehicle	Significant	Significant	Significant
Location-day-vehicle	Nonsignificant	Nonsignificant	Nonsignificant

[a]Includes all drivers and all vehicle classes in Experiment 1.

TABLE 3

RANK ORDER OF TEST AND CONTROL VEHICLE SPEEDS AT DIFFERENT CAMERA LOCATIONS
(Experiment 3)

Camera Location	Speed Highest	Lowest
1	No significant difference	
2	Control	Test
3	Control	Test

TABLE 4

ANALYSIS OF VARIANCE FOR SPEED[a]

Variable	Level of Significance 5 %	10 %	20 %
Camera location	Significant	Significant	Significant
Day	Nonsignificant	Nonsignificant	Nonsignificant
Vehicle type	Significant	Significant	Significant
Location-day	Nonsignificant	Nonsignificant	Nonsignificant
Day-vehicle	Nonsignificant	Nonsignificant	Nonsignificant
Location-vehicle	Significant	Significant	Significant
Location-day-vehicle	Nonsignificant	Nonsignificant	Nonsignificant

[a]Includes all drivers and all vehicle classes in Experiment 2.

The quantitative physical characteristic (dependent variable) of interest was speed and the independent variables were day of experiment, test or control vehicle, and location of camera. The 10 percent level of significance was used for testing the variables. Duncan's "Multiple Range and Multiple F Tests" were used to investigate significant differences.

Experiment 1.—Results (Table 2) indicate that of the main effects, the location of the camera and the type of vehicle were significant. The location of the cameras with respect to the transmitter locations may be seen in Figure 5. Of the interactions, the camera location-vehicle and the camera location-day were significant.

Film analysis showed that the mean speeds of the test and control vehicles were not significantly different at camera locations 1 and 2, but there was a significant difference between the speed at camera location 3 and those at the other locations. Table 3 gives the rank order of speeds of the test and control vehicles observed at the different camera locations. There were significant differences between the speeds of the test and control vehicles at camera locations 2 and 3, but not at location 1. Therefore, up to the first camera location the presence of the test radio did not affect the normal operating speed of the test vehicle operator. By the time the test vehicle operator was in the range of camera location 2, he had received a message informing him of an accident 2 mi ahead. At this point his speed was significantly different from that of the control vehicle operator who heard no message. At camera location 3, prior to which the test vehicle operator had received the message, "Accident ahead, use right lane," the difference was again significant.

Experiment 2.—Table 4 gives the results of the analysis of variance for Experiment 2. Of the main effects, location of camera and vehicle type are significant, as well as the interaction of these two effects.

The control vehicle mean speed was not significantly different at any of the camera locations, but the test vehicle mean speed was lower at camera location 3 than at the other two camera locations. Table 5 gives the rank order of test and control vehicle speeds at the three camera locations. There exists no significant difference at camera location 1. However, at locations 2 and 3, there is a significant difference between the

TABLE 5

RANK ORDER OF TEST AND CONTROL VEHICLE SPEEDS AT DIFFERENT CAMERA LOCATIONS (Experiment 2)

Camera Location	Speed	
	Highest	Lowest
1	No significant difference	
2	Control	Test
3	Control	Test

TABLE 6

ANALYSIS OF VARIANCE FOR SPEED[a]

Variable	Level of Significance		
	5 %	10 %	20 %
Camera location	Significant	Significant	Significant
Day	Nonsignificant	Nonsignificant	Nonsignificant
Vehicle type	Significant	Significant	Significant
Location-day	Nonsignificant	Nonsignificant	Nonsignificant
Day-vehicle	Nonsignificant	Nonsignificant	Nonsignificant
Location-vehicle	Significant	Significant	Significant
Location-day-vehicle	Nonsignificant	Nonsignificant	Nonsignificant

[a]Includes all drivers and all vehicle classes in Experiment 3.

TABLE 7

RANK ORDER OF TEST AND CONTROL VEHICLE MEAN SPEED AT DIFFERENT CAMERA LOCATIONS (Experiment 3)

Camera Location	Speed	
	Highest	Lowest
1	No significant difference	
2	Control	Test
3	Control	Test

TABLE 8

SIGNIFICANT DIFFERENCES IN LATERAL PLACEMENT DISTRIBUTION OF TEST AND CONTROL VEHICLES

Source of Variation			Level of Significance	
Experiment	Location	Lane	10 %	20 %
1	1	1	Nonsignificant	Nonsignificant
	2	1	Significant	Significant
	3	1	Significant	Significant
2	1	1	Significant[a]	Significant[a]
	2	1	Nonsignificant	Nonsignificant
	3	1	Nonsignificant	Nonsignificant
3	1	1	Nonsignificant	Nonsignificant
	2	1	Nonsignificant	Nonsignificant
	3	2	Nonsignificant	Significant[a]

[a]Favors group control.

test and control vehicle speeds which can be attributed to the messages concerning the grass cutting operation received by the test vehicle operators prior to these locations.

Experiment 3.—The results of the analysis of variance for experiment 3, given in Table 6, indicate that only the main effects of camera location and vehicle type and their interaction were significant. These effects were significant even at the 5 percent level.

The mean speeds of both test and control vehicles were lowest at camera location 3, whereas there was little difference in speeds at locations 1 and 2. Table 7 gives the rank order of test and control vehicle speeds observed at the three camera locations. Only at location 1 is there no significant difference between test and control vehicle speeds, indicating that the presence of the test radio did not affect the speed of the test vehicles. However, the messages received by the test vehicle operators prior to camera locations 2 and 3 did contribute to the significant difference in speed between the test and control vehicles at these last two camera locations.

Lateral Placement of Vehicles

In addition to the speed data secured from the analysis of the films, information was obtained for the first three experiments concerning the lateral placement of the test and control vehicles at the three camera locations. To gather this information, the grid used in the speed analysis was modified slightly. After vehicle speed was measured, the position of the right front tire was recorded with respect to the right-hand edge of the pavement. These data were then analyzed using statistical techniques for significant differences in the test and control vehicle lateral placement distribution. Based on amount of data collected and the distribution of lateral placement observations a contingency test was used to analyze the data.

Experiment 1.—At camera location 3, approximately 1,000 ft before the accident scene, the most desirable position for a vehicle was in lane 1; that is, the right-hand wheel should be near the right shoulder. Table 8 indicates that the lateral placement distribution of test and control vehicles at camera location 1 is not significantly different. A similar analysis of the vehicles in lane 2 yielded the same result.

At camera location 2, the test vehicles had received the message, "Accident ahead, 2 mi," which the control vehicles did not receive and a significant difference in lateral placement between test and control vehicles existed. The test vehicles tended to occupy positions closer to the right-hand side of the road than did the control vehicles. Results were similar at camera location 3.

These results indicate that the messages received by the test group did affect their lateral placement at camera locations 2 and 3 to such a degree that placement differed significantly from that of the control group who did not receive the messages.

Experiment 2.—Table 8 also indicates a significant difference during Experiment 2 in the lateral placement distribution of test and control vehicles at location 1. At this location, the control vehicles occupied a position closer to the right-hand shoulder than did the test vehicles. At the other camera locations, no significant differences existed between test vehicles who had received messages concerning the grass cutting operation, and control vehicles who had received no messages.

The results of the contingency tests for Experiment 2 indicate that the messages did not have any consistent influence on the lateral placement distribution of test and control vehicles, especially in the activity area.

Experiment 3.—The maintenance activity in this experiment caused the right-hand lane to be blocked, therefore, in the analysis of lateral placement, the most favorable wheelpath in the vicinity of the activity area was as close as possible to the left-hand shoulder.

At camera location 1, before any messages were received, the analysis of lateral placement (Table 8) indicated that even at the 20 percent level in the contingency test there was no significant difference between test and control vehicles in lane 1 or lane 2. At camera location 2, although the test vehicles had received the message "Men working, 2 mi ahead," the analysis of lateral placement showed the distribution of test and control vehicles were not significantly different.

Prior to camera location 3 the test vehicles received the following message: "Men working, slow to 40," and a flagman and signs were employed in the activity area. Although the test vehicles received messages prior to zone of activity, their lateral placement distribution from the right-hand edge of the pavement was not significantly different from the control group distribution at the 10 percent level.

In the film analysis of the three experiments a record was kept of the test and control vehicle activity—passing, weaving, and lane changes—in the zone from the transmitter to the grid section of the camera field of view. Results indicated that at camera locations 1, 2, and 3 for all experiments the behavior of the test and control vehicles was essentially similar. At the first camera location, no difference was expected. The message received just before location 2 did not request any lane maneuvering and, consequently, no difference was expected. At the third camera location the number of lane changes by the test group was 42 out of 106 vehicles appearing in the film. For the control vehicles, 45 out of 108 made some lane change.

In analyzing these data, cognizance of the many factors that could have biased these data must be made; for example, during the experiments trucks broke down at critical points and exerted influence upon the traffic stream.

INTERVIEW DATA ANALYSIS

During the 8 days in which the tests were conducted, a total of 1,616 interviews were secured. The interview recording form is shown in Figure 8. Of the interviews taken however, 228 were invalidated for various reasons (Fig. 9). The most common reason for rejecting an interview was equipment malfunction. The receivers occasionally failed and so the test vehicles proceeded through the test section receiving some or none of the messages. This problem was especially evident with large trucks who many

INTERVIEW FORM
Project A-674

INTERVIEW NUMBER	DATE	INTERVIEWER			
	AGE:	MALE	(11)	FEMALE	(12)
TYPE OF VEHICLE (5-6)	UNDER 25	☐	1	☐	1
YEAR OF VEHICLE (7-8)	26 - 35	☐	2	☐	2
LICENCE NUMBER	36 - 45	☐	3	☐	3
STATE (9-10)	OVER 45	☐	4	☐	4

1. DESTINATION: LOCAL ☐ 1 OUT OF STATE ☐ 2 (13)
2. PARTICIPATE IN EXPERIMENT BEFORE: YES ☐ 1 NO ☐ 2 (14)
3. MESSAGES:
   - ADEQUATELY UNDERSTOOD .... ☐ 1
   - EASY TO UNDERSTAND .... ☐ 2 (15)
   - DIFFICULT TO UNDERSTAND .... ☐ 3

	YES	NO	NO OPINION	
4. MESSAGES DIFFICULT:				
RADIO NOT CLEAR	☐ 1	☐ 2	☐ 3	(16)
MESSAGE GARBLED	☐	☐	☐	(17)
MESSAGE REPEATED	☐	☐	☐	(18)
INSUFFICIENT INFORMATION	☐	☐	☐	(19)
LACK WARNING	☐	☐	☐	(20)
OTHER ( )	☐	☐	☐	(21)
5. MESSAGES HELP IN DRIVING:				
FELT SAFER	☐	☐	☐	(22)
INCREASED AWARENESS	☐	☐	☐	(23)
SMOOTHER OPERATION	☐	☐	☐	(24)
OTHER ( )	☐	☐	☐	(25)
6. MESSAGES NO HELP:				
NOT CLEAR	☐	☐	☐	(26)
ANNOYED BY RADIO	☐	☐	☐	(27)
NOT NEEDED	☐	☐	☐	(28)
OTHER ( )	☐	☐	☐	(29)

7. COMPARISON WITH SIGNS:
   - BETTER .... ☐ 1
   - SAME .... ☐ 2 (30)
   - WORSE .... ☐ 3

8. COMPARISON WITH NO SIGNS:
   - BETTER ..... ☐ 1
   - SAME .... ☐ 2 (31)
   - WORSE ...... ☐ 3

	YES	NO	NO OPINION	
9. FACILITATE DRIVING IN:				
NIGHT	☐ 1	☐ 2	☐ 3	(32)
FOG	☐	☐	☐	(33)
SNOW	☐	☐	☐	(34)
RAIN	☐	☐	☐	(35)
OTHER ( )	☐	☐	☐	(36)
10. SYSTEM USED FOR:				
COMPLEX INTERCHANGES	☐	☐	☐	(37)
SCENIC OR HISTORIC INFORMATION	☐	☐	☐	(38)
SERVICE AREA	☐	☐	☐	(39)
DETOUR	☐	☐	☐	(40)
TRAFFIC CONGESTION............	☐	☐	☐	(41)
OTHER ( )	☐	☐	☐	(42)
11. GENERAL USE IN MAJOR HIGHWAYS:	☐	☐	☐	(43)
12. CAR RADIO:	☐	☐	☐	(44)
13. ADDITIONAL COST: WOULD NOT PURCHASE (DOLLARS)	☐	☐	☐	(45)

14. REMARKS: ______________________________

Figure 8. Interview form.

