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Impact of Using Freeway Shoulders as Travel Lanes on Fuel Consumption

WILLIAM R. McCASLAND

A general procedure, based on data derived from the American Association of State Highway and Transportation Officials "Redbook", to compute the savings in fuel consumption that result from the use of low-cost conversions of urban freeway shoulders to travel lanes is presented. The data required for the analysis are traffic volumes, speeds, and vehicle classifications before and after the improvements. Three example projects are presented to illustrate the range of benefits in terms of total travel time and fuel consumption. The projects were implemented in Houston with three different objectives: relieve a main-lane freeway bottleneck, bypass a main-lane queue, and provide priority operation for high-occupancy vehicles (HOVs). The results of the study indicate that major improvements in traffic operations can be achieved from the additional capacity provided by the shoulder lanes. The magnitude of the improvements depends on the type of use of the lane, the geometric design, and the traffic conditions in the freeway section to be modified. In the three examples, the annual savings in fuel ranged from 187 875 gal for relieving a bottleneck to 7890 gal for bypassing main-lane queues to 3423 gal for HOV priority operations. The savings are the result of improvements in the average speeds of vehicles that use the modified sections. If the improvements can be related to a modal shift for the HOV priority operations, the savings are much greater.

Over the years, design standards have established a typical cross section for urban freeways with six or more lanes to be 12-ft lanes with 8- to 10-ft shoulders on both sides of the roadways. However, rising traffic demands and declining financial resources have forced transportation agencies to consider modification of these design standards to increase capacity. One approach that is inexpensive and fast to implement is the conversion of the roadway shoulders to travel lanes. The advantages of increasing capacity of a section "overnight" are obvious: increased service volumes, lower travel times, and reduced traffic delays. There are also disadvantages, both to the users and the transportation agency: the quality of ride may be less than desirable, the reduction of space available for emergency parking increases the potential for accidents that involve disabled vehicles, and the pavement structure of the shoulder may not be designed to handle the increased loads, thereby resulting in increases in maintenance costs and a need for early replacement of the shoulder. There was concern that the total accident rate would increase, but that has not happened (1).

There are ways to reduce or eliminate these disadvantages:

1. The shoulder can be strengthened and the riding surface can be improved prior to the conversion,
2. A temporary shoulder or special vehicle turnout bays can be constructed for disabled vehicles, and
3. The traffic loads applied to the shoulder can be reduced by restricting the use by heavier vehicles and/or restricting the use to peak periods only, when the additional capacity is needed.

These measures can also extend the life of the shoulder for many years.

There are several reasons for considering the use of the shoulder for travel (1):

1. Overloading the outside lane: The shoulder can be converted to an auxiliary lane.
2. Bypassing main-lanes queues: Queues formed at freeway-to-freeway interchanges can block traffic movements that are not required to pass through the bottleneck.
3. Relieving freeway bottlenecks: Freeway sections that have traffic demands that exceed the roadway capacity can benefit from the added capacity.
4. Reducing merge conflicts: Freeway interchange merge areas that are overloaded or are high accident locations can be improved by using the shoulder on one roadway to move the merge area away from the interchange or to eliminate it entirely.
5. Providing for priority operation of high-occupancy vehicles (HOVs): A shoulder can be converted to an HOV-only lane to provide a higher level of service during peak periods.
6. Improving capacities in work zones: Shoulders can be used to restore some of the capacity lost by lane closures for maintenance and construction activities.

BENEFITS OF USING SHOULDERS FOR TRAVEL

The primary benefit in the conversion of the shoulder to a travel lane is the increase in capacity at very low costs. The amount of benefits will vary greatly, depending on the reason for the conversion, the geometric design of the section, and the traffic conditions. The following examples are presented to illustrate the range of benefits that can be obtained.

Example A: Relieving a Freeway Bottleneck

The Southwest Freeway (US-59) in Houston was restriped to add an additional lane for a distance of 1 mile (2) (see Figure 1). The section was four lanes for one-third of a mile and three lanes for two-thirds of a mile. The added lane changed the cross section to five lanes for one-third of a mile, four lanes for one-third of a mile, and three lanes for one-third of a mile. In this example, the additional lane had a very high use because of the high volumes destined for the two exit ramps. The results were an increase of 22 percent in the service volumes to 1700 vehicles/h, a reduction in total travel time during the 2-h peak period of 1000 vehicle-h, and an increase in the average speeds from 20 to 40 mph over a distance of 3 miles (2 miles upstream of the modified section).

Many projects have reported similar results [Table 1 (3)], such as

1. Denver, on I-25, with travel time savings of 500 vehicle-h, and the speeds increased from 26 to 46 mph, and
2. Los Angeles, on the Santa Monica Freeway (I-10), with travel time savings of 850 vehicle-h, and the speeds increased from 22 to 40 mph (3).

Example B: Bypassing Main-Lane Queues

The West Loop Freeway (I-610) in Houston was restriped to provide a bypass lane 0.5 mile in advance of the interchange of I-10 (see Figure 2). The traffic in the evening peak period destined for westbound (outbound) I-10 formed queues of a maximum

Figure 1. Use of shoulder lane on Southwest Freeway (US-59) in Houston.

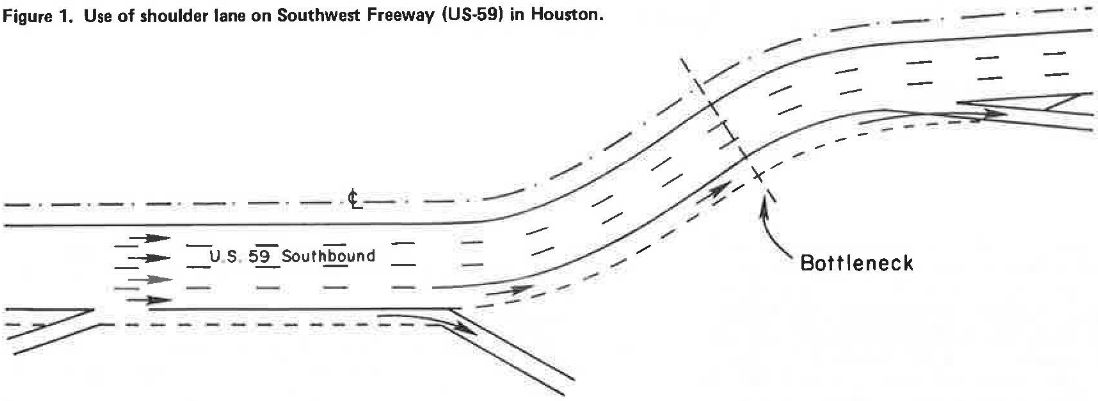
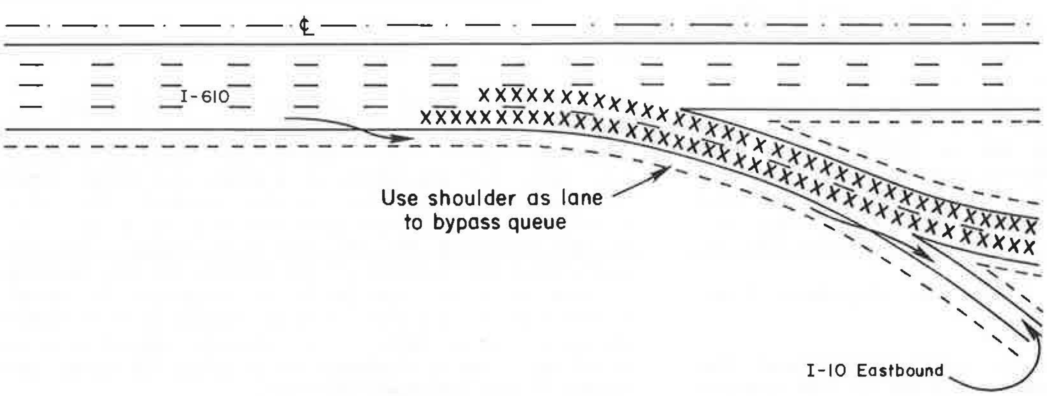


Table 1. Summary of operational experience.

City	Freeway	Capacity Increases (vehicles/h) Before	After	Delay Reduction (vehicle-h/day)	Speed Increase (mph) Before	After
Denver	I-25	6376	7146	500	26	46
Houston	Southwest (US-59)	5800	7100	1000	-	-
Los Angeles	Santa Monica (I-10)	7680	9600	850	22	40
	San Bernardina	6900	7600	850	-	-
	Pomona	-	-[a]	100	30	40
	Ventura	7700	8500	750	30	56
	Golden State	-	-[b]	820	37	55
	Santa Ana	7700	8500	625	20	50
Nashville	I-65, I-265	4150	5200	-	-	-[d]
Portland	I-5, I-405	3400	4120	-	-	-[d]
	Banfield	-	-[c]	-	33	38
San Francisco	CA-280	5460	7090	250	-	-

[a]Increased capacity by providing a truck climbing lane.
[b]Increased volumes.
[c]Vehicles increased by 2 percent and persons increased by 10 percent.
[d]Free flow.

Figure 2. Use of shoulder lane on West Loop Freeway in Houston.



length of 1 mile. The traffic destined for eastbound I-10 was delayed unnecessarily. Providing the bypass lane during the peak period saved 16 vehicle-h for 830 vehicles that used the 0.5-mile bypass. This represents an increase in speeds from 20 to 45 mph.

Example C: HOV Priority Lane

The 3 miles of the inside shoulder of the North Freeway (I-45) in Houston was converted to an HOV priority lane for vehicles that were authorized to use a contraflow lane (4) (see Figure 3). There are 62 buses and 218 vanpool vehicles that use the shoulder lane during the morning peak period. Travel time savings to HOVs were 3.17 min/vehicle, which resulted in a daily savings of 14.8 vehicle-h. In terms of persons carried, the savings were much more significant, resulting in 190 person-h/day. The average speeds for HOVs increased from 26 to 48 mph, while the speeds in the main lanes were unchanged.

Many other projects can be cited for all of the various reasons for converting shoulders to travel, and each would have a unique set of benefits in terms of travel time saved and improvements in average speeds.

IMPACT ON FUEL CONSUMPTION

Estimates of savings of fuel as a result of the

Figure 3. Use of median shoulder as HOV lane on North Freeway (I-45) in Houston.

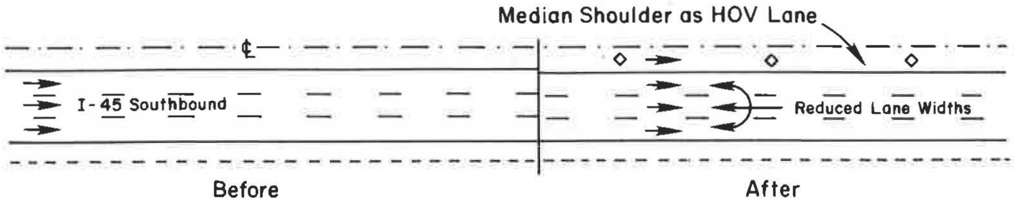


Table 2. Fuel-consumption rates for vehicle type 1 on freeways by LOS and average speed.