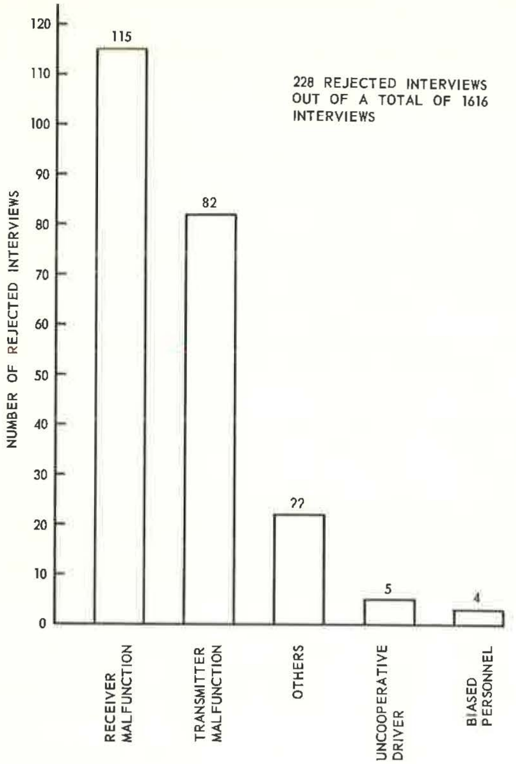

Figure 9. Number and causes of rejected interviews.

times heard only static. Another reason for interview rejection was malfunction of the transmitting equipment. In this case, the message repeater usually was the cause of the trouble. When a transmitter breakdown occurred, the test vehicles would receive an unclear message or no message at all at that location. The seriousness of the situation depended on which particular transmitter malfunctioned. The equipment malfunctions were a flaw in the experiments that allowed bias to enter into even the objective film analysis, because it was impossible to determine if the test car appearing on the film had received a message.

The traffic was predominantly composed of passenger vehicles, but the truck percentage on Tuesdays, Wednesdays and Thursdays represented approximately 30 percent of the volume. The male-female ratio of drivers in the test group was comparatively large. Of the 1,388 acceptable interviews taken, 1,136 were males and only 252 were females.

On the last day of testing, during the second part of the route information experiment, the interviewed drivers were asked the purpose of their trip. The results show a large percentage of recreational-type traffic, especially evident in the nonlocal destination traffic.

In response to the question concerning the ability of the messages to be understood by the test vehicle operators, the respondents generally indicated that the messages were well within the limits required for adequate comprehension, as over 95 percent of the drivers in every experiment thought the messages in general were of adequate quality. Of those drivers who had difficulty in understanding one or all of the messages, most of them indicated that the message was unintelligible or garbled because of malfunction of the receivers or transmitters. Other reasons given were insufficient number of repetitions of the messages and insufficient information contained in the messages.

The majority of the drivers indicated that the messages received did aid them in some way. Results showed that for Experiments 1, 2, and 3, the radio messages did make the drivers feel safer and more alert while at the same time contributing to a smoother operation of their vehicle. There were some drivers, however, who felt that the messages were of no help to them while driving over the test section. Message clarity, annoyance and the opinion that messages were not needed formed the majority of the dissensions.

The opinion of 90 percent of the respondents was that the joint use of radio and signs was better than just signs alone and also that the use of radio communications would be very advantageous in places where presently no signs are normally used. The latter situations arise principally at accident scenes and perhaps at some maintenance areas. Some drivers considered the radio communications an advantage in that messages could be kept up to date as contrasted to construction signs sometimes left in place after all hazards are removed.

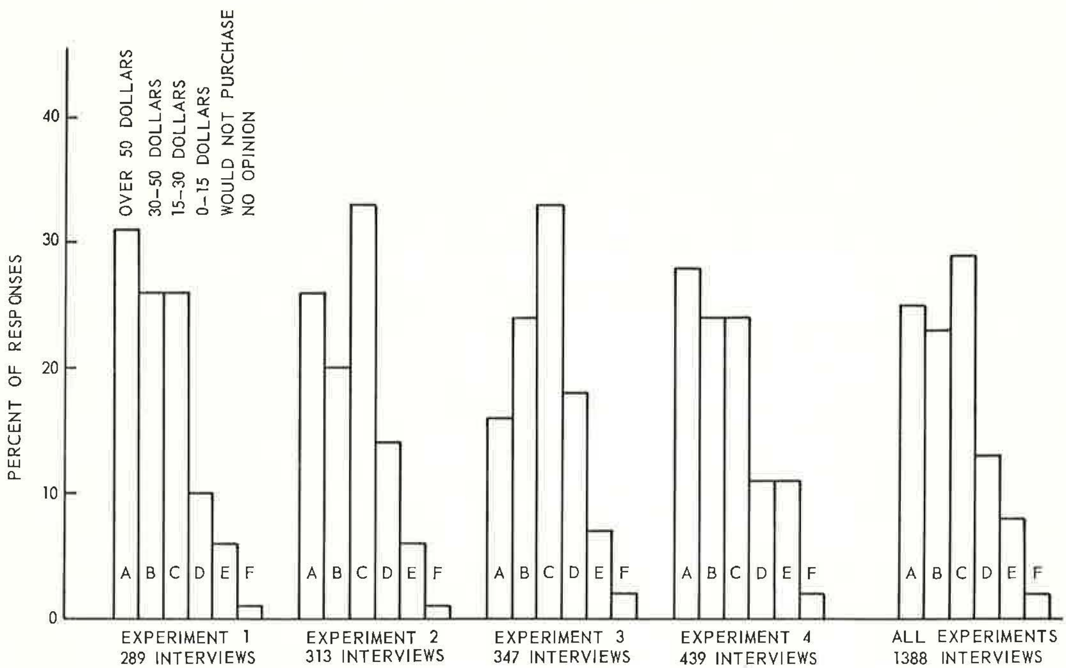

Figure 10. Amount drivers were willing to pay above cost of car radio for radio system comparable to test radio.

More than 95 percent of the drivers agreed that the use of roadside radio communications would be an advantage during inclement weather conditions. Other uses suggested by the drivers indicated a variety of applications. The possibility of using radio communications to inform drivers of scenic and historic locations, as well as service areas, was accepted very well and little difference was found between the responses given by local and nonlocal drivers. Approximately 70 percent of all drivers were of the opinion that the information on scenic and historic information would be useful, whereas more than 80 percent were receptive to the idea of receiving information about service areas. Perhaps some of the attractiveness of this information service could be attributed to the large percentage of recreational type traffic at this time of the year. More than 95 percent of the respondents thought that the system would be of help in the vicinity of complex interchanges. Similar reception was accorded the use of radio communications to warn of detours and traffic congested areas. The opinion that a radio communication system should be incorporated into all major highways in the nation was almost unanimous.

In order to properly formulate the question of willingness to pay, it was first determined if the vehicle was equipped with a radio. Approximately 15 percent of the vehicles interviewed did not have radios. Included in this figure are all the commercial trucks that ordinarily do not have radios.

To evaluate the driver acceptability of the radio communications system, the last question asked was how much more the driver would be willing to pay than the cost of his car radio for an installation of this radio equipment as part of a standard car radio, with the assumption that this installation would work automatically whether the radio was on or off and could be used on all of the major State highways.

The replies to this question were summarized in various groupings according to sex and destination of trip, that is, local or nonlocal. No significant differences were evident between the amount the male and female or the local and nonlocal drivers were willing to pay.

Figure 10 presents the cost results for each experiment summed over all drivers.

Greater than 75 percent of drivers in all four experiments were willing to invest from \$15 to \$30 in the system. Considering all four experiments together, approximately 48 percent were willing to pay more than \$30, whereas 25 percent were willing to pay more than \$50 for the system. The amounts over \$50 varied up to \$200, but for statistical analysis a mean value of \$75 was used. Approximately 8 percent of the drivers indicated that they would not purchase such an installation. It is interesting to note that in Experiments 1, 2, and 3, there were only about 6 percent who would not purchase the system, whereas in Experiment 4, the route information experiment, 11 percent indicated they were unwilling to purchase the system.

SUMMARY OF RESULTS

The results of the analysis of data collected in this study can be outlined as follows:

1. In the first three experiments, the analysis of the data using analysis of variance and multiple range tests indicated no significant differences in the speeds of the test and control vehicles at camera location 1. No information was transmitted immediately in advance of camera location 1.
2. In the first three experiments, a significant difference between speeds of test and control vehicles existed at camera location 2, where the transmitted message in advance of the camera location was advisory only, and at camera location 3, where the transmitted message was both advisory and directive.
3. In Experiment 1, significant differences in the lateral placement of test vs control vehicles occurred at camera locations 2 and 3 but not at 1.
4. In Experiment 2, significant differences in the lateral placement of test vs control vehicles occurred only at camera location 1 and favored the control group.
5. In Experiment 3, significant differences in the lateral placement of test vs control vehicles occurred only at camera location 3, but the significance was in the 20 percent level and favored the control vehicles.
6. Of the 1,616 interviews, 228 were considered biased and rejected. Of those rejected, equipment malfunction accounted for 197.
7. Ninety percent of the unbiased interviews indicated the broadcast messages were adequately or easily understood.
8. Most of the difficulty in understanding was caused by messages that were not clear or garbled in reception.
9. Messages helped in making the test vehicle operators feel safer, more alert, and contributed to a smoother operation of the vehicle through the test section.
10. Almost every interviewed driver thought that roadside radio communications in addition to standard signs were better than signs alone in most situations where it was necessary to give information or to caution drivers. The respondents also indicated that radio communications could be used effectively in situations where ordinarily no signing is used, such as in the vicinity of an accident.
11. It was almost the unanimous opinion of the interviewed drivers that roadside radio communication is a useful device in aiding the driver during inclement weather conditions.
12. More than 95 percent of the drivers favored the use of roadside radio communications in the vicinity of complex interchanges, traffic congested areas and detours. The use of the radio system to give information related to scenic and historic areas as well as service areas was acceptable to more than 70 percent of the drivers.
13. Most drivers would like to see this roadside radio communications system used on all major State highways.
14. Based on willingness-to-pay, most drivers indicated that the radio system had potentials. In response to the cost question, more than 25 percent of the operators were willing to pay in excess of \$50 for an installation; 48 percent indicated that they would be willing to invest more than \$30 for an installation; and only 8 percent of those vehicle operators interviewed indicated that they would not purchase such a system. In analyzing the willingness-to-pay for the various groupings of the data, it was found that no significant difference existed in the amounts that males and females or local and nonlocal drivers were willing to pay.

CONCLUSIONS

An evaluation of data developed the following conclusions:

1. The speed of a test vehicle was not significantly affected by the presence of the test radio equipment mounted on the vehicle. This is evident from the fact that at camera location 1 in all three experiments, no significant difference was found between test and control vehicle speed.

2. The messages received by the test vehicle operators did have a significant effect on the speed of their vehicles when compared to that of control vehicles who did not receive the messages.

3. In general, the messages received by the test vehicle operators did not always cause them to operate their vehicles in a manner such that the lateral placement distribution of the test vehicles differed significantly from the lateral placement distribution of the control vehicles.

In Experiment 1, the radio messages had a significant influence on the lateral placement distribution of test and control vehicles at camera locations 2 and 3. The test vehicles gave a greater clearance to the accident than did the control vehicles. In the mowing experiment, the control vehicles were closer to the right-hand shoulder than the test vehicles at camera location 1, but at camera locations 2 and 3, no significant difference in lateral placement was observed. In Experiment 3, no difference existed at camera locations 1 and 2. At camera location 3, significance was encountered only at the 20 percent level and it indicated that the control vehicles were giving more lateral clearance to the maintenance operation than the test vehicles.