Avg Speed (mph)	Fuel Consumption (gal/vehicle mile) by Level of Service					
	A	B	C	D	E	F
5						0.3970
10						0.1649
15						0.1028
20						0.0772
25						0.0641
30					0.0433	0.0574
35				0.0420	0.0428	0.0431
40			0.0426	0.0429	0.0444	
45		0.0427	0.0443	0.0450	0.0465	
50	0.0438	0.0454	0.0471	0.0486		
55	0.0468	0.0489	0.0512			
60	0.0494	0.0519				
65	0.0567					

Note: Table is based on the proportion of fuel cost to total cost at various speeds as reported in the 1977 AASHTO Redbook (5) and applied to total costs as reported in Buffington and McFarland (7). The table is adjusted for 1980 costs of fuel. The fuel costs of the latter report were originally based on the fuel-consumption rates reported by Claffey (8) and Winfrey (9).

Table 4. Fuel-consumption rates for vehicle type 3 on freeways by LOS and average speed.

Avg Speed (mph)	Fuel Consumption (gal/vehicle mile) by Level of Service					
	A	B	C	D	E	F
5						3.1346
10						1.0550
15						0.5660
20						0.3784
25						0.2963
30					0.1567	0.2445
35				0.1503	0.1552	0.1613
40			0.1529	0.1566	0.1646	
45		0.1613	0.1676	0.1722	0.1745	
50	0.1778	0.1860	0.1951	0.2026		
55	0.1928	0.2041	0.2167			
60	0.2017	0.2128				
65	0.2208					

Note: Table is based on fuel-consumption rates and fuel costs as a proportion of total costs as reported in the 1977 AASHTO Redbook (5) and on total costs reported by Buffington and McFarland (7). The table is adjusted for 1980 costs of fuel.

Table 3. Fuel-consumption rates for vehicle types 2 and 4 on freeways by LOS and average speed.

Avg Speed (mph)	Fuel Consumption (gal/vehicle mile) by Level of Service					
	A	B	C	D	E	F
5						0.6765
10						0.3491
15						0.2249
20						0.1772
25						0.1635
30					0.1139	0.1577
35				0.1250	0.1281	0.1319
40			0.1329	0.1346	0.1395	
45		0.1412	0.1457	0.1486	0.1528	
50	0.1486	0.1542	0.1603	0.1654		
55	0.1613	0.1687	0.1769			
60	0.1782	0.1862				
65	0.1981					

Note: Table is based on fuel-consumption rates and fuel costs as a proportion of total costs as reported in the 1977 AASHTO Redbook (5) and on total costs reported by Buffington and McFarland (7). The table is adjusted for 1980 costs of fuel.

improved traffic operations can be determined from the vehicle miles of travel at the average speed for the before and after conditions. Fuel-consumption rates for freeways have been developed from data reported in the 1977 American Association of State Highway and Transportation Officials (AASHTO) "Redbook" (5) and updated to 1980 costs by Buffington and McFarland (6) (see table below and Tables 2-4):

Vehicle Type No.	Vehicle Type Description
1	Automobiles, pickups, and panel trucks (2-axle, 4-tire)
2	Single-unit trucks (other than 2-axle, 4-tire)
3	Truck tractor-semitrailer or trailer combinations
4	Buses

The rates in Tables 2-4 provide conservative measures, since the average fuel-consumption rates have continued to decline with newer vehicle models.

The fuel-consumption tables provide a range of values at speeds greater than 30 mph based on the quality of flow as measured by the level of service (LOS). Any type of freeway improvement can be analyzed if before-and-after data on traffic volumes, speeds, and composition of traffic are known. For the study of the conversion of shoulders to travel lanes, LOS D is used to describe the after conditions because of the reduction of lateral clearances, quality of roadway surface, and lane widths. For the example projects, the impact on fuel consumption is calculated in the following manner.

Example A: Relieving a Freeway Bottleneck

The statistics for this example are as follows:

13 800 vehicles during a 2-h peak period,
20-mph before speed (LOS F),
40-mph after speed (LOS D),
3-mile length of freeway affected by modification,
97 percent type 1 vehicles (passenger cars, light trucks),
2 percent type 2 and 4 vehicles (medium trucks and buses), and
1 percent type 3 vehicles (heavy trucks)

The change in the vehicle fuel-consumption rate is as follows (from Tables 2-4):

$$(0.97)(0.0772 - 0.0429) + (0.02)(0.1772 - 0.1346) + (0.01)(0.3784 - 0.1566) = 0.0363 \text{ gal/vehicle mile.}$$

The average daily savings can be calculated for the total vehicle miles traveled during the peak period:

Table 5. Summary of example projects.

Project Designation	No. of Vehicles Affected	Change in Speed (mph) Before	After	Length of Project (miles)	Change in Travel Time (vehicle-h)	Change in Fuel Consumption (gal/year)
A	13 800	20	40	3.0	-1000	-187 875
B	830	20	45	0.5	-16	-7 890
C	280	26	48	3.0	-14.8	-3 423
C′	2 765	48	26	3.0	+304	+115 135

Table 6. Speed-fuel relation for passenger cars.

Travel Time (min/mile)	Avg Speed (mph)	Fuel Consumption (gal/mile)	Fuel Economy (miles/gal)
60.0	1	0.7822	1.278
20.0	3	0.2849	3.510
12.0	5	0.1854	5.394
8.6	7	0.1428	7.004
6.7	9	0.1191	8.397
5.5	11	0.1040	9.614
4.6	13	0.0936	10.686
4.0	15	0.0859	11.637
3.5	17	0.0801	12.487
3.2	19	0.0755	13.251
2.9	21	0.0717	13.942
2.6	23	0.0686	14.570
2.4	25	0.0660	15.142
2.2	27	0.0638	15.667
2.1	29	0.0619	16.149
1.9	31	0.0603	16.594
1.8	33	0.0588	17.005
1.7	35	0.0575	17.387

0.0363 gal/vehicle mile x 13 800 vehicles x 3 miles = 1503 gal of fuel saved each weekday.

These benefits are only achieved when the roadway operates without incident. The number of incident-free days is assumed to be 125/year, which results in an annual savings of 187 875 gal of fuel.

Example B: Bypassing Main-Lane Queues

The statistics for this example are as follows:

830 vehicles use the 0.5-mile bypass lane,
20-mph before speed (LOS F) on incident-free days,
10-mph before speed (LOS F) on incident days,
45-mph after speed (LOS D), and
Only type 1 vehicles can use the bypass lane.

The average change in the vehicle fuel-consumption rate for nonincident days is as follows:

(0.0772 - 0.0450) = 0.0322 gal/vehicle mile.

For incidents days it is

(0.1649 - 0.0450) = 0.1199 gal/vehicle mile.

The annual savings in fuel consumption are

(0.0322 gal/vehicle mile + 0.1199 gal/vehicle mile) 830 vehicles) (0.5 mile) (125 days) = 7890 gal.

Example C: HOV Priority Lane

The statistics for this example are as follows:

62 buses (type 4) and 218 vanpools (type 1) use the 3-mile priority lane each weekday,
26-mph before speed (LOS F), and
48-mph after speed (LOS D).

The before speed is assumed to be the same for incident days. The average change in the vehicle fuel-consumption rate is

(62/280) (0.1775 - 0.1587) + (218/280) (0.0628 - 0.0472) = 0.0163 gal/vehicle mile.

The annual savings in fuel consumption are

0.0163 gal/vehicle mile x 280 vehicles x 3 miles x 250 days = 3423 gal.

Example C': Elimination of HOV Priority Lane and Return to Passenger Vehicles

If the 3595 persons who use the buses and vanpools in the HOV lane were to switch back to passenger vehicles and use the main lanes at an average speed of 26 mph, the fuel consumption would be

3595 persons ÷ 1.3 persons/vehicle x 0.0628 gal/vehicle x 3 miles x 250 days/year = 130 232 gal, compared with 15 097 gal used in the priority lane.

Table 5 summarizes the results of the example projects.

MAINTENANCE AND RECONSTRUCTION OF SHOULDER USE

Routine maintenance of a shoulder lane can be conducted during off-peak periods when volumes can be accommodated on the main lanes of the freeway without additional delay and operating costs to the motorists. However, if the shoulder must be rebuilt, the capacity of the freeway will be less than the original capacity. The additional energy expended during construction by traffic that is displaced by the reduced capacity can then be estimated.

Consider example A. To reconstruct the shoulder lane, the capacity of the three original lanes would be reduced to 5100 vehicles/h during the construction period. This would result in the diversion of 3600 vehicles to an alternate route. For this analysis, assume that

1. The freeway would revert to the original operating conditions of 20 mph.
2. The 3600 vehicles would use arterial streets with an average speed of 10 mph over a 4-mile trip. The fuel-consumption rate for this traffic is taken from a recent study of arterial street operation [Table 6 (10)].
3. All trucks and buses will stay on the freeway.

The daily fuel consumption prior to the modification of the shoulder lane is calculated as follows:

(0.0772 gal/vehicle mile x 13 386 vehicles + 0.1772 gal/vehicle mile x 276 vehicles + 0.3784 gal/vehicle mile x 138 vehicles) (3 miles) = 3403 gal/day.

The daily fuel consumption during the reconstruction of the lane is calculated as follows:

(0.1108 gal/vehicle mile)(3600 vehicles)(4 miles)
+ (0.0772 gal/vehicle mile x 10 386 vehicles
+ 0.1772 gal/vehicle mile x 276 vehicles + 0.3784 gal/vehicle mile x 138 vehicles)(3 miles) = 4304 gal/day.

Therefore, the reconstruction of the shoulder would cost 901 gal/day.

In examples B and C, the traffic that uses the shoulder is returned to the main lanes of travel during reconstruction. The only deterioration in operation would be a reduction in capacity. For these examples, the bottleneck is further downstream, and thus the freeway operating characteristics in the area would be unaffected.

SUMMARY

The conversion of a freeway shoulder to a travel lane is an immediate and low-cost solution for increasing capacity. The results are higher travel speed, lower total travel times, and reduced fuel consumption.

Problems of shoulder pavement deterioration can be lessened by limiting the use of the lane to passenger vehicles during the peak periods only. The impact of the added capacity on fuel consumption will vary, depending on geometric design, traffic conditions, and use of the shoulder lane.

The reconstruction of a shoulder lane after several years of travel may be necessary. However, the daily fuel consumption during the period of reconstruction should not exceed the amount saved during the use of the shoulder for travel.

REFERENCES

1. W.R. McCasland. Modifying Freeway Geometrics to Increase Capacity. Transportation Engineering Journal, ASCE, Vol. 106, No. TE6, Nov. 1980.
2. W.R. McCasland. The Use of Freeway Shoulders to Increase Capacity. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 210-2, Sept. 1978.
3. W.R. McCasland and R.G. Biggs. Freeway Modifications to Increase Traffic Flow. FHWA, Technology Sharing Rept. FHWA-TS-80-203, Jan. 1980.
4. I-45N Concurrent Flow Shoulder Lane, Initial Findings. Transit Systems Development, Metropolitan Transit Authority of Harris County, TX, Project Development Rept. 81-7, Aug. 1981.
5. A Manual on User Benefit Analysis of Highway and Bus Transit Improvements. AASHTO, Washington, DC, 1977.
6. J.L. Buffington and G.P. Ritch. An Economic and Environmental Analysis Program Using the Results for the FREQ3CP Model. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 210-5, Sept. 1981.
7. J.L. Buffington and W.F. McFarland. Benefit-Cost Analysis: Updated Unit Costs and Procedures. Texas Transportation Institute, Texas A&M Univ., College Station, Res. Rept. 202-2, 1975.
8. P. Claffey. Running Costs of Motor Vehicles as Affected by Road Design and Traffic. NCHRP, Rept. 111, 1971, 97 pp.
9. R. Winfrey. Economic Analysis for Highways. International Textbook Company, Scranton, PA, 1969.
10. J. Raus. A Method for Estimating Fuel Consumption and Vehicle Emissions on Urban Arterials and Networks. Office of Research and Development, FHWA, Rept. FHWA-TS-81-210, April 1981.

Vehicular Fuel-Consumption Maps and Passenger Vehicle Fleet Projections

ALBERTO J. SANTIAGO

The procedures and preliminary results of a study aimed at assessing the fuel-consumption characteristics of passenger vehicles that are representative of the current and near-future fleet in order to update the fuel-consumption models of computerized traffic simulation and optimization programs are presented. The paper identifies 21 engine-drivetrain combinations that are representative of 74 percent of the 1979-1985 passenger vehicle fleet and describes an instrumentation system that permits the collection of the microscopic on-the-road and laboratory test data necessary to fully assess the real-world fuel-consumption characteristics of vehicles.

The problem that this paper discusses is very simple to state: How can we reduce fuel consumption from vehicles operating on a street network? Unfortunately, the answers are quite complex.

Resolving this problem requires a dual approach. First, we need more energy-efficient vehicles; second, we need a means of accurately estimating and predicting fuel consumption from vehicles that operate on a network in order to accurately assess the energy impacts of different traffic-control strategies and roadway designs.

Breakthroughs in technology achieved by automotive engineers have provided the means of manufacturing more energy-efficient vehicles. Today automobiles that average 25-35 miles/gal (10.6-14.8 km/L) are common (1). The problems that still remain for the transportation engineer are how to assess and predict vehicular fuel consumption in any given operating environment and how to enhance roadway designs and traffic-control strategies in order to provide an environment where vehicles can operate more efficiently.

The Federal Highway Administration (FHWA) and others have developed computer programs that evaluate geometric designs and traffic-control strategies (primarily for urban areas) from environmental and energy conservation standpoints. Use of these models by many users has demonstrated their potential as effective tools in the development of traffic engineering measures that reduce motorist operating costs; fuel consumption; costs associated with planning, designing, and implementing new traffic-

Table 1. Engine-drivetrain combinations to be representative of 74 percent of the 1979-1985 passenger vehicle population.

Engine Size (in^3 of displacement)	Cylinders	Transmission[a] and No. of Forward Gears
90	4	A3 and M5
97[b]	4	A3 and M4
105	4	A3 and M5
107	4	A3 and M5
108	4	A3 and M5
140	4	A3 and M4
151	4	A3
156	4	A3
173	6	A3
200	6	A3
229	6	A3
231	6	A3
267	8	A3
302	8	A4
350[b]	8	A3

Note: 1 in^3 = 0.016 L.
[a]A = automatic and M = manual transmission.
[b]Diesel-powered engines.

control strategies; and costly and inconvenient retrofits when problems in a strategy are detected only after implementation.

These computer programs can be categorized into three major groups:

1. Simulation models: Models that simulate the performance of traffic under a given set of geometrics and control strategies (NETSIM, TRAFLO, TRANSYT, and SIGOP).
2. Optimization programs: Programs that optimize traffic signal settings by maximizing through bandwidth or by reducing delay and fuel consumption (SOAP, PASSER, MAXBAND, TRANSYT, and SIGOP). (Note, TRANSYT and SIGOP contain simulation capabilities.)
3. Control programs: Programs that control traffic signal settings based on traffic-flow fluctuations and/or time-of-day operation on a real-time basis (UTCS Enhanced and UTCS Extended).

The simulation models and the optimization programs are further subdivided into macroscopic or microscopic models, depending on the level of detail that the programs simulate traffic. For example, macroscopic programs simulate traffic in platoons or groups, while microscopic models simulate the performance of each vehicle independently.

Although most of these computer programs have been developed sporadically over the past 10-15 years, most of them use fuel-consumption models that represent the fuel-consumption characteristics of the vehicle fleet of the 1960s and/or early 1970s.

The energy crises of 1973-1974 and 1979, in addition to the 1977 Clean Air Act Amendments, triggered changes in the vehicle manufacturing policy. New vehicles are to be smaller, lighter, cleaner, and, most important, more energy efficient. In a short time, these policy changes have made the fuel-consumption models obsolete.

SOLUTION

FHWA, recognizing the need to update such models, is currently sponsoring the study, Fuel Consumption and Emission Values for Traffic Models. The scope of the study is to determine vehicular fuel consumption and emissions for the passenger vehicle fleet exclusively in two phases. Phase 1 is already completed and phase 2 is currently under way.

Phase 1: Current and Near-Future Vehicle Fleet

The main objectives of phase 1 were to define the current and near-future passenger vehicle fleet and to determine the vehicles for which fuel-consumption and emission maps (graphical representations of the relations between variables that effect fuel consumption and emissions) should be developed.

To make the study cost effective, the boundaries of the time frame that define "current" and "near future" had to maximize the period of time for which the fuel-consumption and emission tables would be valid. Based on this constraint, 1979 was selected as the lower boundary and 1985 as the upper boundary for the following reasons:

1. Between 1975 and 1977, vehicle manufacturers responded to the energy crisis of 1973-1974 by manufacturing more fuel-efficient vehicles than those manufactured before 1973. Unfortunately, this manufacturing policy started to fade in 1976 and 1977, and by 1979 most vehicles manufactured in the United States were eight-cylinder-engined vehicles (2).
2. In 1979, a second energy crisis convinced the U.S. vehicle manufacturing industry and the general public of the severity of the energy problem.
3. As of May 1981, none of the manufacturers of foreign or domestic vehicles had developed product plans beyond 1985 (3).

After determining that 1979-1985 would define the current and near-future passenger vehicle population, all the passenger vehicles that were and will be manufactured and sold in the United States (including domestic and foreign) during that time were investigated. These vehicles were defined by engine size (displacement) and engine-drivetrain combination (engine and transmission) instead of by model because of the availability of different engine sizes and transmissions within the same vehicle model.

The 1979-1985 passenger vehicle population was divided into two groups--the 1979-1981 population and the 1982-1985 population. The 1982-1985 population was defined in terms of the 1979-1981 population so that fuel-consumption and emission maps could be developed for the nonexistent vehicles.

From an analysis of the 1979-1981 passenger vehicle population, 57 engine sizes were identified, which ranged from 70 to 368 in^3 (1.1-6.0 L) of displacement. These engines were in 176 vehicle models, which yielded more than 500 engine-drivetrain combinations (transmissions were classified as automatic or manual and by the number of forward gears) (2,4). Of these 500 engine-drivetrain combinations, more than 350 combinations will be available between 1982 and 1985 (3).

It is not cost effective to develop fuel-consumption and emission maps for more than 350 engine-drivetrain combinations, so accurate sales figures were used as weighting factors to determine the most common combinations in the 1979-1985 passenger vehicle population (2,3). (Note, confidential figures were submitted by the vehicle manufacturers to the National Highway Traffic Safety Administration for corporate automobile fuel economy certification.) From an analysis of these data, 21 engine-drivetrain combinations were identified to account for 74 percent of the 1979-1985 passenger vehicle population.

Because of budget limitations, 15 of the 21 engine-drivetrain combinations will be selected for testing in phase 2 (Table 1). In the worst case, where the 15 combinations selected for testing are the "less common", the resulting fuel-consumption maps will be representative of 57 percent of the 1979-1985 passenger vehicle population. In the best case, where the 15 combinations selected are the "most common", the resulting fuel-consumption maps will be representative of 66 percent of the 1979-

1985 passenger vehicle population. Maximizing the percentage of representation will be attempted; however, the final selection of vehicles will depend on their availability from rental and leasing agencies.

The development of fuel-consumption and emission maps representative of the entire automobile fleet requires that individual tables be developed for each category--four-, six-, and eight-cylinder vehicles--and subsequently pooled by using sales figures as weighting factors. The resulting table is then statistically representative of the automobile fleet.

Projections indicated that, if the vehicle manufacturers remain relatively close to the 1981-1985 product plans developed in 1981, the fuel-consumption and emission maps to be developed in phase 2 will be valid, as a minimum, until 1992. This rationale is based on several factors. First, since 15 maps will be developed by using the pooling procedure discussed above, periodic updates could be made without having to develop additional maps; second, projections indicate that prior to 1987, non-gasoline-powered engines will not constitute a significant share of the automobile fleet; and third, as of June 30, 1981, the median age of automobiles has increased from 4.9 to 6.0 years in a decade (5).

Phase 2: Development of Fuel-Consumption and Emission Maps

The objective of this phase is to develop accurate fuel-consumption and emission maps for 15 of the 21 engine-drivetrain combinations specified in phase 1. For simplicity, the development of the fuel-consumption and emission maps will be discussed separately.

Development of Fuel-Consumption Maps

To assess vehicular fuel consumption requires the analysis of three basic systems: the engine, the vehicle (which includes body and engine), and the driver. This paper exclusively discusses the first two systems because the driver (the system that controls the operation of the engine and the vehicle) is simulated by the computerized traffic programs. In other words, the computer programs that simulate traffic take the role of the driver by using car-following algorithms derived from driver behavior. This implies that the fuel-consumption maps to be developed in this study must cover the range of speeds and accelerations that the vehicles are capable of so that fuel consumption can be accurately estimated for any driving cycle.

A requirement of the study is that the fuel-consumption maps to be developed must reflect the fuel consumption of the vehicle as it operates on realistic real-world conditions. This requirement is only accomplished by on-the-road testing.

In the past, research has been oriented toward developing fuel-consumption and emission maps from computer simulations of engine performance or chassis dynamometer testing. These approaches produce accurate maps that describe the performance of the engine exclusively and not the performance of the vehicle as it operates on the road. This, then, justifies developing maps from field experimentation.

The engine and the vehicle must be analyzed as two separate systems because the engine is the element that actually consumes fuel and the vehicle is the medium by which loads are applied to the engine. This analysis encompasses the development of the following two separate data bases that describe fuel consumption from each system:

1. A data base developed through chassis dynamometer tests that relates fuel flow to engine speed [revolutions per minute (RPM)] and manifold vacuum. This data base describes the engine.
2. A road-test data base that relates vehicle speed and acceleration to engine RPM, manifold vacuum, fuel flow, and operating temperatures for each gear. This data base describes the vehicle.

Maps derived from chassis dynamometer testing are extremely useful because fuel-flow rate is identified uniquely by engine RPM and manifold vacuum independent of the secondary effects of temperature and atmospheric pressure on the combustion efficiency itself. The resultant maps are independent of driveline efficiencies, lubricant temperatures, and rolling resistance because any changes in these parameters will change the manifold vacuum pressure at a given RPM.

At low speeds and high accelerations, the combined effects of fuel sloshing in the carburetor bowl and the time lags induced by the filtering effects of the bowl make it impossible to draw any conclusions concerning fuel-flow rate into the engine. On the dynamometer, fuel sloshing effects are negligible, and it is possible to remain at any operating point sufficiently long to overcome the filtering effects of the carburetor bowl.