4. During personal interviews, the test group, in general, approved of the roadside radio communication system. They agreed that the system helped them while driving over the test section, that the system could give desirable and necessary information concerning a variety of conditions that exist on the highways, and that the radio system could supplement the signs in some cases and provide acceptable service in other cases where signs are not used. The radio system, even though in experimental stages of development, was not noticeably annoying to the driver.

5. Based on the results of the willingness-to-pay question, the driver acceptance for this system was considered good. Recognizing the limitations of the data collected, it may be concluded that if the roadside radio communications system did become a reality, and its performance was at least comparable to the equipment tested, at least half the motoring public with similar driving habits as those in the experiment would be willing to pay at least $30 for an installation.

RECOMMENDATIONS

Additional research should be conducted to investigate the effect of radio communications on repeat traffic in an urban area. Also, research must be conducted into the number of transmitters needed to provide effective radio communication service for a typical freeway, the type and characteristics of messages given, and most important, the possibility of a central control system for the transmitters. The message repeater should be modified to eliminate the mechanical and electrical noises associated with the magnetic drum repeater assembly. Research also might be conducted to investigate the influence of field strength and configuration on the operation of receivers of various locations in the field.

REFERENCES

1. Blankenship, A. B., "Consumer and Opinion Research." Harper and Brothers Pub., New York and London (1943).
2. Blankenship, A. B., "How to Conduct Consumer and Opinion Research." Harper and Brothers, Pub., New York and London (1946).
3. Covault, D. O., "Time-Lapse Movie Photography." Traffic Eng., 3:6 (March 1960).
4. Duncan, D. B., "Multiple Range and Multiple F Tests." Biometrics, 11:1-4 (1955).
5. Hanysz, E. A., "Highway Alert Radio." Res. Lab. Brochure. General Motors Corp., Warren, Mich.

6. Hanysz, E. A., Stevens, J. E., and Meduvsky, A., "Communication System for Highway Traffic Control." Electronics (Oct. 1960).
7. Hopkins, R. C, et al., "Long Range Research and Development Program for Individual Transportation Systems." Public Roads, 32:7 (April 1963).
8. "Hy-Com Highway Communicator." General Motors Corp., Delco Radio Div., Kokomo, Ind., Eng. Dept. Rep.
9. Keller, J. E., "The FCC and Radio Control of Traffic Control Devices." Traffic Eng. (May 1959).
10. Morrison, H. M., Welch, A. F., and Hanysz, E. A., "Automatic Highway and Driver Aid Developments." Presented at SAE Nat. Automobile Week, Detroit, Mich. (March 1960).
11. Olsen, P. L., "Road-Driver Communications—1980." Proc., Inst. of Traffic Eng. (1962).
12. Quinn, C. E., "New Voice System Alerts Drivers." Traffic Eng. (July 1959).
13. "Will Engineering Advances Reduce Our Traffic Deaths?" Amer. Eng. (Oct. 1959).

Appendix

OPERATION OF THE TRANSMITTER SYSTEM

The transmitting system consists of a transmitter cabinet and two loop antennas laid on the shoulder of the road. The transmitter cabinet (Fig. 2) is watertight and can be set up at any required point along the side of highway. The cabinet contains the message repeater and associated transmitting equipment and two 12-v storage batteries for a power supply.

The transmitter system is a single sideband, suppressed carrier, one-way communication link. Audio information to be transmitted is recorded on a magnetic drum in the repeater. The repeater records messages of any duration up to 10 sec and will automatically repeat them.

A handset, located in the transmitter cabinet, serves as a microphone to permit recording and as a receiver to allow verification of proper recorder operation. A message which the driver is intended to hear is inserted on the recording drum. With the control in the playback position, the recorded message is automatically inserted at the input of the information transmitter. At the end of the message, the repeater will reset itself and the message will be repeated. In the transmitter the 12.1 kc carrier is amplitude modulated. Suppression filters then remove the carrier and the lower sidebands and deliver the upper sidebands to the power output stage which energizes a loop antenna. The loop antenna establishes an inductive field which can be sensed by the receiver antenna as it passes the loop. To avoid the confusion of a southbound driver receiving a message intended for a northbound motorist, an additional trigger feature has been incorporated. This consists of a 12.1 kc trigger transmitter and its associated trigger antenna, a loop of plastic-coated 19-strand copper wire. When the induction field of the trigger transmitter is sensed by the receiver antenna, a trigger circuit in the receiver is activated which energizes the audio stages of the receiver. A time delay is designed into the system to hold the audio section in the "on" position to permit the automobile to pass the trigger loop and to reach the information loop. As the receiver enters the information loop, it senses the information signal and provides an audible message to the driver. With this system, a southbound driver would pass the information loop before he would pass the trigger loop. His receiver would be off and no audible message would be heard.

The information loop used in the experiments was 1,000 ft long and consisted of a loop of plastic-coated wire laid just off the shoulder of the turnpike. A distance of 5 ft separated the legs of the loop.

The trigger loop, made of similar wire, had seven turns of the wire in the loop with a similar separation. However, the trigger antenna loop was only about 25 ft long. The trigger loop was located before the transmitter, whereas the information loop was located after the transmitter. A sketch of the layout of the antennae is shown in Figure 3.

The Traffic Pacer System

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•MANY ATTEMPTS to solve modern traffic control problems have taken the form of high speed freeways, limited access highways, or multi-lane divided highways which eliminate intersections by grade separation. These forms of traffic control have been aimed at solving high volume traffic problems. According to recent statistics compiled by the Michigan State Highway Department, approximately 12 percent of the existing traffic volume operates on such roads today in Michigan. The bulk of vehicular traffic moves on conventional road networks with intersection conflicts controlled by conventional methods such as stop signs and traffic lights.

Other than the obvious safety advantages of the limited access highway, efficient traffic flow is of prime concern. Smooth efficient traffic flow should also be the goal for signalized streets and arteries. Disturbances in traffic flow have a significant effect down the line of cars on the road (1, 2, 3). If stops or large fluctuations in speed can be reduced, traffic flow should become more stable. The traffic flow will be affected by the concentration of the vehicles on the road (3, p. 139). Furthermore, the flow can be optimized by regulating speed at which traffic moves (4). On conventional urban and suburban street systems, the flow is limited by the performance of the intersections. Existing traffic control systems, such as the progressive or interconnected system, attempt to time successive intersection signals so that uninterrupted flow can be maintained at a particular speed. Although somewhat effective, this system does not give the driver an accurate indication of his temporal position in relation to the beginning or end of the green phase of the cycle, particularly where intersections are widely spaced. This poses problems for traffic merging into the system or for motorists who are stopped or delayed between signalized intersections.

A unique system of vehicular traffic control was first developed in Germany which not only incorporates accurate phasing of successive intersection signals, but provides continuous supplementary speed information for arriving during the green phase of the cycle (5). A modification, called the "Traffic Pacer System," was developed and installed on a suburban roadway and is evaluated in this paper.

HISTORY

One attempt to meet local traffic needs was initiated in 1954, when Wolfgang von Stein installed the first traffic funnel in Dusseldorf, Germany. Since then the popularity of the signals that comprise the German traffic funnel has grown such that today there are more than 200 of these novel traffic control devices throughout Europe. The principles of the traffic funnel were presented formally by Dr. von Stein at a symposium, held at the General Motors Research Laboratories in December 1959, on the theory of traffic flow (6). He theorized that three main improvements should be realized by installation of the speed and pre-signals of the traffic funnel:

1. A 2-car-per-lane-per-cycle increase in capacity,
2. An increase in safety, and
3. A decrease in stops.

The first United States traffic pacer installation was placed in operation on Mound Road, between 11 and 15 Mile Roads, in Macomb County, Mich., on July 31, 1961.

Paper sponsored by Special Committee on Electronic Research in the Highway Field.

OPERATION OF THE TRAFFIC PACER

Although the actual hardware used in the traffic pacer system is different from that used in the German counterpart, the basic control philosophy is the same. The object of the traffic pacer is to form compact groups of moving vehicles, timed to arrive at the intersection at the onset of the green phase. As the last car of the artery group of vehicles passes, a time gap in artery flow should be provided large enough to allow the cross traffic to pass. To accomplish this the traffic pacer system employs two extra control elements (Fig. 1) not used in the ordinary interconnected progressive traffic systems: speed signals and pre-intersection stop signals (pre-signals). The elements are, from front to back, a speed signal, two pre-signals with a speed signal mounted between them, and the normal intersection traffic signal. The distance between the speed signal and the intersection traffic signal is approximately 1,500 ft. The indicated speed on the speed signal varies according to the vehicle's time of passing the signal. Vehicles at the end of the group are told to travel faster than vehicles that have already passed the speed signal, resulting in the concentration of vehicles into a more compact group. In every instance, the maximum speed limit shown by the signals is the maximum speed limit of the highway.

Figure 2 is a typical time space diagram showing precisely how this is accomplished. The heavily shaded area between the two intersections indicates the segment of the time-speed plot which drivers should avoid if they want to arrive at the next intersection during the green-light phase. The slopes of the lines indicate the speed which drivers must maintain to keep within the lighter zone, thereby arriving at the next intersection during the green-light phase. The first speed signal has a cycle which changes from 25 to 30 to 40 mph. The pre-signal installation has a cycle which changes from amber

Figure 1. Speed signs and pre-signal.

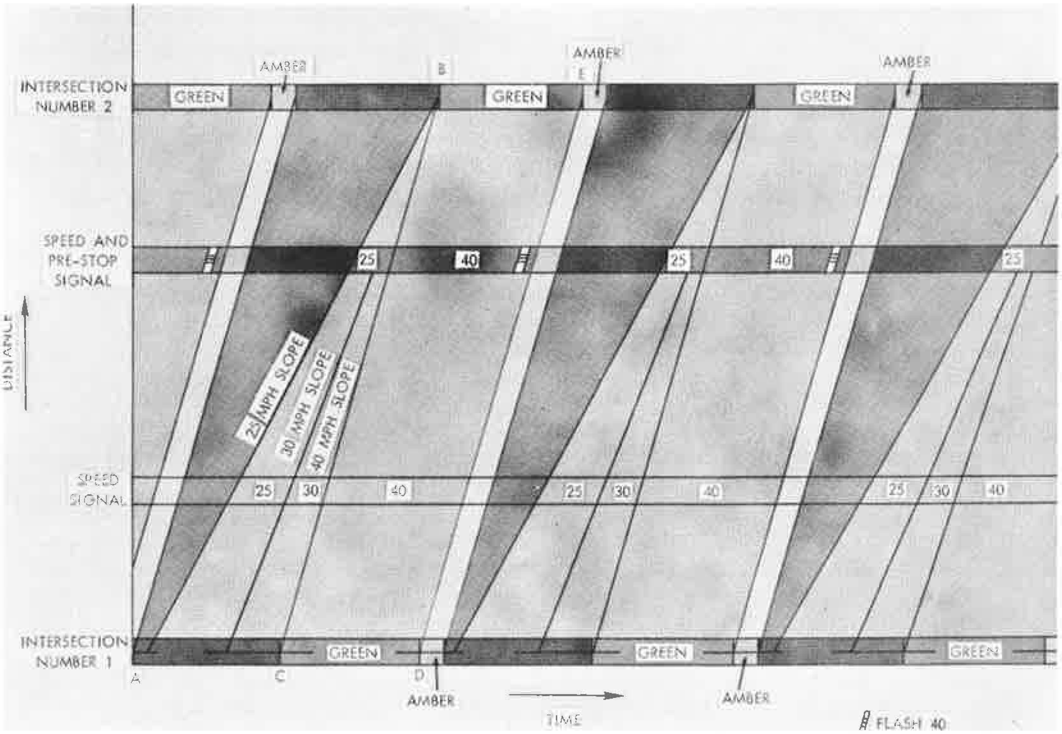

Figure 2. A simple time-space diagram.

to red, and from 25 to 40 mph during the green portion of the cycle. For example, a driver leaving intersection No. 1 at the beginning of the red light interval A will reach intersection No. 2 at the beginning of its green light interval B by maintaining a 25 mph speed. Similarly, a driver leaving intersection No. 1 halfway through the light cycle at C will reach intersection No. 2 at the start of its green light interval at B by maintaining a speed of 40 mph. Finally, the last vehicle of the group leaving intersection No. 1 on the amber signal D will reach intersection No. 2 at its amber signal at E by also maintaining a 40 mph speed.