On-the-road testing of vehicles is necessary to assess vehicular fuel consumption in the vehicles' operating environments. These tests consist of fully instrumenting each vehicle and driving it through combinations of speed and acceleration until sufficient data have been collected to fully characterize the behavior of the vehicle. To obtain statistically valid results, the tests are repeated several times by randomizing the order of the individual runs to prevent any systematic bias in the recorded data.

The instrumentation installed in the vehicles consists of the data logger (which samples and records data), fifth-wheel assembly, inclinometer, electronic tachometer, pressure transducers, electrical thermocouples, fuel-flow meter, and power supply.

The data logger records each data element every 0.05 s on a cassette tape that is later analyzed in a computer. The fifth-wheel assembly measures distance, speed, and acceleration, and the inclinometer measures road grades to correct the acceleration readings to what they would have been on a level road. The electronic tachometer, pressure transducers, and electrical thermocouples measure engine RPM, vacuum pressure, and engine temperature, respectively. The equipment weighs approximately 150 lb (68 kg) and fits in the rear passenger area of a vehicle.

On-the-road tests are being conducted on an airport runway because of the safety advantages over conducting the tests on public roads. The tests require the collection of fuel-consumption data for high speeds and abrupt accelerations, and this might be hazardous if carried out on public roads. In addition, conducting the tests on a smooth, leveled pavement increases the accuracy of the data collected by the fifth-wheel assembly. Bumps, cracks, potholes, and joints in pavements make the fifth wheel bounce, which results in erroneous measurements of speed, acceleration, and distance.

It is important to point out that the resulting maps will be sensitive to ambient temperature. It is not possible to define and test at a single normal operating temperature without imposing some, possibly quite severe, limitations on the applicability of the test results. The use of fuel-consumption maps generated at 70°F (21°C) ambient temperature to calculate fuel use by winter traffic in Minneapolis-St. Paul, for example, could easily result in errors greater than 20 percent. Furthermore, data available in the current literature are not of sufficient detail to permit an analytical correction of the maps for ambient temperature with the necessary confidence.

The approach taken to resolve this problem is to test a vehicle over a bandwidth of ±20°F (±11°C) around a preselected value of 50°F (10°C). The data collected from this vehicle will be used to define an ambient temperature sensitivity curve from which values could be extrapolated for all other vehicles tested. This would result in the development of temperature-sensitive fuel-consumption maps that are representative of the automobile fleet. Preliminary results indicate a reduction of approximately 0.4 percent in fuel consumption per degree centigrade increase. [Note, this is for ambient temperatures around 70°F (21°C). It is expected to vary for different temperatures.]

Subsequently, the data bases generated by the dynamometer testing and the on-the-road testing are merged and run through statistical mapping procedures to produce maps that relate fuel-flow rate to vehicular speed and acceleration for each available gear.

Before the actual development of the maps, the raw data are reformatted by performing the following manipulations in a computerized preprocessor:

1. The data collected are simultized in time. The data logger records each data point sequentially and not simultaneously as desired. This implies that there is discontinuity in the data collected caused by the time lag of the sampling rate. To correct this problem, the data are slightly adjusted and fitted through a least-squares estimated quadratic function of the parameter over time. The result is a continuous function of the parameters as if they would have been collected simultaneously.

2. The acceleration readings are adjusted to what they would have been on a level road by using

Figure 1. Dynamometer fuel-flow observations, neutral gear.

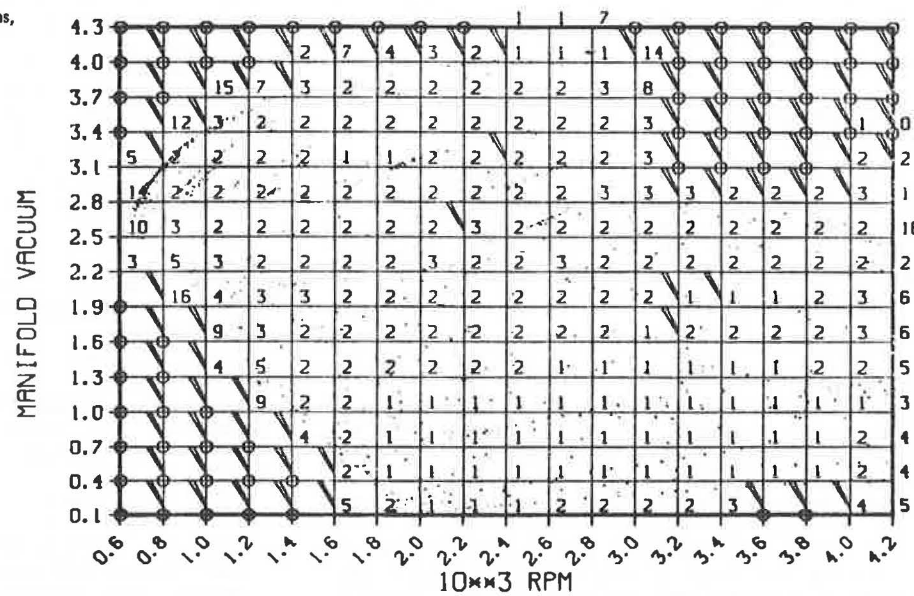
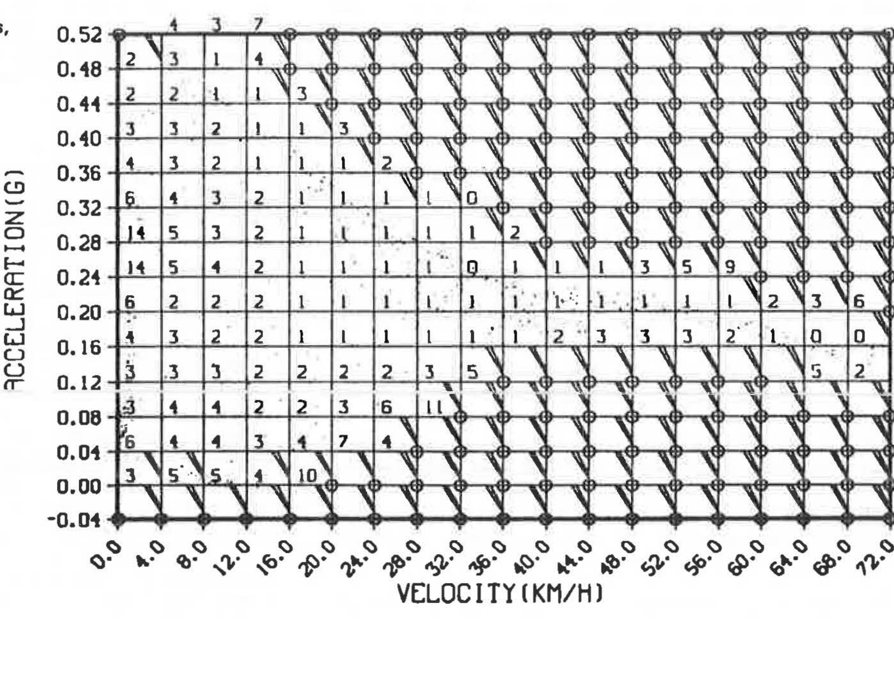


Figure 2. On-the-road fuel-flow observations, first gear.



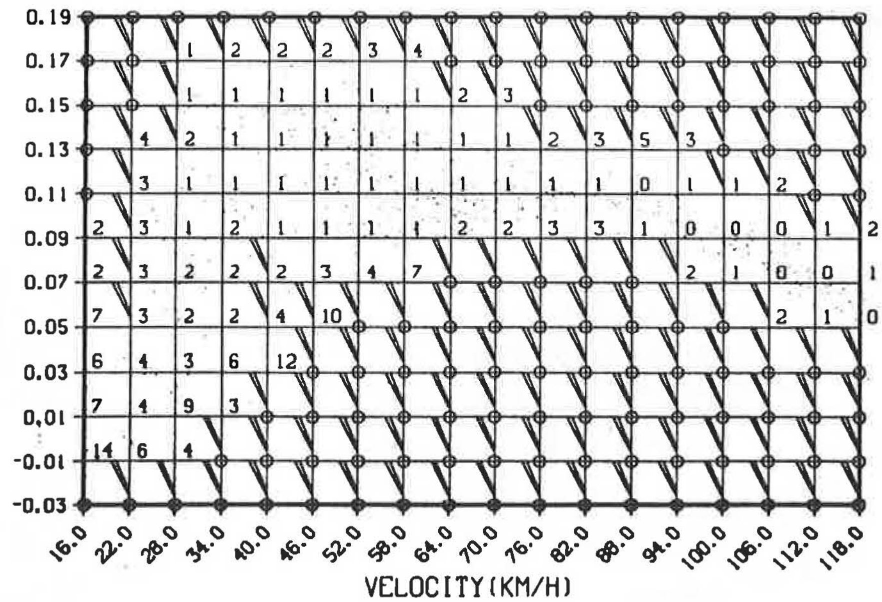


Figure 3. On-the-road fuel-flow observations, second gear.

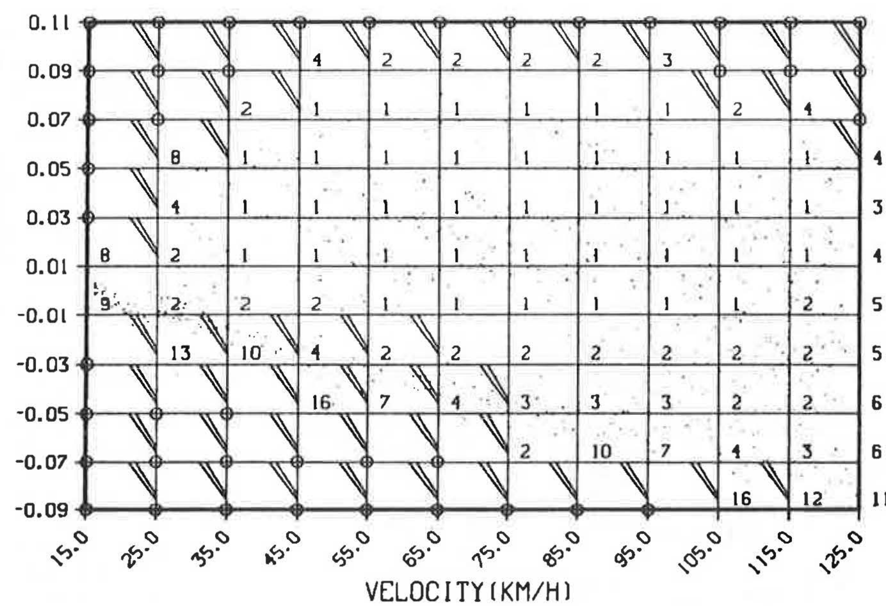


Figure 4. On-the-road fuel-flow observations, third gear.

the inclination readings. If the tests are run on a level road, this step is not necessary, as the inclination reading (total newtonian acceleration) may be substituted for the acceleration.

3. The fuel-flow values derived from dynamometer testing are inserted into the on-the-road data base by using engine speed (RPM) and manifold vacuum as the common parameters between laboratory and field testing.

4. The gears are inserted into each on-the-road observation based on the observed ratios of engine RPM over velocity. For vehicles with automatic transmission, ambiguous ranges of ratios occur that fall into several gears. In these overlap cases, the current and preceding observations of RPM, manifold vacuum, and fuel flow are checked to see if a gear change took place in the time interval between observations.