The purpose of the pre-signal is to provide a moving start for vehicles approaching an intersection. Even rapidly accelerating vehicles, starting from a standstill, lose from 3 to 6 sec headway compared with vehicles that have been paced to the intersection signal. By releasing the traffic queued at the pre-signal early, the vehicles arrive at the intersection with a moving start just as the intersection light turns green. More specific details concerning equipment and its operation, as well as data acquisition, can be obtained from previous literature (5).

EXPERIMENTAL TESTING PROGRAM

The traffic pacer system was compared over a 12-wk testing period in 1961 with two commonly used traffic light installations:

1. "Past System," a simple non-interconnected system in which each intersection signal operates independently of the other intersection signals; and
2. "Progressive System," an interconnected system in which cars proceeding at the speed limit arrive at successive intersections during the green phase of the traffic light cycle.

In this comparison, the cycle length was 60 sec for all three systems, and the progression speed for the progressive and the pacer systems was 40 mph, with the exception of the City of Warren and the 1-mi section of Mound Road between 14 and 15 Mile Roads.

The following performance criteria were compared:

1. Average trip time, average speed, and average number of stops per trip throughout the 4-mi section of roadway;
2. Intersection capacity, the number of cars through an intersection per cycle;
3. Queue length, the number of cars waiting during the red-amber portion of the cycle; and
4. Public opinion.

During the summer of 1962, simplification of the traffic pacer system was examined. Progression speeds and cycle lengths were varied. A fixed single-bulb-type speed sign was compared with the variable multi-bulb configuration previously installed (Fig. 3, 7). Elimination of various components of the system were made in order to test individual and joint contributions to system performance. Table 1 summarizes the system examined, and a more detailed description is given in the Appendix. The same performance data were collected for the summer months of 1961 and 1962 with the exception of public opinion evaluations.

The 1-mi section of Mound Road between 14 and 15 Mile Roads could not be included in the 1962 testing programs due to heavy construction activity. In addition to these performance measures, mechanical volume counters were employed; intersection arrival time and speed were measured for different offset times between the pre-signal and intersection signal, and accident statistics were compiled for a 12-mo period and compared with pre-pacer accident data.

RESULTS

Summer 1961

During the initial 12-wk testing program, traffic volume counts were taken and the volume was found to be relatively constant for the 3-mo period. The three systems under test, (a) the past system, (b) the progressive system, and (c) the pacer system, appeared in the following order during the 12 weeks: a, c, b, a, b, c, a, c, b, a, b, c. Due to certain time and scheduling restrictions, a completely balanced experimental design could not be incorporated.

Average Trip Time and Speed. —Continuous records were made of the time taken to travel the 4-mi experimental test area. Average speeds were computed from the time records (Table 2). A record of the number of complete stops per trip was also recorded (Table 3). A trip is defined as traveling from 11 to 15 Mile Roads or 15 to 11 Mile Roads, a distance of 4 mi with 8 intersection signals.

The greater number of average stops in the evening is attributable to an increase in the traffic volume. No statistically significant differences in average trip time were found to exist between the pacer and progressive systems; however, both systems showed a significant reduction in trip time when compared with the past system. The frequency of stops indicated that the additional speed information provided by the pacer's system's speed signals permitted more stable speed control when compared with the progressive system.

TABLE 1

SUMMARY OF SYSTEMS UNDER TEST

Phase	Pre-Signals	Speed Signs	Funneling Speeds	Prog. Speed[a] (mph)	Cycle Length (sec)
A	Yes	Yes	Yes	40	60
B	Yes	Yes[b]	Yes	40	60
C	Yes	Yes[b]	Yes	35	70
D	Yes	Yes[b]	Yes	40	66
E	Yes	No	No	40	60
F	No	Yes[b]	Yes	40	60
G	Yes	Yes[b]	No	40	60
H	Yes	Yes[b]	Yes	45	55
K (prog.)	No	No	No	40	60
L (past)	No	No	No	-	60

[a]Speed in front of General Motors Technical Center.
[b]Single-bulb speed signs on southbound lane only.

TABLE 2

AVERAGE TRIP TIME AND SPEED

System	Morning (6:30-9:00 AM)		Evening (3:00-5:30 PM)	
	Avg. Trip Time (sec)	Avg. Speed (mph)	Avg. Trip Time (sec)	Avg. Speed (mph)
Pacer	401.6 ± 24.3[a]	36.6	428.4 ± 41.7	34.3
Progressive	398.4 ± 23.9	36.9	432.7 ± 48.3	33.9
Past	463.8 ± 41.2	31.6	482.9 ± 46.3	30.4

[a]Indicates one standard deviation.

Figure 3. Single-bulb-type speed sign: (a) front view, and (b) assembly.

TABLE 3

AVERAGE NUMBER OF STOPS PER TRIP

System	Avg. No. of Stops	
	Morning (6:30-9:00 AM)	Evening (3:00-5:30 PM)
Pacer	0.23	0.73
Progressive	0.41	1.17
Past	2.35	3.20

TABLE 4

AVERAGE NUMBER OF CARS QUEUED PER LANE PER CYCLE FOR ALL INTERSECTIONS

System	Avg. No. of Cars Queued	
	At Intersection	At Intersection and Pre-Signal
Pacer	0.40	1.79
Progressive	2.50	-
Past	3.14	-

Intersection Capacity. —In an attempt to ascertain any over-all improvement in intersection capacity, all the heavy volume intersections at rush periods were compared. Arbitrarily, the frequency of cycles in which 15 or more cars per lane got through during the green portion of the light cycle, as compared with the total number of cycles were recorded. The average of 25.4 percent of the light cycles allowed this passage under the pacer system as opposed to 19.4 and 17.1 percent under the progressive and past systems, respectively. The 49 percent improvement over the past system and the 31 percent over the progressive system were both statistically significant.

Queue Length. —A tabulation was made of the total number of cars queued per lane per cycle at each intersection for the three systems. The average queue per lane per cycle for all intersections was computed (Table 4). It was hypothesized that some relationship might exist between traffic density and queue length. Figure 4 illustrates this relationship for the three systems. A linear least squares fit, in the general form $y = mx + k$, was performed on the data for traffic densities up to and including 600 cars per lane per hour. Data beyond this point was fit visually.

Public Opinion. —Another comparison was made between the three experimental systems by means of a questionnaire. A total of 600 questionnaires were distributed each week to drivers entering or leaving the experimental test area from 11 to 15 Mile Roads. The drivers were asked to compare the system in operation that week with the system in operation the preceding week. An explanation of each of the three systems under test accompanied each questionnaire.

Results indicated that approximately 65 percent of the people felt that the pacer system was safer, faster, and caused fewer stops than did the progressive or past systems. Twenty-five percent of the responses were neutral and only 10 percent of the responses

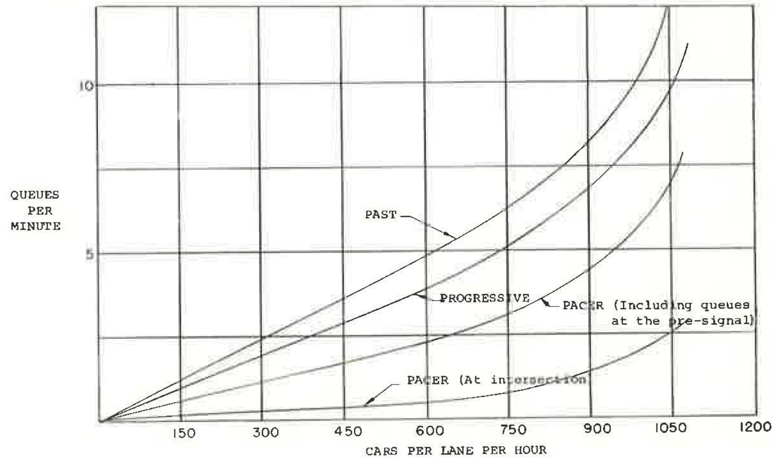

Figure 4. Queues vs traffic density.

rated the pacer system inferior to the other two systems on the three factors mentioned previously. Further results indicated that 75 percent of the respondents would like to see the pacer system installed on other roads. A more complete description of the summarized results of the initial 12-wk testing program can be obtained from the original paper (5).

Summer 1962

Average Trip Time, Speed and Stops. —Figures 5 to 12 show points of individual trip times plotted at the corresponding traffic volume for each of the systems tested. The trip time measurements were taken by a pace car moving in the traffic stream as an "average car"; that is, the car was driven at speeds which, in the opinion of the driver, were representative of the average speed of all the traffic in the stream. The volume count is based on 15-min subtotals, each subtotal being multiplied by 4 to obtain the hourly rate. Counts were taken for two lanes and the total was halved to give cars per lane per hour. For the purpose of comparison, the road section from 11 to 12 Mile Roads was used as a base for volume counts; i.e., a counter in the 11 to 12 northbound section gave the northbound volume data for the complete trip, and a counter in the 12 to 11 southbound section gave the southbound volume data. The starting time for each trip determined which 15-min volume count was to be used as the corresponding volume figure. All data given in Figures 5 to 12 were taken between 6:45 and 8:15 AM and between 3:45 and 5:15 PM; that is, during the heaviest traffic volume periods of the weekday.

Table 5 summarizes the results of Figures 5 to 12. The number of trips made in each direction was approximately the same and Table 5 presents the total of one-way trips, together with the average stops made by the pace car for a one-way trip. The average and theoretical trip times also indicate a one-way trip in either direction. It will be noted that the two-way progression and fixed geometry of the roadway result in fractional theoretical progression speeds. It is clear from Figures 5-12 that the system is operating below capacity at all times; i.e., at no time, for any of the systems tested, was the traffic density (based on 15-min counts) high enough to increase trip time or affect the number of stops.

Because the capacity of the roadway is not reached for the highest number of cars with the highest progression speed, the same average trip time with the higher progression speed (40 and 45 mph) must be due to some drivers dropping back a progression band. Examination of Figure 11 (45 mph progression speed) clearly shows the grouping of the trip times about three separate means, one at the progression speed and the others approximately one and two cycle lengths slower. The increases in trip times are most likely due to the fact that the road users habitually travel at 40 mph. A persistence with the higher progression speed over several months might eventually result in an average shorter trip time. A disadvantage of the higher progression speed and corresponding shorter cycle length is that they produce a loss of efficiency in cross-street traffic and main artery truck traffic.

Intersection Capacity and Queues. —With a 50 percent time split between red and green plus amber, the maximum capacity attainable at an intersection is half the capacity of the roadway itself. Throughout the summer of 1962, counts were made for the 12 Mile intersection, both northbound and southbound. The traffic counts were taken Monday through Friday during the hours of 6:45 to 8:15 AM and 3:45 to 5:15 PM. From both the 1961 and 1962 test programs, it was observed that traffic volume was the highest during these hours of the day. Human traffic counters recorded the total number of cars which went through the intersection during the available green time. Figure 13 shows the relationship between the average queues per minute and the number of cars through the intersection per lane per hour.

All the curves in Figure 13 are visual fits to the data. For all cycle lengths tested, the asymptotic capacity for the intersection seems to be somewhere between 1,050 and 1,200 cars per lane per hour. These curves compare favorably with the queue data in Figure 4. The points, plotted in Figure 13, from the 1961, 60 sec cycle data agree quite well with the curve fit to the 1962, 60 sec cycle data. In comparing various sys-

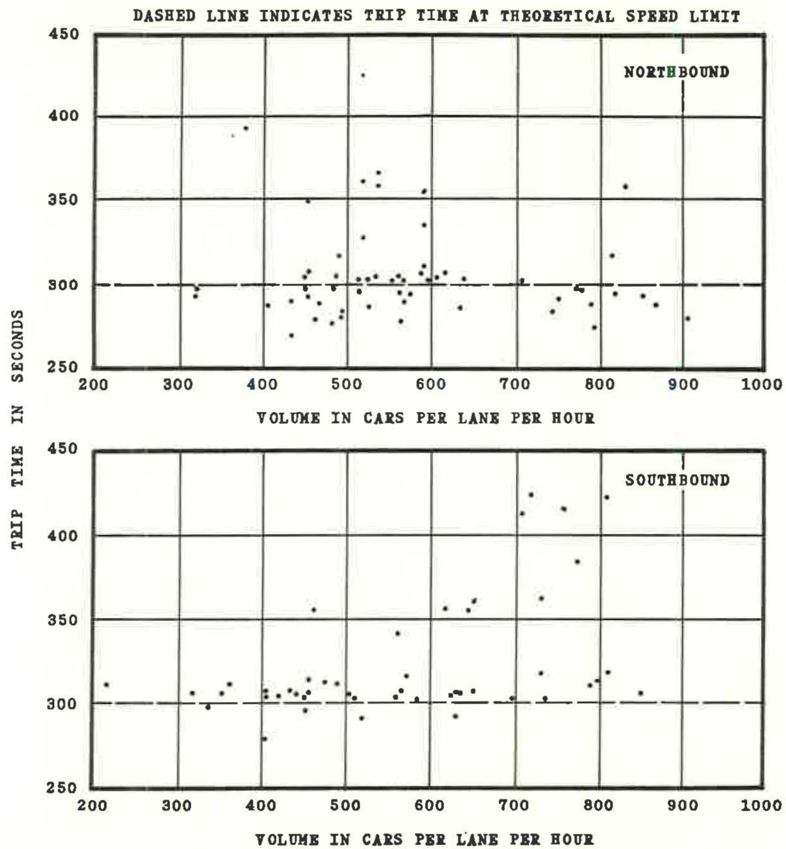

Figure 5. Systems A and B: 60 sec cycle; full pacer system; average stops, 0.62 per trip; average trip time, 318 sec per trip.

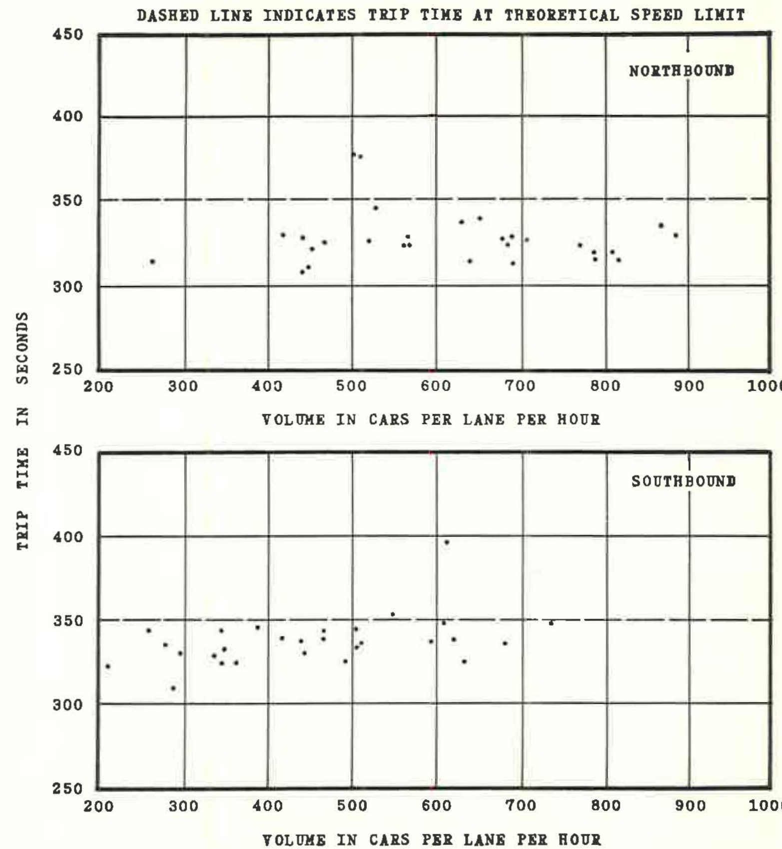

Figure 6. System C: 70 sec cycle; full pacer system; average stops, 0.62 per trip; average trip time, 337 sec per trip.

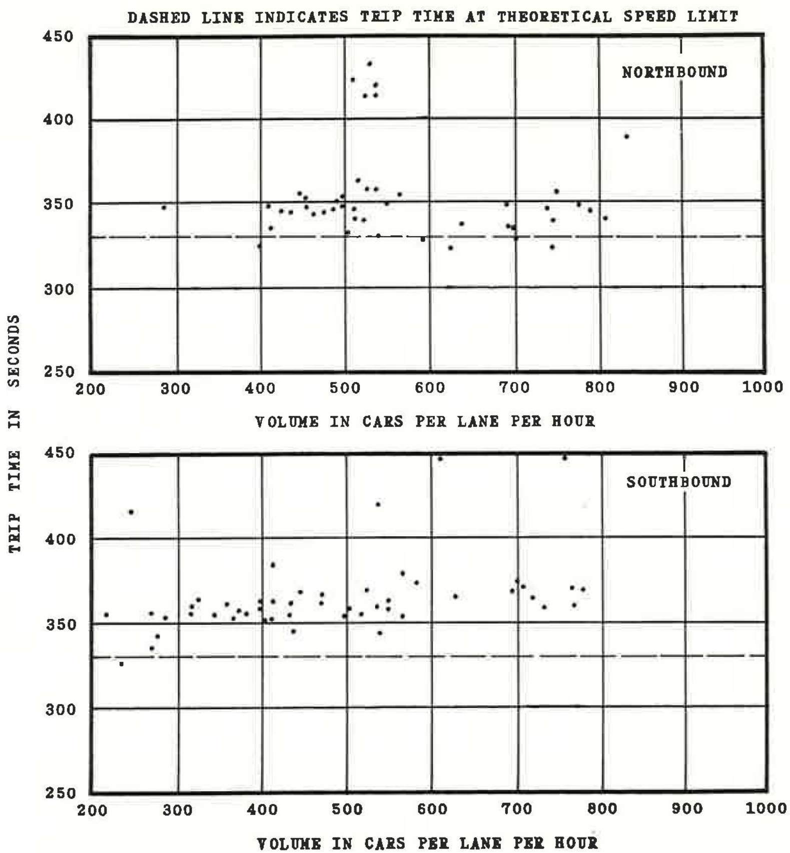

Figure 7. System D: 66 sec cycle; full pacer system; average stops, 1.23 per trip; average trip time, 359 sec per trip.

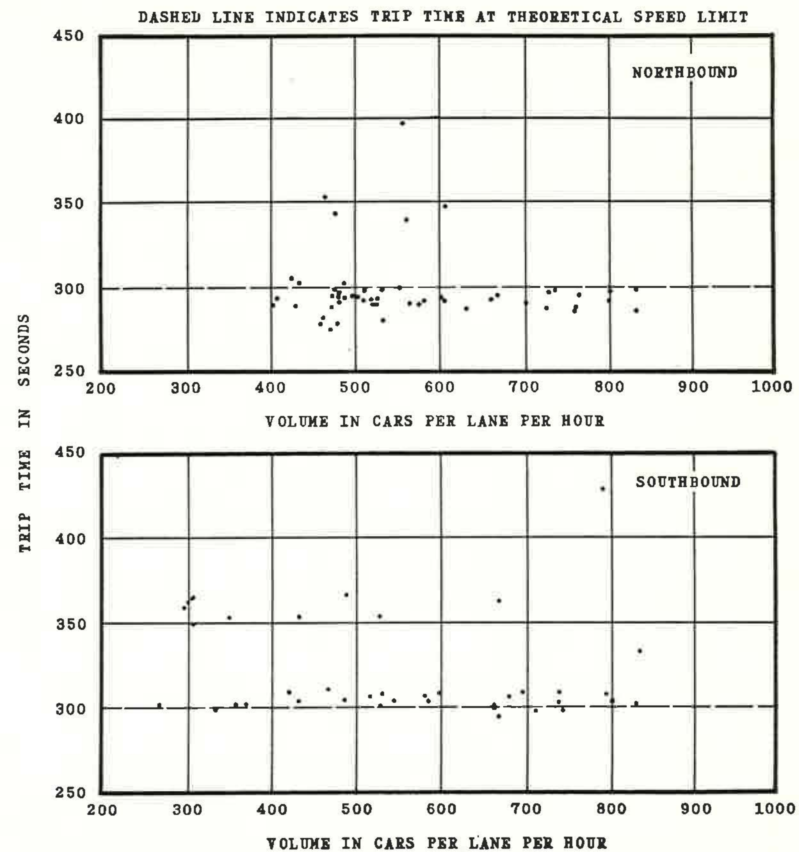

Figure 8. System E: 60 sec cycle; pre-stop only in operation; average stops, 0.57 per trip; average trip time, 315 sec per trip.

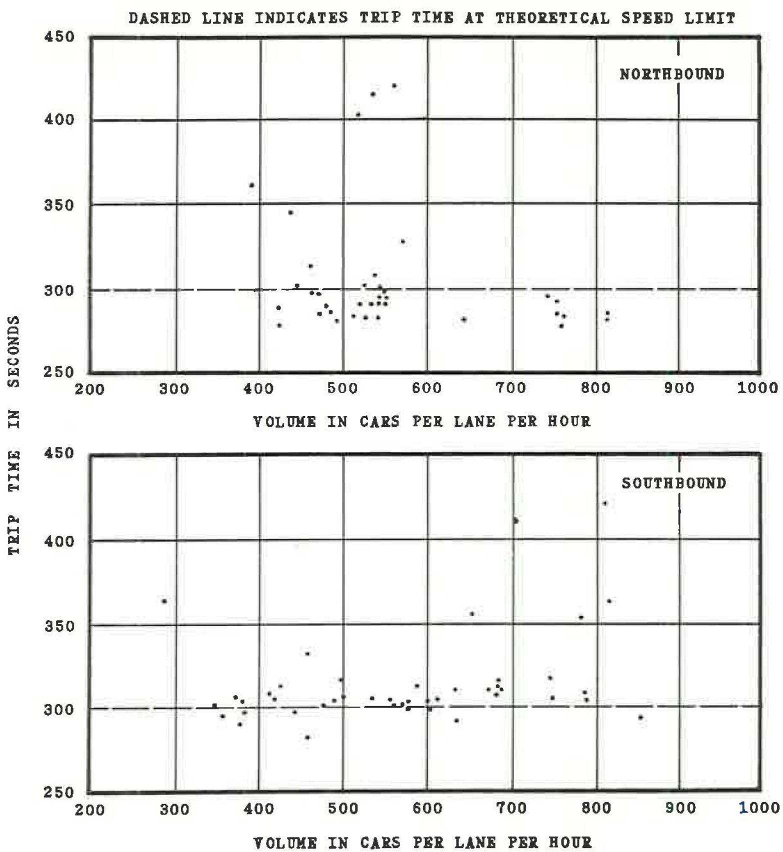

Figure 9. System F: 60 sec cycle; speed signs only in operation; average stops, 0.57 per trip; average trip time, 315 sec per trip.

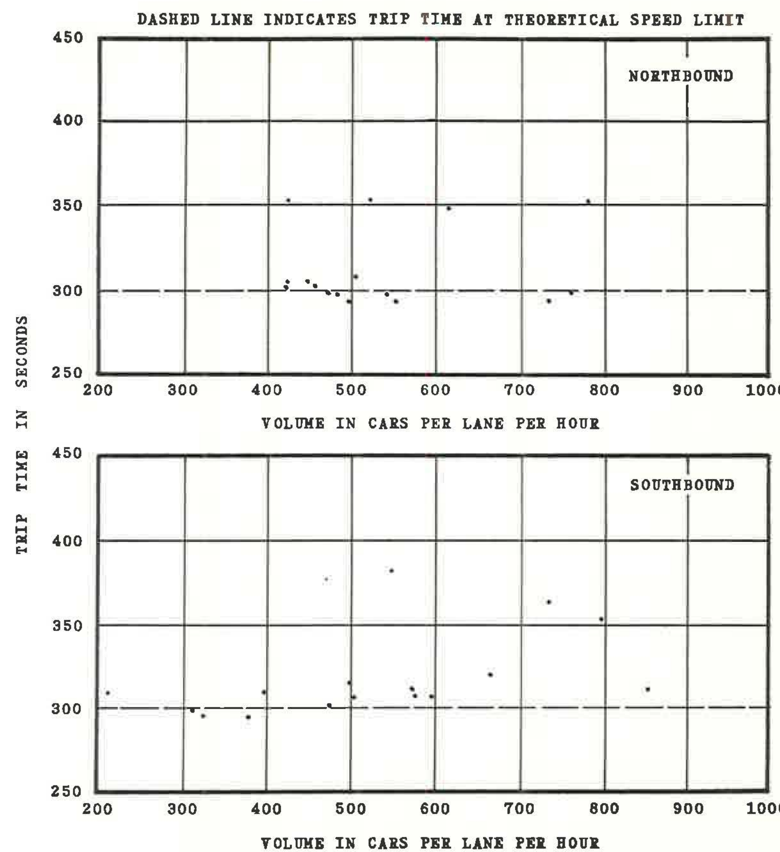

Figure 10. System G: 60 sec cycle; normal pacer except speed signs show only progression speed (no funneling); average stops, 0.54 per trip; average trip time, 320 sec per trip.