Following these manipulations, the actual development of the maps can be undertaken. For simplicity, the development of the maps is discussed by using data collected from a 200 CID Ford Fairmont station wagon equipped with automatic transmission. (Note, the following seven figures were derived from data collected from this type of station wagon.)

The laboratory and on-the-road data are placed on a grid with suitable intervals that relate fuel consumption to engine RPM and manifold vacuum, as shown in Figure 1. By merging the laboratory and on-the-road data as previously discussed, fuel consumption is then related to velocity and acceleration by using engine RPM and manifold vacuum as the common parameters between the tests, as shown in Figures 2 through 4, for each available gear.

The scatter points represent the individual observations from which the maps were constructed (789 in gear 1, 686 in gear 2, and 2526 in gear 3). The intersection of the grid lines represent the poles around which a quadratic surface is fitted. The size of the region over which the fit is made depends on the data density and usually has a width of

Figure 5. Fuel use (mL/s), bicubic spline, first gear.

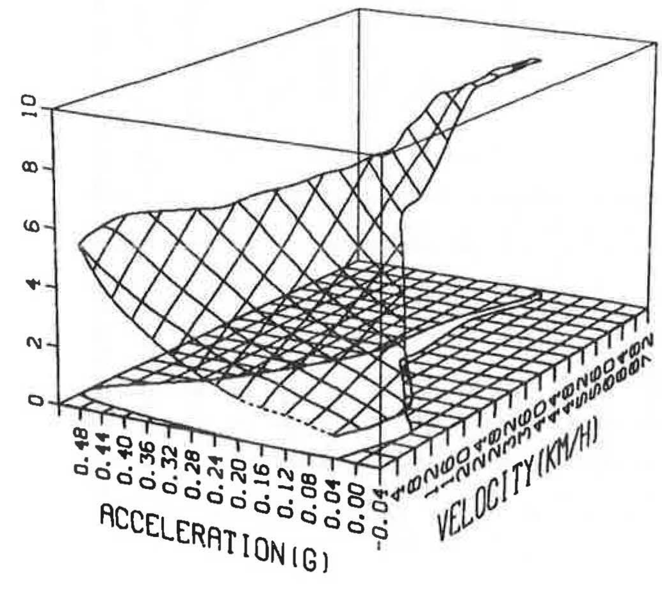
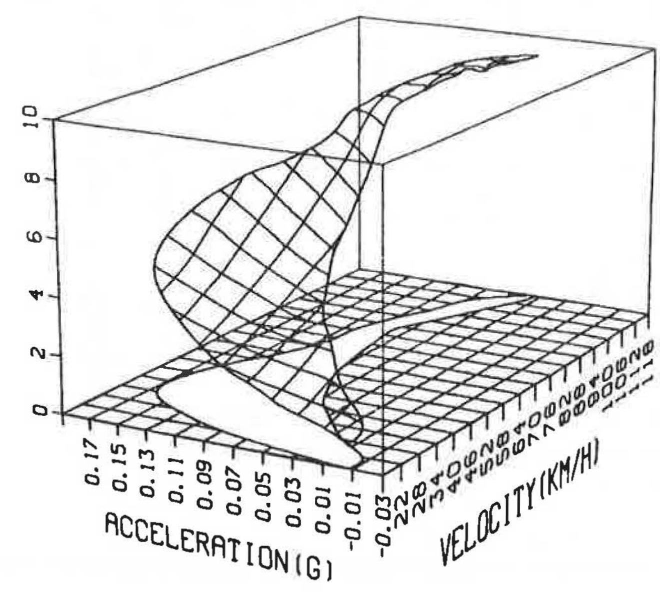


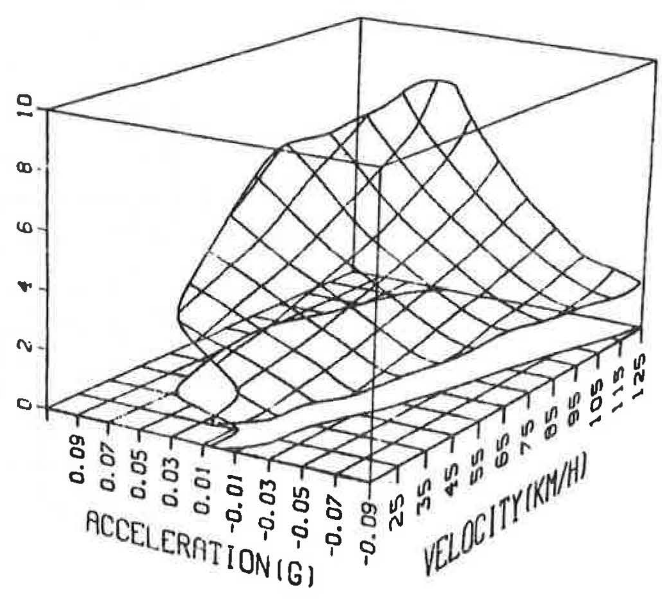
Figure 6. Fuel use (mL/s), bicubic spline, second gear.



2-4 grid squares. The resultant quadratic is used to estimate fuel use at each pole and to estimate the confidence interval for each estimate. The small circles highlight poles that were not estimated either because of lack of data or due to numerical problems (primarily matrix singularities) (6).

The numbers inside the boxes pertain to the pole at the lower left corner of the rectangle and represent a measure of the quality of the fit of the quadratic surface to the underlying data. A 2, for example, indicates that one can have a 90 percent confidence that the mean of the actually observed flow rates is within 2 percent of the predicted value. In the event that the calculated confidence intervals are not within acceptable limits, the grid pattern could be changed in the hope that this will improve the quality of the fit. If changing the grid pattern results in no improvement, then the required procedure is to collect additional data on

Figure 7. Fuel use (mL/s), bicubic spline, third gear.



the dynamometer or test track, as appropriate.

Provided that the quality of the quadratic fits is satisfactory, a bicubic spline surface is run through the poles of the quadratic fits, which results in a smooth surface like that of Figures 5 through 7 for each gear. The diagonal stripes mark rectangles for which satisfactory bicubics could not be derived. If one or more of the poles that surround a striped rectangle is encircled, no bicubic could be estimated, since an underlying quadratic was missing. Otherwise, a bicubic was estimated but failed to come within 1 percent (3 percent for the dynamometer maps) of the underlying surface with 90 percent confidence. Failure of the 1 percent level test, on occasion, has been caused by a quadratic that differs slightly from the others that surround it instead of a poor bicubic fit. Again, this problem would be resolved by changing the grid size or collecting additional data, although a more cost-effective procedure is to extend one of the adjacent bicubics to cover the poorly fitting area. The step of calculating the bicubics is required because the quadratic surfaces were estimated independently, therefore introducing discontinuity at their junctions. Aside from the bicubics fitting the dynamometer data, most bicubics that fail the 1 percent tolerance unit test pass it at 2 or 3 percent tolerance.

Development of Emission Maps

The relatively long response times of current emission analyzers coupled with their bulk, weight, and high power requirements relegate any testing program to one of steady-state determinations on a chassis dynamometer. In this study, it is anticipated that test procedures for hydrocarbon and carbon-monoxide emissions can be developed that are analogous to and compatible with those used for the fuel-consumption tests. That is, if each engine operating point, as defined by engine RPM and manifold vacuum, determines the emission rates of the vehicle (subject to verification through this study), then emission maps can be developed on the dynamometer that can be linked to on-the-road performance through measurements of the determining factors in an on-the-road test in a manner similar to the methodology used in developing the fuel-consumption maps.

SUMMARY AND RECOMMENDATIONS

From a transportation perspective, the scenario for the 1980s is restricted by energy and environment concerns. The era of abundant energy supplies is past, and reducing air pollution is imperative. It is time to work with greater commitment and urgency toward implementing environmentally safe and energy-efficient solutions.

In the United States, it has been estimated that all modes of highway transportation account for 74 percent of the total transportation energy and 45 percent of all U.S. fuel consumption (7). In addition, it has been estimated that highway transportation accounts for 50 percent of the total annual emissions of air pollutants such as carbon monoxide, hydrocarbons, nitrogen oxides, sulfur oxides, and particulates (8). These statistics dramatically demonstrate the seriousness of the energy and environmental problems as related to highway transportation.

Future research should be oriented toward developing fuel-consumption and emission maps for trucks and buses in a manner similar to the one described in this paper. Also, efforts should be directed toward developing feasible roadway design practices that would provide an operating environment where vehicles could operate efficiently.

To cope with environmental and energy problems, major traffic engineering actions, which require accurate analysis tools, must be planned and pursued aggressively over many years. The successful completion of this study will update and improve the capabilities of the traffic models in accurately estimating fuel consumption and emissions from passenger vehicles that operate in a street network. This enhancement will provide the traffic engineering community with powerful tools for developing, testing, and evaluating traffic-control strategies in addition to determining the environmental and energy impacts of such strategies.

REFERENCES

1. 1980 Gas Mileage Guide, 2nd ed. U.S. Department of Energy, Oak Ridge, TN, Feb. 1980.
2. Mid-Model Year Reports. National Highway Traffic Safety Administration, U.S. Department of Transportation, 1979-1981.
3. 1982-1985 Vehicle Manufacturers Product Plans. National Highway Traffic Safety Administration, U.S. Department of Transportation, 1981.
4. Pre-Model Year Reports. National Highway Traffic Safety Administration, U.S. Department of Transportation, 1980-1981.
5. Cars in Operation Growing Older. Automotive News, April 1982.
6. A.B. Rose, J.N. Hooker, and G.F. Roberts. A Data-Based Simulator for Predicting Vehicle Fuel Consumption. SAE, Paper 820302, 1982.
7. F.A. Wagner. Energy Impacts of Urban Transportation Improvements. ITE, Washington, DC, Aug. 1980.
8. A.M. Ertugul and H.R. Hammond. Fundamentals of Air Quality. FHWA, Rept. FHWA-IP-76-5, Feb. 1977. NTIS: PB 292788.

Effect of Freeway Work Zones on Fuel Consumption

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The objective of this study was to investigate the effect of freeway work zones on fuel consumption. The development of a procedure for estimating the excess fuel consumption caused by lane closures on 3-, 4-, and 5-lane freeway sections is presented. The procedure is applicable to both undersaturated and oversaturated traffic-flow conditions. Tables and graphs designed to facilitate the implementation of the procedure are included. An example that illustrates the application of the procedure is also presented.

The excess fuel consumption associated with the movement of traffic through work zones is a major factor in the increased operating expense to the highway user. In recent years, there has been a shift of priorities at all levels of government from building new highway facilities to upgrading the existing highway system. At the same time, the public has become increasingly aware of the need to conserve energy due to the increasing costs associated with that energy. In light of these facts, the prudent engineer must consider the effect of work zones on fuel consumption.

A development of user costs associated with construction activities was presented by Graham and others (1) in a Federal Highway Administration (FHWA) report completed in June 1977. Formulas were developed from curve fits, which resulted in equations for excess fuel consumed as a function of average daily traffic (ADT) for various combinations of lane-closure configurations and schedules. Although the report provided useful information on fuel consumption in a general sense, no method for computing the fuel use for site-specific lane-closure schedules and hourly volumes was presented.

The purpose of the study presented in this paper was to investigate the impact of freeway work zones on fuel consumption. This impact was evaluated for the following lane-closure situations:

1. Two unidirectional lanes reduced to one lane,
2. Three unidirectional lanes reduced to two lanes,
3. Three unidirectional lanes reduced to one lane,
4. Four unidirectional lanes reduced to three lanes,
5. Four unidirectional lanes reduced to two lanes, and
6. Five unidirectional lanes reduced to two lanes.

This paper presents the development of a procedure for estimating the excess fuel consumption caused by these freeway work zones during both undersaturated and oversaturated traffic flow conditions. Tables and graphs designed to facilitate the calculation of these estimates are included, and an example that illustrates the application of the procedure is presented.

UNDERSATURATED CONDITIONS

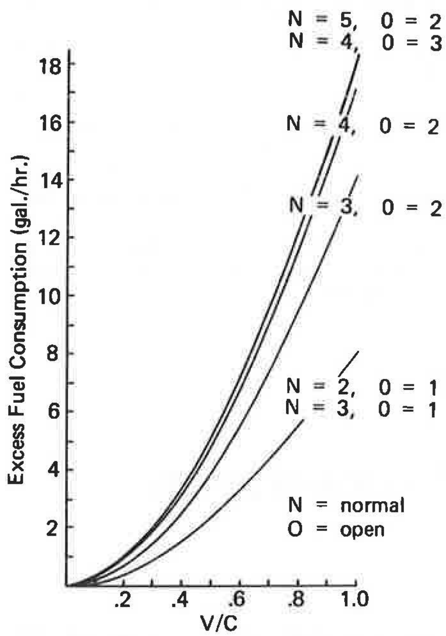
During time periods when the volume-capacity ratio

Table 1. Excess gallons of gasoline consumed per slowdown speed-change cycle—passenger car.

Speed (mph)	Excess Gasoline Consumed (gal) by Amount of Speed Reduction Before Accelerating Back to Speed (mph)					
	10	20	30	40	50	60
20	0.0032	-	-	-	-	-
30	0.0035	0.0062	-	-	-	-
40	0.0038	0.0068	0.0093	-	-	-
50	0.0042	0.0074	0.0106	0.0140	-	-
60	0.0046	0.0082	0.0120	0.0155	0.0190	-
70	0.0051	0.0090	0.0130	0.0167	0.0203	0.0243

Note: Data derived from Claffey (4) for the composite passenger car representative of the following vehicle distribution: 20 percent large cars, 65 percent standard cars, 10 percent compact cars, and 5 percent small cars.

Figure 1. Excess fuel consumption due to speed-change cycles—undersaturated conditions.



(V/C) of the work zone is less than one, there are two major factors that have an effect on the amount of fuel consumed. These two factors are (a) an increased fuel consumption due to speed-change cycles and (b) a decreased fuel consumption as a result of vehicles traversing the work zone at a reduced speed. The algebraic sum of the effects of these two factors is the total excess fuel consumed during undersaturated conditions. In many cases, particularly for longer work zones, this sum will be negative, which indicates a net fuel savings that results from freeway work zones during undersaturated traffic flow conditions.

Effect of Speed-Change Cycles

Due to the reduced capacity of a multilane facility when one or more lanes are closed for construction or maintenance, the operating speed of vehicles in the affected section is decreased. The combined effect of decelerating to and accelerating from this reduced speed is a net increase in fuel consumed when compared with the consumption at a constant speed through the normal section.

If the V/C of a work zone is known, an estimate of the operating speeds can be obtained from idealized speed-volume relations. Studies have shown that the speed-volume relation for uninterrupted flow conditions on multilane highways can be reasonably represented by a straight line (2). This line extends from the average highway speed at V/C equal to zero down to 30 mph at V/C equal to one. From this linear relation, the operating speed on any multilane highway is computed as follows:

$$OS = AHS - (V/C)(AHS - 30) \quad (1)$$

where OS is the operating speed (mph) and AHS is the average highway speed (mph).

At a given volume, the difference between the operating speed obtained with the normal capacity of the roadway and that obtained with the reduced capacity due to a lane closure is the amount of speed reduction caused by the lane closure. Capacities through freeway work zones for the six lane-closure situations considered in this study were measured by Dudek and Richards (3). These capacities are given in the table below (3):

No. of Lanes		Avg Capacity	
Normal	Open	Vehicles per Hour	Vehicles per Hour per Lane
2	1	1340	1340
3	2	3000	1500
3	1	1130	1130
4	3	4560	1520
4	2	2960	1480
5	2	2740	1370

These capacities were used in this study to compute speed reductions caused by the lane closures as follows.

For example, suppose the normal capacity of a two-lane section of freeway is 4000 vehicles/h and the average speed on the facility is 55 mph. If one lane is closed, the capacity in the work zone is reduced to 1340 vehicles/h (from the table above). If the hourly volume on the section is 800 vehicles/h, the V/C for the normal and reduced capacity conditions would be computed as follows:

$$V/C_{normal} = 800/4000 = 0.20 \quad (2)$$

$$V/C_{reduced} = 8/1340 = 0.60 \quad (3)$$

From Equation 1, the operating speeds for these two conditions would be 50 mph in the normal section and 40 mph in the reduced section. Thus, the amount of the speed reduction caused by closing one lane of the two-lane section of freeway would be 10 mph.

The excess fuel consumed as a result of a slowdown cycle is a function of not only the amount of speed reduction but also the operating speed before and after the reduction. In the example above, this speed would be 50 mph. Table 1, which was derived from data presented by Claffey (4), shows the gallons of excess fuel consumed per speed-change cycle by passenger cars. From this table, the amount of excess fuel consumed per passenger car can be determined for any combination of speed reduction and normal operating speed. Then, multiplying the value obtained from Table 1 by the hourly volume gives the excess fuel consumed per hour by passenger car speed-change cycles.

Based on a normal freeway capacity of 2000 vehicles/h/lane and the work-zone capacities shown in the in-text table above, Figure 1 shows the relation between excess fuel consumption per hour due to speed-change cycles and the work-zone V/C for the six lane-closure situations. By using this figure, the excess fuel used per hour can be determined directly. For the example given above, the excess

Figure 2. Fuel savings due to speed reduction—undersaturated conditions.

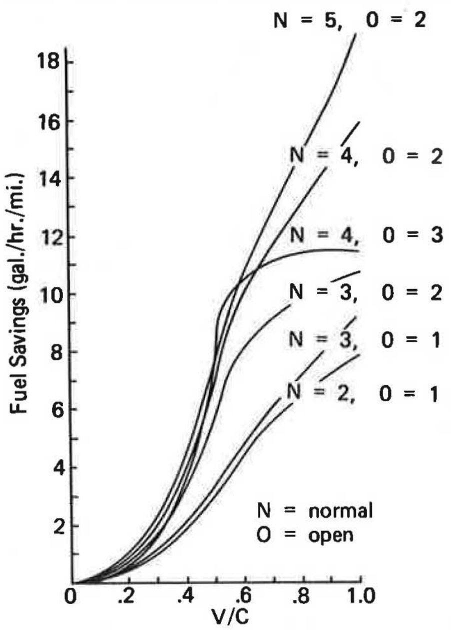


Table 2. Truck factors for fuel consumption due to speed-change cycles—undersaturated conditions.

V/C	Truck Factors at Following Percentages 0	10	20	30	40
0.1	1.00	1.75	2.50	2.90	3.28
0.2	1.00	1.74	2.48	2.85	3.22
0.3	1.00	1.71	2.42	2.80	3.16
0.4	1.00	1.70	2.40	2.75	3.10
0.5	1.00	1.68	2.36	2.70	3.04
0.6	1.00	1.66	2.32	2.65	2.98
0.7	1.00	1.65	2.30	2.65	2.98
0.8	1.00	1.63	2.26	2.60	2.92
0.9	1.00	1.61	2.22	2.55	2.86
1.0	1.00	1.58	2.16	2.45	2.74

fuel consumed per hour when a two-lane freeway is reduced to one lane with a V/C of 0.6 is 3.4 gal/h.

Effect of Reduced Operating Speeds

At speeds greater than 30 mph, the rate of fuel consumed by passenger cars increases with operating speed, as shown in the table below, which was derived from data presented by Claffey (4) (note, data are for a composite passenger car representative of the following vehicle distribution: 20 percent large cars, 65 percent standard cars, 10 percent compact cars, and 5 percent small cars):

Uniform Speed (mph)	Fuel Consumption (gal/mile)
10	0.072
20	0.050
30	0.044
40	0.046
50	0.052
60	0.058
70	0.067

Thus, the reduced operating speeds associated with e lower capacities of freeway work zones will se a fuel savings. The difference between the

Table 3. Truck factors for fuel consumption as affected by speed—undersaturated conditions.

V/C	Truck Factors at Following Percentages 0	10	20	30	40
0.1	1.00	1.52	2.03	2.56	3.08
0.2	1.00	1.51	2.02	2.53	3.04
0.3	1.00	1.48	1.96	2.44	2.92
0.4	1.00	1.31	1.62	1.93	2.24
0.5	1.00	1.34	1.68	2.02	2.36
0.6	1.00	1.28	1.56	1.84	2.12
0.7	1.00	1.28	1.56	1.84	2.12
0.8	1.00	1.28	1.56	1.84	2.12
0.9	1.00	1.28	1.56	1.84	2.12
1.0	1.00	1.28	1.56	1.84	2.12

fuel-consumption rate in gallons per mile shown in the table above for the normal freeway operating speed and that shown for the reduced work-zone speed multiplied by the traffic volume in vehicles per hour represents the amount of this fuel savings in gallons per hour per mile of work zone. This fuel saving over the range of work-zone V/C for the six lane-closure situations is shown in Figure 2.

Truck Fuel-Consumption Factors

To include the fuel consumption of trucks in the analysis of the effect of freeway work zones on fuel consumption, the factors presented in Tables 2 and 3 must be applied to the values obtained from Figures 1 and 2, respectively. The composite truck represented by the factors is an average of counts taken at four rural Interstate locations in Nebraska (5). The distribution of these trucks is as follows: 30.4 percent pickup and panel trucks, 8.2 percent two-axle six-tire trucks, and 61.4 percent tractor semitrailer truck combinations, of which 80 percent contain diesel engines. These percentages were divided into weight distributions so that the consumption tables presented by Claffey (4) could be used to calculate the fuel consumed by the composite truck.

The truck fuel-consumption factors in Tables 2 and 3 were computed by comparing the amount of fuel consumed with each percentage of trucks in the traffic stream to the amount of fuel consumed without any trucks in the traffic stream. The amount of fuel consumed per speed-change cycle for each combination of truck percentage and V/C was divided by the amount of fuel consumed per speed-change cycle for the corresponding V/C without trucks to obtain the factors in Table 2. Likewise, the amount of fuel consumed per hour per mile for each combination of truck percentage and V/C was divided by the amount of fuel consumed per hour per mile for the corresponding V/C without trucks to obtain the factors shown in Table 3. Thus, to adjust for trucks, an excess fuel-consumption value obtained from Figure 1 is multiplied by the appropriate factor in Table 2 and a fuel savings value obtained from Figure 2 is multiplied by the appropriate factor in Table 3.