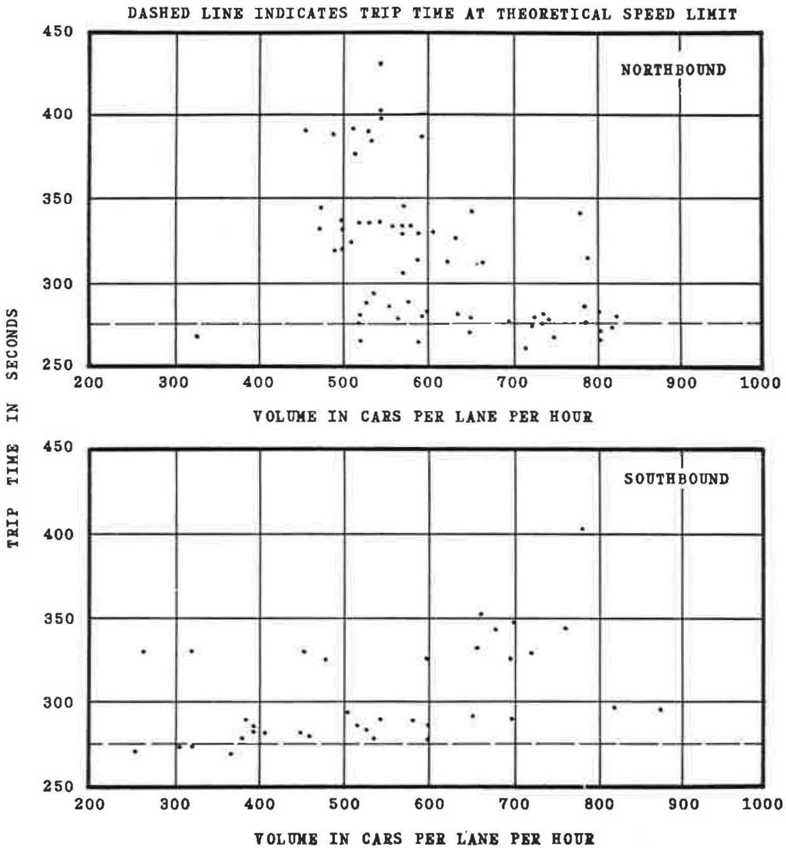

Figure 11. System H: 55 sec cycle; full pacer system; average stops, 0.84 per trip; average trip time, 315 sec per trip.

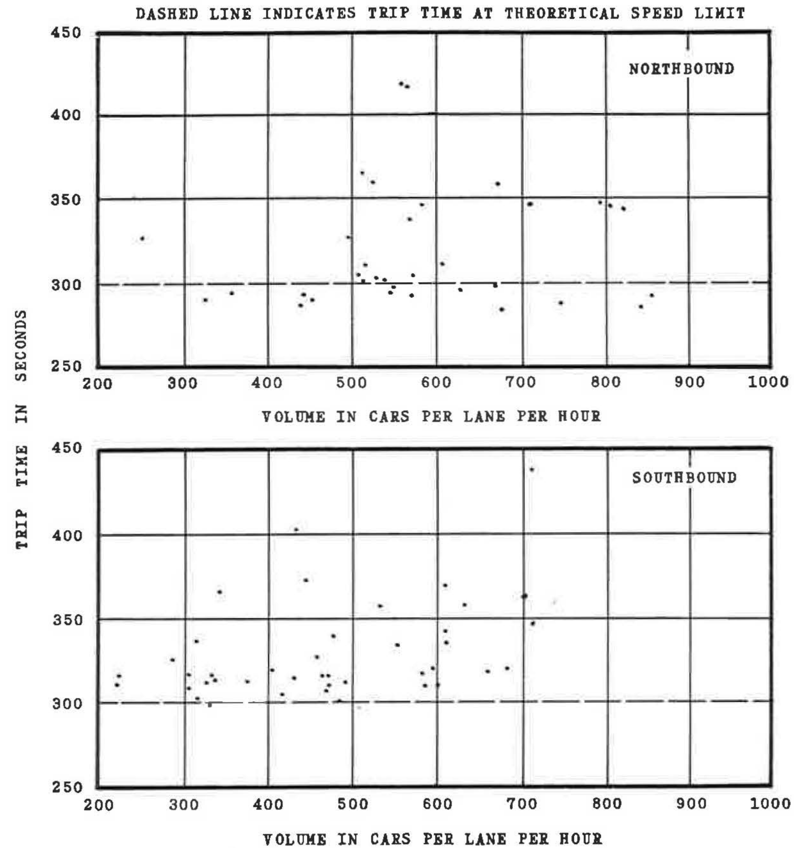

Figure 12. System K: 60 sec cycle; progressive system; average stops, 1.17 per trip; average trip time, 327 sec per trip.

TABLE 5

SUMMARY OF AVERAGE TRIP TIME, SPEED AND STOPS

Phase	No. of Trips	Avg. Stops Per Trip	Avg. Trip Time (sec)	Avg. Speed (mph)	Theo. Trip Time (sec)	Theo. Speed (mph)
A	180	0.62	320	33.8	300	36.0
B	65	0.63	316	34.2	300	36.0
C	59	0.41	337	32.0	349	30.9
D	102	1.23	359	30.1	330	32.7
E	106	0.57	315	34.3	300	36.0
F	100	0.57	315	34.3	300	36.0
G	62	0.54	320	34.7	300	36.0
H	153	0.84	315	34.3	275	39.3
K	96	1.17	327	33.0	300	36.0
L	56	3.33	375	28.8	-	-

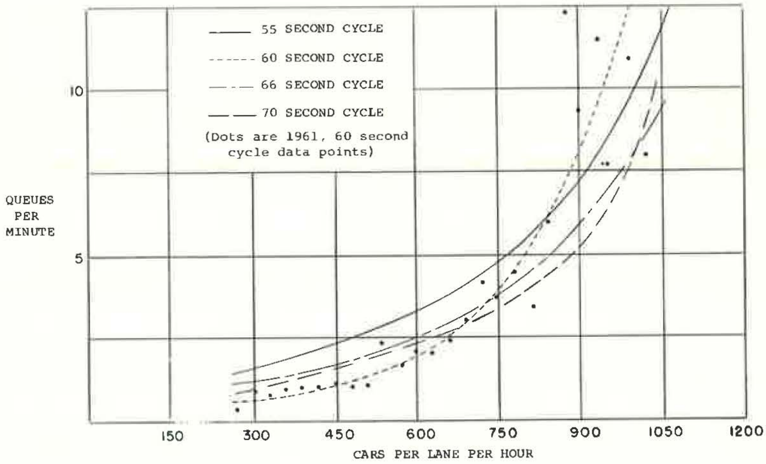

Figure 13. Queues vs traffic density.

tems over a specified time interval, the average number of cars through the intersection per cycle is not a particularly sensitive measure of intersection capacity unless the roadway is near saturation. When a roadway is not saturated, at a particular time interval during the day in which the traffic density is highest, the cars through per cycle are measured for N cycles. By plotting the percent of cycles in which n or more, $n + \Delta n$ or more, $n + 2\Delta n$ or more, etc., cars per lane per second go through the intersection, a comparison of several systems can be made, and the capacity of each system can be approximated by extrapolation.

Figure 14 shows a plot for the 12 Mile intersection over the given morning and afternoon time periods. Each cycle length tested is shown together with the 1961, 60 sec cycle data of the pacer system and the 1962, 60 sec cycle progressive system data. The 55 sec cycle is the most efficient on a car through per second basis.

Because of the small proportion of high volume cycles on Mound Road, the hypothesized 2-car-per-lane-per-cycle increase in capacity could not be adequately tested. A 25 percent increase in the number of cycles in which 15 or more cars per lane got through during the available green was observed for the pacer system over the progressive system. This compares with an increase of 31 percent during the summer of 1961.

System Simplification

In the simplification of the system (Table 1), phases B, E, F, and G were evaluations of a single-bulb-type speed display (southbound only), pre-signals only, speed signs only, and a full pacer system with no funneling action, respectively. All four of these phases were based on a 60 sec cycle and a 40 mph progression speed. Phase B

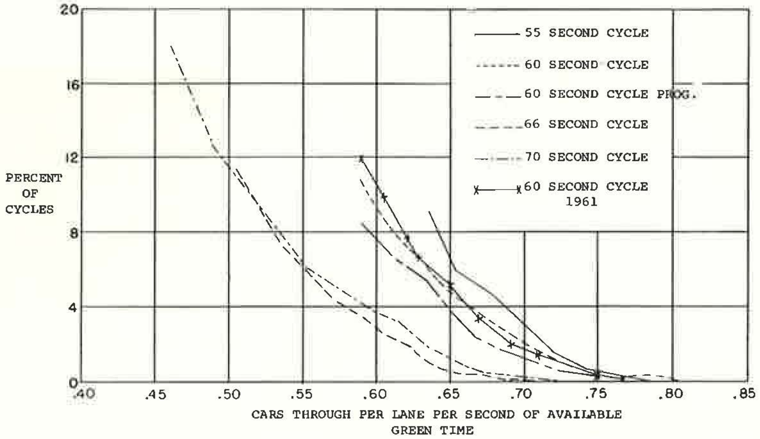

Figure 14. Intersection capacity.

is worthy of further comment. Although the speed display window of the single-bulb-type sign is much smaller than the multi-bulb sign, no impairment in efficiency was noted. The motorist is requested to comply with the indicated speed from the location of the display. Thus, if he is closer to it before he can read it, it is possible that better compliance to the system is achieved. A disadvantage of the single-bulb-type display is the limited number of funneling speeds that can be displayed. This presents no problem if only one progression speed is in use at all times. The cost and maintenance of the single-bulb-type speed sign is much less than the multi-bulb sign.

In phases E and F, the partial effect of the pre-signals and speed signs were examined. Table 5 and Figures 8-10 indicate a similarity between phases E, F, and G and the complete pacer system. The advantages of E, F, or G, as well as the complete pacer system, over the progressive system were in the fewer number of stops made. A 50 percent reduction was achieved. There were 12 mandatory stop points with the pre-signals in operation as compared to 7 with the progressive system (Phase K), yet the number of stops were halved (Table 5). The pre-signal is primarily useful within a small range of traffic densities close to road capacity. The pre-signal operation can break down at excessive traffic densities and can be an annoyance to light traffic. It is suggested that for the present level of traffic on Mound Road, pre-signals might be omitted or used as speed signs.

Intersection Arrival Time and Speed

The capacity of intersections will be increased if the stream of cars is moving as the light turns green, provided that the front of the "stream packet" is close enough to the intersection (6, 8). Because the leader of the stream packet is proceeding toward a red light as he leaves the pre-signal, his behavior is important to the efficiency of the system.

Figure 15 shows the "Traffic Signal Time Loss Meter" which was designed and built at the General Motors Research Laboratories. By installing two lengths of "Tape-Switch" on the roadway a known distance apart and just inside the intersection, the meter measured the arrival time and speed of the first motorist after onset of the green. Experiments were carried out at the 12 Mile and Mound Road intersection (northbound) with the corresponding pre-signal set back 846 ft from the intersection.

Figures 16 to 18 show individual arrival times and speeds for pre-signal time offsets of 13, 16.5 and 19.2 sec., respectively. These curves are a visual fit to the mean arrival speeds of Table 6. The present standard offset for the 60 sec. cycle is 17 sec. All the curves were based on the behavior of the first motorist leaving the pre-signal, whether he stopped or came through with a rolling start. Cycles in which cars were already stopped at the intersection or in which left turns into the roadway between the pre-signal and the intersection were made were deleted.