OVERSATURATED CONDITIONS

As the demand volumes through a work zone increase, the V/C of the section also increases. Assuming uniform arrivals, a queue begins to form when V/C exceeds one. At this point, the energy consumption increases dramatically due to two factors: (a) idling time in the queue and (b) additional speed-change cycles experienced by vehicles coming to a complete stop. Of course, as in the case of under-

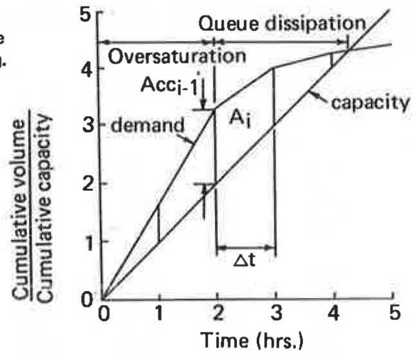


Figure 3. Graphical representation of queue forming and dissipating.

saturated conditions, there is a decrease in fuel consumption as a result of vehicles traversing the work zone at lower speeds. However, unlike under-saturated conditions, the algebraic sum of the effects of these three factors is nearly always positive, which indicates a net excess fuel consumption that results from freeway work zones during oversaturated traffic flow conditions.

Effect of Idling

Figure 3 is a plot of the cumulative V/C versus time for a hypothetical pattern of demand during over-saturation in a work zone. The area between the demand and the capacity V/C curves, multiplied by the capacity of the work zone, represents the total number of vehicle hours of idling time. The area between the demand and capacity V/C curves for any constant time increment during oversaturation when demand V/C is greater than one is as follows:

$$A_i = [(1/2)(V/C_i - 1)\Delta t + ACC_{i-1}]\Delta t \tag{4}$$

where

- A_i = area between the demand and capacity curves during the ith time increment (hours²);
- V/C_i = volume-to-capacity ratio during the ith time increment;
- ACC_i = accumulation of demand over capacity during the ith time increment, which is equal to $(V/C_i - 1)\ \Delta t + ACC_{i-1}$ (hours); and
- Δt = constant time increment (hours).

At the end of periods of oversaturation, when the demand drops below the capacity (i.e., demand V/C is less than one), the queue will dissipate. The area between the demand and capacity V/C curves for any constant time increment during the dissipation of the queue is

$$A_i = [ACC_{i-1} - (1/2)(1 - V/C_i)\Delta t]\Delta t \tag{5}$$

The instant that the queue has dissipated does not necessarily occur at the end of a constant time increment. Therefore, the final triangular area between the demand and capacity V/C curves when the queue is dissipating is

$$A_n = [(1/2)(ACC_{n-1})^2]/(1 - V/C_n) \tag{6}$$

where n is the total number of hours during a period of oversaturation and queue dissipation divided by the constant time increment Δt plus one for any remainder. One is not added if there is no remainder.

The total idling time caused by oversaturation is as follows:

$$T = C \sum_{i=1}^{n} A_i \tag{7}$$

where T is the total idling time (vehicle-h) and C is the capacity of the work zone (vehicles/h).

Thus, the fuel consumed by vehicles idling in a queue is

$$F_I = Tg_i \tag{8}$$

where F_I is the fuel consumed by vehicles idling in queue (gal) and g_i is the fuel-consumption rate for i percent trucks (gal/h). Fuel-consumption rates for idling vehicles are presented in the table below. These values were derived from data presented by Claffey (4):

Trucks (%)	Fuel Consumed (gal/h)
0	0.58
10	0.57
20	0.57
30	0.56
40	0.56

Effect of Speed-Change Cycles During Queuing

During periods of oversaturation, speed-change cycles have a greater impact on fuel consumption because the vehicles must come to a complete stop as opposed to just reducing their operating speed. In reality, a vehicle must reduce its speed from the normal operating speed to zero, accelerate from zero to 30 mph (i.e., the operating speed in the work zone at capacity), and then accelerate from 30 mph back to the previous normal operating speed. This effect can be approximated by one speed-change cycle from normal operating speed to a complete stop.

The excess fuel consumed as a result of the stop-go cycles is calculated by the same method as for the speed-change cycles in the undersaturated case, except that the reduced speed is always zero. During a given time increment, the operating speed in the section without a lane closure can be found by using the idealized speed-volume relation previously discussed. The excess fuel consumed during a stop-go speed-change cycle at this operating speed can be determined from the table below, which was derived from data presented by Claffey (4) (note, data are for the composite passenger car representative of the following vehicle distribution: 20 percent large cars, 65 percent standard cars, 10 percent compact cars, and 5 percent small cars):

Speed (mph)	Fuel Consumed (gal)
10	0.0016
20	0.0066
30	0.0097
40	0.0128
50	0.0168
60	0.0208
70	0.0243

The values from this table multiplied by the hourly volume during the time increment provide the excess fuel consumed per time increment due to stop-go cycles caused by the presence of a queue that results from oversaturation during the time increment, as follows:

$$f_{si} = g_{si} V_i \Delta t \tag{9}$$

where

- f_{si} = excess fuel consumed due to stop-go speed-change cycles during ith time increment (gal),

Figure 4. Excess fuel consumption due to speed-change cycles during oversaturation.

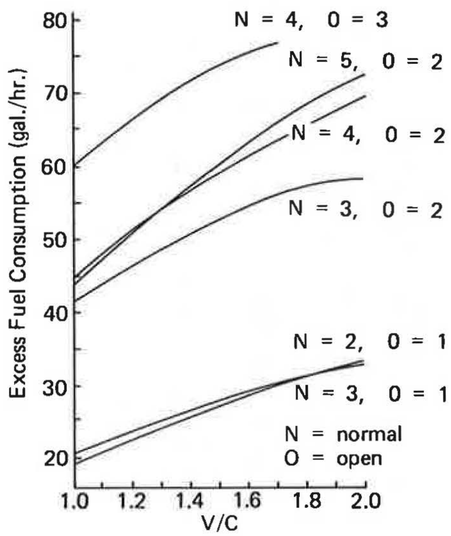
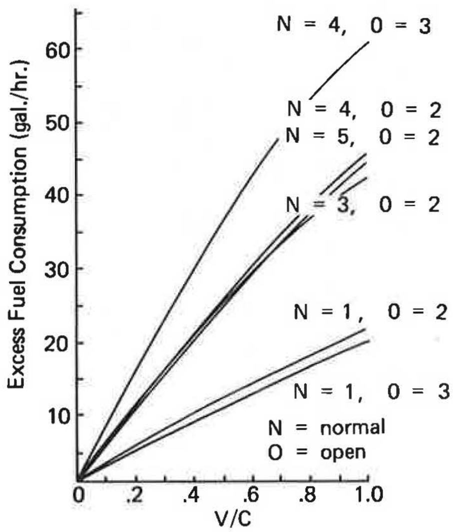


Figure 5. Excess fuel consumption due to speed-change cycles during queue dissipation.



g_{si} = excess fuel consumed per stop-go speed-cycle change during ith time increment (gal), and

V_i = hourly flow rate during ith time increment (vehicles/h).

Figure 4 shows the excess fuel consumed per hour by stop-go speed-change cycles during oversaturation on the six lane-closure situations considered in this study. Figure 5 shows the excess fuel consumed per hour by stop-go speed-change cycles while the queue caused by oversaturation is dissipating.

As discussed in the previous section, the instant that the queue has dissipated does not necessarily occur at the end of a constant time increment. Therefore, the excess fuel consumed due to stop-go speed-change cycles during the final time increment is as follows:

$$f_{sn} = g_{sn} V_n \Delta t \; [ACC_{n-1}/(1 - V/C_n)] \qquad (10)$$

where n is the total number of hours during a period of oversaturation and queue dissipation divided by the constant time increment Δt plus one for any remainder. One is not added if there is no remainder.

Thus, the total excess fuel consumed due to stop-go speed-change cycles during a period of oversaturation and queue dissipation is

$$F_s = \sum_{i=1}^{n} f_{si} \qquad (11)$$

where F_s is the excess fuel consumed due to stop-go speed-change cycles during period oversaturation and queue dissipation (gal).

Effect of Reduced Speed Operation During Queuing

It is assumed that the speed of operation through a construction work zone when a queue is present is 30 mph. This means that the fuel savings realized as a result of a lower operating speed can be determined by the same method as for the undersaturated case, except that the reduced speed is always 30 mph.

For a given volume, the operating speed in the section with all lanes open is calculated and the fuel-consumption rate per mile is read from the in-text table in the section on Effect of Reduced Operating Speed. The fuel-consumption rate per mile at 30 mph is subtracted from this value to find the fuel saved per vehicle per mile of the work zone. Figure 6 shows this fuel savings during oversaturation on the six lane-closure situations considered in this study. Figure 7 shows this fuel savings while the queue caused by oversaturation is dissipating.

Truck Fuel-Consumption Factors

The truck adjustment factors found in Tables 4 and 5 were developed and applied in the same manner as the tables for undersaturated conditions. The V/C during a given time increment, along with the percentage of trucks, is used to determine the factor to be applied to the fuel consumption for the composite car. Table 4 contains adjustment factors for fuel consumption due to stop-go speed-change cycles, and Table 5 contains the adjustment factors for the change in consumption due to the reduced speed in a work zone.

EXAMPLE PROBLEM

The following example illustrates the procedure used to determine excess fuel consumption. The volumes used were taken from actual data collected at a continuous traffic count location in the eastbound lane of Interstate 80 east of 42nd Street in Omaha, Nebraska (5). This section is a tangent three-lane roadway with high-type pavement.

Table 6 is a summary of the computations that were done to estimate the excess fuel consumption due to the closing of one lane for maintenance work. The length of the lane closure is 1 mile.

Columns 1 and 2 give the hour of the day and its respective volume. The V/C's of the section with all lanes open and with one lane closed are shown in columns 3 and 4. These were obtained by dividing the volume from column 2 by the capacities for this lane-closure condition (data from in-text table in section on Effect of Speed-Change Cycles).

To compute the values in column 5, column 4 is scanned to locate V/C's greater than 1.0. In this example, there are two periods of oversaturation: 7:00-9:00 a.m. and 4:00-5:00 p.m. Columns 5 and 6 can be computed by using equations for oversaturated conditions. For example, Equation 5 gives the area value for the period between 9:00 and 10:00 a.m., i.e.,

Figure 6. Fuel savings due to speed reduction during oversaturation.