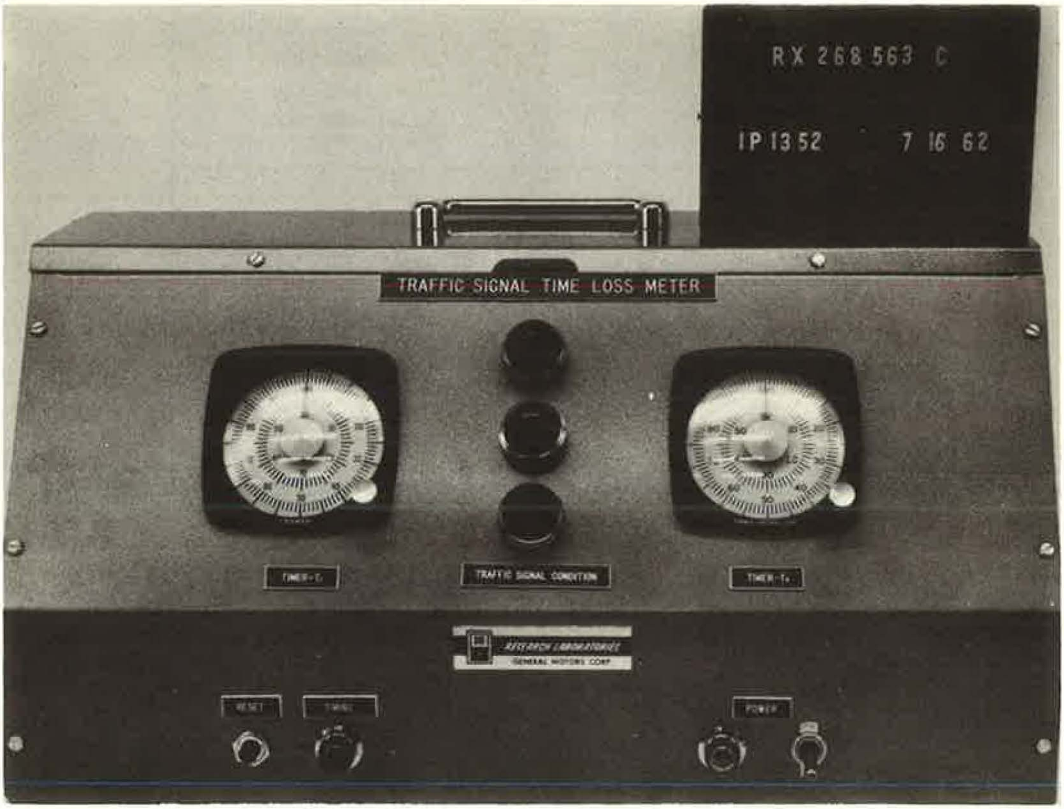

Figure 15. Traffic signal time loss meter.

Table 6 gives the arrival speed and time to the nearest second for the three time offsets. The mean speed is given for all cars arriving in the same 1-sec time interval. Table 7 gives the normalized arrival time to a 100 car base and the weighted mean arrival times for the three time offsets.

Pre-Signal Placement

It was considered that work carried out on driver responses to the amber phase of the traffic signal would give a relative measure of a motorist's behavior to a red light at the beginning of the cycle (9). The data from the drivers' responses gave an estimate of the probability of stopping for vehicles as a function of their distance from the intersection at the onset of the amber phase of the traffic signal. Similarly, if the motorist has to travel a distance of L ft from the pre-signal to the intersection, his motion must be such that at some distance, S, from the intersection the red to green light change must occur, or he will start braking. Dividing the motion into an acceleration period and a constant speed period, his position at a particular time is given by:

$$S = L - (\tfrac{1}{2} a t_1^2 + v t_2) \tag{1a}$$

in which

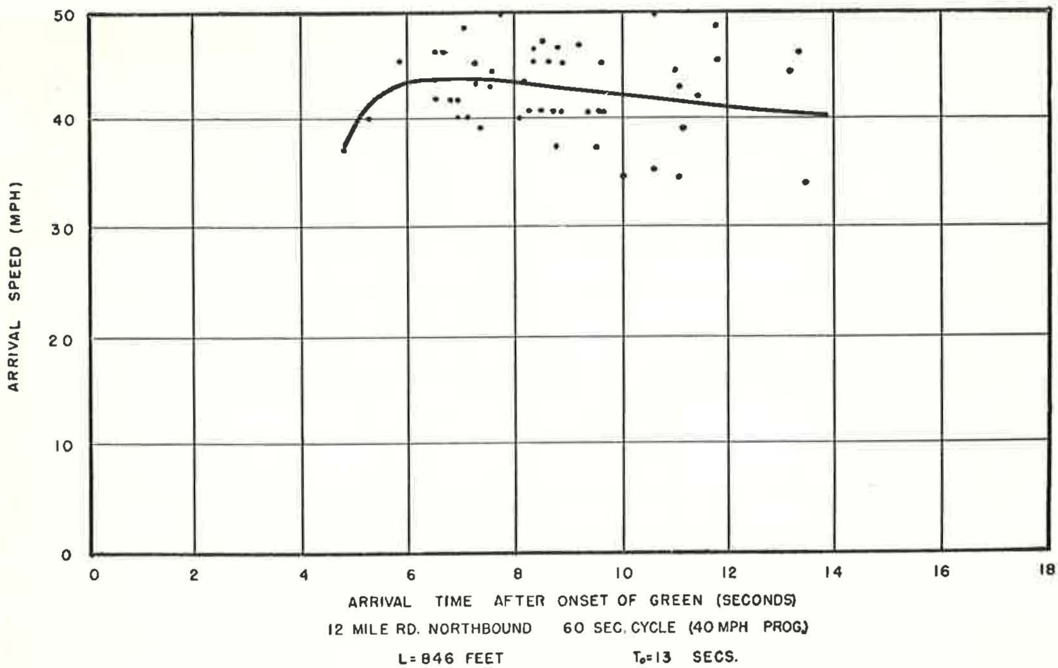

Figure 16. Arrival time and speed for pre-signal offset of 13 sec.

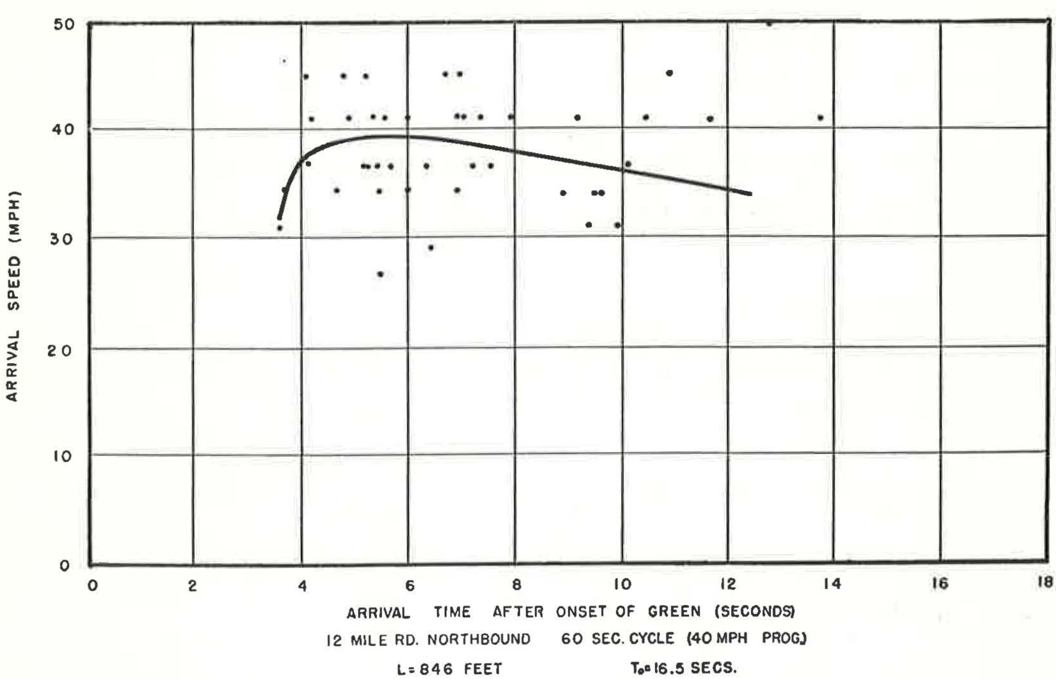

Figure 17. Arrival time and speed for pre-signal offset of 16.5 sec.

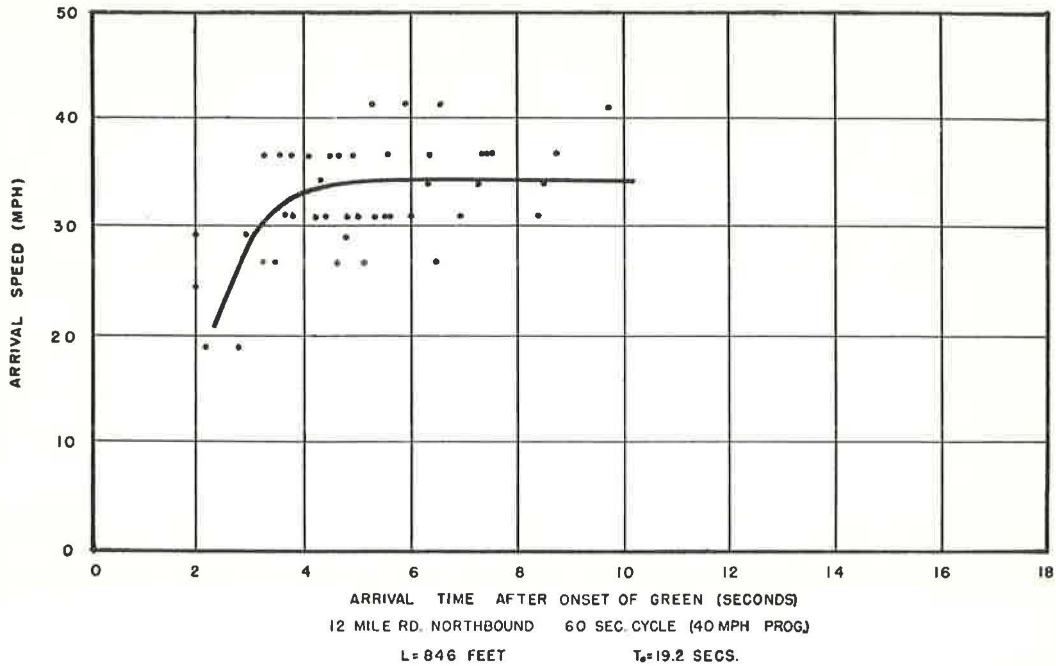

Figure 18. Arrival time and speed for pre-signal offset of 19.2 sec.

a = acceleration in ft/sq sec,
t_1 = time spent accelerating in sec,
v = velocity attained at end of acceleration in fps,
t_2 = time spent at constant speed in sec,
L = distance from pre-signal to intersection in ft, and
$(t_1 + t_2)$ = time after pre-signal release in sec.

If a position of 260 ft from the intersection is taken as the edge of the dilemma zone for 90 percent of the drivers at 40 mph, (9), the following relationship can be obtained at $(t_1 + t_2)$ sec:

$$L - (\tfrac{1}{2} a t_1^2 + v t_2) = 260 \tag{1b}$$

and time offset = $(t_1 + t_2)$ sec. It is assumed that the motorists starts from rest on the pre-signal release. However, if he is moving toward the pre-signal at the progression speed, he will experience a dilemma zone and a subsequent loss in time at the pre-signal. This time loss

TABLE 6
ARRIVAL TIME AND SPEED OF FIRST CAR

Green Time (sec)	Time Offset					
	13 Sec		16.5 Sec		19.2 Sec	
	No. of Cars	Mean Arrival Speed	No. of Cars	Mean Arrival Speed	No. of Cars	Mean Arrival Speed
0-1	0	-	0	-	0	-
0-2	0	-	0	-	0	-
2-3	0	-	0	-	5	24
3-4	0	-	1	33	8	32
4-5	1	37	6	41	11	33
5-6	2	43	10	37	9	34
6-7	8	44	7	38	5	34
7-8	10	44	6	38	5	36
8-9	14	43	2	36	3	34
9-10	6	42	5	34	1	41
After 10	12	42	6	41	0	-

TABLE 7
ARRIVAL TIME (NORMALIZED TO 100 CARS)

Green Time (sec)	Time Offset		
	13 Sec	16.5 Sec	19.2 Sec
0-1	0	0	0
1-2	0	0	0
2-3	0	0	11
3-4	0	3	17
4-5	2	17	23
5-6	5	26	20
6-7	20	19	10
7-8	23	16	10
8-9	35	6	7
9-10	15	13	2
Weighted Average Arrival Time (sec)	8.3	7	5.7

TABLE 8

OPTIMUM PRE-SIGNAL PLACEMENT

Progression Speed (mph)	Acceleration Rate (ft per sq sec) 4		5		6	
	L (ft)	t (sec)	L (ft)	t (sec)	L (ft)	t (sec)
30	285	7.5	263	6.0	248	5.0
35	371	8.75	338	7.0	318	5.83
40	460	10.0	420	8.0	393	6.67
45	572	11.25	521	9.0	488	7.5
50	691	12.5	629	10.0	587	8.33

compensates for the acceleration time of the stationary start.

Thus, Eq. 1b gives a relationship by which pre-signal time offsets can be approximated. If the time offset ($t_1 + t_2$) is set to a motorist who accelerates at 6 ft per sq sec to the speed limit, the smallest reasonable offset is readily determined; i.e., for 12 Mile Road (northbound) where L = 846 ft, $846 - [1/2 6 t_1^2 + (6 t_1) t_2] = 260$ and (6 ft per sq sec) (t_1 sec) = (40) mph = 58.7 fps, $t_1 = 9.8$ sec and $t_2 = [846 - 260 - 1/2 6 (9.8)^2] / [6 (9.8)] = 298/58.7 = 5.1$ sec. Thus, a time offset of 14.9 sec is a reasonable minimum. The actual time loss in this instance (green time before arrival) would be $t = s/v = 260/58.7 = 4.4$ sec.

For any new installation of a pacer type system, optimum placement of pre-signals and offset times can be determined for any desired speed limit. If the lead car of a platoon has to slow down because the light has not changed, the impact can be felt down the line of cars, reducing any capacity advantage that the pre-signal provides. Ideally, the system should be designed so that for some upper limit of acceleration (6 ft per sq sec), the traffic signal will change to green just as the lead car reaches the progression speed, i.e., the end of the acceleration period.

TABLE 9

ACCIDENTS ON VAN DYKE (11-14 MILE)

Period	No.
Jan. 1-Dec. 31, 1958	138
Jan. 1-Dec. 31, 1959	187
Jan. 1-Dec. 31, 1960	228
Jan. 1-Sept. 30, 1961	177
Oct. 1-Dec. 31, 1961	74
Jan. 1-Sept. 30, 1962	251

TABLE 10

ACCIDENTS IN WARREN (ALL CITY)

Period	No.
Jan. 1-Dec. 31, 1958	2,072
Jan. 1-Dec. 31, 1959	2,831
Jan. 1-Dec. 31, 1960	3,421
Jan. 1-Sept. 30, 1961	2,401
Oct. 1-Dec. 31, 1961	949
Jan. 1-Sept. 30, 1962	2,993

TABLE 11

ACCIDENTS ON MOUND ROAD (11-14 MILE)

Period	No.
Jan. 1-Dec. 31, 1958	51
Jan. 1-Dec. 31, 1959	111
Jan. 1-Dec. 31, 1960	138
Jan. 1-Sept. 30, 1961	66
Oct. 1-Dec. 31, 1961, Pacer	29
Jan. 1-Sept. 30, 1962, Pacer	80

Table 8 shows the optimum placement, L, of pre-signals from the intersection and the corresponding offset time, t, for three acceleration rates and five progression speeds. The dilemma zones for 90 percent of the drivers for the progression speeds of 30, 35, 40, 45, and 50 mph are 173, 216, 260, 319, and 379 ft, respectively. It, therefore, appears that somewhat shorter pre-signal to intersection distances than were used in the traffic pacer installation are desirable.

Accident Statistics

The analysis of accident data for the purpose of comparing two kinds of traffic systems is difficult. There are no two sections of roadway on which the geometry is the same and the number of cars using the road is identical. If the same road is used over different time periods, the weather has an important effect on how many accidents will occur.

The following accident data has been collected from Warren Police Department records and includes accidents of every degree of severity. In each case there was generally property damage to at least one vehicle. Personal injuries are not differentiated from property damage accidents but are included in the totals.

Tables 9 to 11 show the accident totals for the entire City of Warren, Van Dyke be-

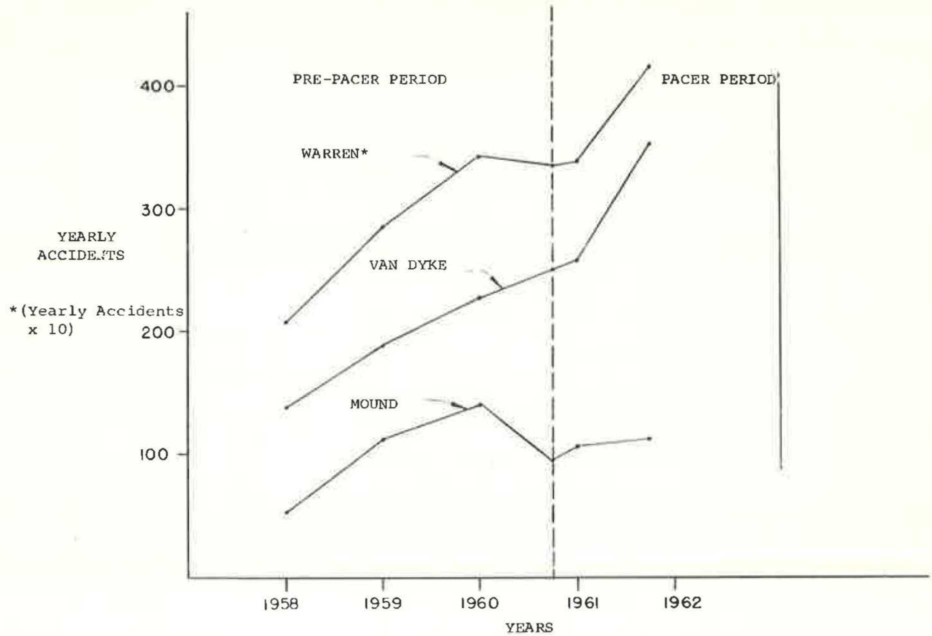

Figure 19. Accident statistics.

tween 11 and 14 Mile Roads (the adjacent parallel road 1 mi east of Mound Road) and the test section of Mound Road (11 to 14 Mile Roads). The year 1961 is split into the first 9 mo and the last 3 mo in each table so that comparisons could be made with the pacer system which was in continuous use from Oct. 15, 1961. The data from Tables 9 to 11 are plotted in Figure 19. The incomplete year totals are given projected yearly totals for easier comparison.

The percentage increase in accidents during the pacer period, for Warren, Van Dyke, and Mound were 24.7, 41.8, and 19.8 percent, respectively. The increase in accident rate from Mound during this 1-yr period was approximately 20 percent less than that for the City of Warren and 53 percent less than that for Van Dyke Road. Statistical significance cannot be assigned to these differences because of the insufficient data.

SUMMARY AND CONCLUSIONS

During the 15 mo of testing, results indicated that the pacer system significantly decreased the number of stops when compared with two conventionally used systems of traffic control. Although increases in intersection capacity were observed during the operation of the pacer system, the hypothesized 2-car-per-lane-per-cycle increase was not realized. This does not necessarily reflect on the pacer system but may be due to the lack of peak volume periods of any appreciable duration. Although accident statistics were in general agreement with the original hypothesis, additional long-term data are needed to reach a valid conclusion.

The pacer system, depending on its application, can be significantly simplified without any loss in efficiency. For example, the variable multi-bulb speed sign can be replaced by a more economical and dependable fixed single-bulb type. Possibly an even more economical way of presenting speed information to the driver can be developed. Pre-signals can be eliminated for low and moderate volume traffic densities. The funneling action of speed signs can be eliminated if cost and system simplicity are prime considerations. Optimum placement of pre-signals has also been determined so that intersection delays can be minimized.

APPLICATIONS

The traffic pacer system has many potential uses in traffic control. A few of the more obvious uses are:

1. The placement of two or three speed signs near the end of an expressway or freeway to funnel traffic into the surface street progressive system;
2. Speed signals before small towns so that motorists will be able to adjust to the existing local traffic system;
3. The use of pre-signals before school crossings at major intersections;
4. The proper timing of pre-signals to eliminate the dilemma zone the motorist faces when he is confronted by an amber light. If the motorist makes it through the pre-signal, even on the amber, he will make the intersection signal on the green, assuming no loss in car speed;
5. The use of speed signs and pre-signals to regulate traffic flow through tunnels;
6. Placement of speed signs on the approaches to isolated intersections.

REFERENCES

1. Greenberg, H., "An Analysis of Traffic Flow." Operations Research 7, 79 (1959).
2. Herman, R., Montroll, E.W., Potts, R.B., and Rothery, R.W., "Traffic Dynamics Analysis of Stability in Car Following." Operations Research 7, 86 (1959).
3. Herman, R., and Potts, R.B., "Single-Lane Traffic Theory and Experiment." Theory of Traffic Flow, Elsevier Pub. Co., Amsterdam (1961).
4. Bidwell, J., "The Car Road Complex." Theory of Traffic Flow, Elsevier Pub. Co., Amsterdam (1961).
5. Morrison, H.M., Underwood, A.F., and Bierley, R.L., "Traffic Pacer." HRB Bull. 338, pp. 40-68 (1962).
6. von Stein, W., "Traffic Flow with Pre-Signals and the Single Funnel." Theory of Traffic Flow, Elsevier Pub. Co., Amsterdam (1961).
7. Bauer, H.J., "Some Solutions of Visibility and Legibility Problems in Changeable Speed Command Signs." HRB Bull. 330, pp. 60-68 (1962).
8. Robbins, J., "Capacity Effects of Advance Signal." Masters Thesis, Yale Univ. (1960).
9. Olson, P., and Rothery, R., "Driver Response to the Amber Phase of Traffic Signals." Operations Research 9, 650 (1961).

Appendix

1962 PACER TEST PROGRAM

The following is a detailed description of the traffic systems tested (Table 1):

Phase A—Cycle length, 60 sec; progression speed, 40 mph; pacer system with large multi-bulb signals in both north and southbound lanes.

Phase B—Cycle length, 60 sec; progression speed, 40 mph; pacer system with 8-in. high numerals ½ in. wide in southbound lane, northbound lane signs as A.

All other phases of test program will have speed signs as phase B. Display uses a 12-in. square, crosshatched amber lens, with single "Verteray" 67-w frosted bulb.

Phase C—Cycle length, 70 sec; progression speed, 35 mph.
Phase D—Cycle length, 66 sec; progression speed, 40 mph.
Phase E—Cycle length, 60 sec; progression speed, 40 mph; pre-signals only (speed signs shut down).
Phase F—Cycle length, 60 sec; progression speed, 40 mph; speed signs only (pre-signals shut down).
Phase G—Cycle length, 60 sec; progression speed, 40 mph; pre-signals and speed signs in operation but showing progression speed only at progression time, i.e., no funneling for early vehicles.

Phase H—Cycle length, 55 sec; progression speed, 45 mph; with pacer system in full operation including funneling speeds. This phase involved changing most of the road side speed limits and was cleared through the Macomb County Road Commission prior to operation.

Phase K—Cycle length, 60 sec; progression speed, 40 mph; progressive system with pre-signal lights turned sideways and all speed display signs turned off.

Phase L—Cycle length, 60 sec; arbitrary speeds between any adjacent intersection signals; past system with pre-signal lights turned sideways and all speed display signs turned off. Time offsets were picked from a table of random numbers. Stops and trip times were taken from 1961 test program.