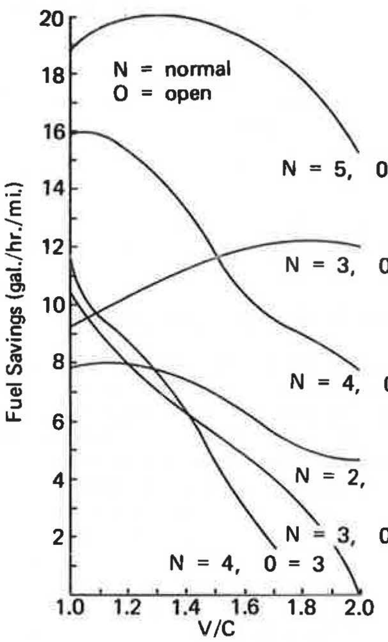
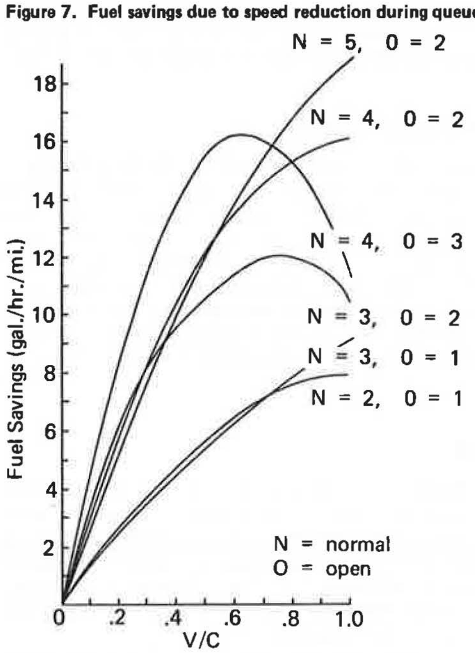


Figure 7. Fuel savings due to speed reduction during queue dissipation.



$$A = [0.39 - (1/2)(1 - 0.70)(1)] = 0.24\ h^2 \quad (12)$$

The accumulator value is as follows:

$$ACC = (0.70 - 1.0)(1) + 0.39 = 0.09\ h \quad (13)$$

During the period between 10:00 and 11:00 a.m., the accumulator value for the previous hour (0.09) is less than the V/C subtracted from 1.0 (0.35). This means that the queue is dissipating during this hour. The area value for this hour is calculated by using Equation 6:

$$A = [(1/2)(0.09)^2]/(1 - 0.65) = 0.01\ h^2 \quad (14)$$

The remainder of column 5 is computed in the same manner.

Table 4. Truck factors for fuel consumption due to stop-go speed-change cycles—oversaturated conditions.

	Truck Factors at Following Percentages						
V/C	0	5	10	15	20	30	40
0.1	1.00	1.25	1.50	1.75	2.00	2.50	3.00
0.2	1.00	1.25	1.50	1.75	2.00	2.50	3.00
0.3	1.00	1.25	1.49	1.74	1.98	2.47	2.96
0.4	1.00	1.24	1.48	1.72	1.96	2.44	2.92
0.5	1.00	1.24	1.48	1.72	1.96	2.44	2.92
0.6	1.00	1.24	1.47	1.71	1.94	2.41	2.88
0.7	1.00	1.23	1.46	1.69	1.92	2.38	2.84
0.8	1.00	1.23	1.46	1.69	1.92	2.38	2.84
0.9	1.00	1.23	1.46	1.69	1.92	2.38	2.84
1.0	1.00	1.23	1.45	1.68	1.90	2.35	2.80
1.1	1.00	1.22	1.43	1.65	1.86	2.29	2.72
1.2	1.00	1.22	1.43	1.65	1.86	2.29	2.72
1.3	1.00	1.21	1.42	1.63	1.84	2.26	2.68
1.4	1.00	1.21	1.41	1.62	1.82	2.23	2.64
1.5	1.00	1.20	1.40	1.60	1.80	2.20	2.60
1.6	1.00	1.20	1.40	1.60	1.80	2.20	2.60
1.7	1.00	1.20	1.40	1.60	1.80	2.20	2.60
1.8	1.00	1.20	1.39	1.59	1.78	2.17	2.56
1.9	1.00	1.19	1.38	1.57	1.78	2.14	2.56
2.0	1.00	1.19	1.38	1.57	1.76	2.14	2.52

Table 5. Truck factors for fuel consumption as affected by speed—oversaturated conditions.

	Truck Factors at Following Percentages						
V/C	0	5	10	15	20	30	40
0.1	1.00	1.17	1.33	1.50	1.66	1.99	2.32
0.2	1.00	1.16	1.32	1.48	1.64	1.96	2.28
0.3	1.00	1.16	1.32	1.48	1.64	1.96	2.28
0.4	1.00	1.16	1.31	1.47	1.62	1.93	2.24
0.5	1.00	1.15	1.30	1.45	1.60	1.90	2.20
0.6	1.00	1.15	1.29	1.44	1.58	1.87	2.16
0.7	1.00	1.15	1.29	1.44	1.58	1.87	2.16
0.8	1.00	1.15	1.29	1.44	1.58	1.87	2.16
0.9	1.00	1.15	1.29	1.44	1.58	1.87	2.16
1.0	1.00	1.14	1.28	1.42	1.56	1.84	2.12
1.1	1.00	1.13	1.26	1.39	1.52	1.78	2.04
1.2	1.00	1.16	1.31	1.47	1.62	1.93	2.24
1.3	1.00	1.11	1.21	1.32	1.42	1.63	1.84
1.4	1.00	1.14	1.27	1.41	1.54	1.81	2.08
1.5	1.00	1.09	1.18	1.27	1.36	1.54	1.72
1.6	1.00	1.13	1.26	1.39	1.52	1.78	2.04
1.7	1.00	1.09	1.17	1.26	1.34	1.51	1.68
1.8	1.00	1.12	1.23	1.35	1.46	1.69	1.92
1.9	1.00	1.15	1.29	1.44	1.58	1.87	2.16
2.0	1.00	1.16	1.31	1.47	1.62	1.93	2.24

Column 7 is obtained directly from the appropriate figure by using the V/C from column 4. If no queue is present, Figure 1 is used. If a queue is present, a value will appear in column 5, and Figure 4 or 5 should be used instead.

At the end of the queuing period, the consumption value for column 7 is computed as the weighted average of the values obtained from the undersaturated and the oversaturated cases. The weighting factor (wf) for the hour from 10:00 to 11:00 a.m. would be

$$wf = 0.09/(1 - 0.65) = 0.275 \quad (15)$$

By using this weighting factor, the value for column 7 is obtained from the appropriate values found in Figures 1 and 5 at a V/C of 0.65, i.e.,

$$f_{sn} = 0.257 \times 30.3 + (1 - 0.257) \times 6.5 = 12.6 \quad (16)$$

Column 8 is obtained by using the procedure outlined above and Figures 2, 6, and 7.

The excess fuel consumption for the entire day is

Table 6. Example problem computations.

(1) Hour	(2) Volume (vehicles/h)	(3) Normal V/C	(4) Work-Zone V/C	(5) A (h^2)	(6) ACC (h)	(7) Fuel Excess (gal/h)	(8) Fuel Savings (gal/h)
12:00-1:00 a.m.	580	0.10	0.19	0	0	0.6	0.6
1:00-2:00 a.m.	334	0.06	0.11	0	0	0.2	0.2
2:00-3:00 a.m.	201	0.03	0.07	0	0	0.1	0.1
3:00-4:00 a.m.	169	0.03	0.06	0	0	0.1	0
4:00-5:00 a.m.	234	0.04	0.08	0	0	0.1	0.1
5:00-6:00 a.m.	467	0.08	0.16	0	0	0.4	0.5
6:00-7:00 a.m.	1845	0.31	0.62	0	0	5.9	8.2
7:00-8:00 a.m.	4173	0.70	1.39	0.20	0.39	50.2	6.4
8:00-9:00 a.m.	2997	0.50	1.00	0.39	0.39	41.4	10.5
9:00-10:00 a.m.	2091	0.35	0.70	0.24	0.09	32.1	12.0
10:00-11:00 a.m.	1950	0.33	0.65	0.01	0	12.6	9.4
11:00-12:00 a.m.	2071	0.35	0.69	0	0	7.2	8.9
12:00-1:00 p.m.	2022	0.34	0.67	0	0	6.9	8.7
1:00-2:00 p.m.	2200	0.37	0.73	0	0	7.9	9.2
2:00-3:00 p.m.	2308	0.38	0.77	0	0	8.9	9.5
3:00-4:00 p.m.	2763	0.46	0.92	-	-	12.2	10.3
4:00-5:00 p.m.	3094	0.52	1.03	0.02	0.03	21.1	10.0
5:00-6:00 p.m.	2635	0.44	0.88	0	0	10.5	10.6
6:00-7:00 p.m.	1998	0.33	0.67	0	0	6.9	8.7
7:00-8:00 p.m.	1692	0.28	0.56	0	0	4.9	7.3
8:00-9:00 p.m.	1305	0.22	0.44	0	0	3.1	4.2
9:00-10:00 p.m.	1225	0.20	0.41	0	0	2.7	3.5
10:00-11:00 p.m.	1043	0.17	0.35	0	0	2.0	2.5
11:00-12:00 p.m.	878	0.15	0.29	0	0	1.4	1.6
Total				0.86		239.4	143.0

obtained from the sum of columns 5, 7, and 8. The sum of column 8 is multiplied by the capacity of the work zone and the idling consumption rate found in Table 5. Thus, the excess fuel consumption due to idling is

$$0.84\ h^2 \times (3000\ \text{vehicles/h}) \times (0.58\ \text{gal/h}) = 1462\ \text{gal} \quad (17)$$

The sum of column 7 gives the amount of excess fuel consumed due to speed-change cycles. This value is 239.4 gal. The decrease in fuel consumption caused by reduced-speed operation through the work zone (143.0 gal) is given by the sum of column 8.

The combined effect of all factors results in a net increase in fuel consumption of

$$1462\ \text{gal} + 239\ \text{gal} - 143\ \text{gal} = 1558\ \text{gal} \quad (18)$$

CONCLUSIONS

The procedure developed and demonstrated in this paper can be used in planning and scheduling freeway work zones to estimate the effect of lane closures on fuel consumption. The graphs presented can be used to facilitate the application of the procedure to the following lane-closure situations:

1. Two unidirectional lanes reduced to one lane,
2. Three unidirectional lanes reduced to two lanes,
3. Three unidirectional lanes reduced to one lane,
4. Four unidirectional lanes reduced to three lanes,
5. Four unidirectional lanes reduced to two lanes, and
6. Five unidirectional lanes reduced to two lanes.

However, in using the tables and graphs presented in this paper, it should be noted that they are based on composite vehicles derived from specific vehicle distributions and data presented by Claffey (4). Therefore, if it is determined that these composite vehicles are not acceptable for a particular situation, appropriate adjustment factors should be applied to the values obtained from these tables and figures when using the procedure.

REFERENCES

1. J.L. Graham and others. Accident and Speed Studies in Construction Zones. FHWA, Rept. FHWA-RD-77-80, 1977, pp. D-1 to D-32.
2. Highway Capacity Manual. HRB, Special Rept. 87, 1965, 397 pp.
3. C.L. Dudek and S.H. Richards. Traffic Capacity Through Urban Freeway Work Zones in Texas. Texas Transportation Institute, Texas A&M Univ., College Station, 1982.
4. P.J. Claffey. Running Costs of Motor Vehicles as Affected by Road Design and Traffic. NCHRP, Rept. 111, 1971, pp. 16-30.
5. Continuous Traffic Count Data and Traffic Characteristics on Nebraska Streets and Highways. Nebraska Department of Roads, Lincoln, 1981.

Fuel Consumption Related to Roadway Characteristics

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In 1979, the Federal Highway Administration contracted the Texas Research and Development Foundation to prepare an updated set of vehicle operating cost tables for use in the Highway Performance Monitoring Program. Included in this research was a reinvestigation of the interrelations between roadway characteristics and fuel consumption, which required the performance of a set of experiments to investigate the effect of grade, curvature and surface type, and pavement condition on fuel consumption. These experiments were conducted in 1980 and 1981 by using a set of eight vehicles, which ranged from a small economy car to a 2-S2 semitruck. The tests were conducted while the vehicles were idling, accelerating, decelerating, and traveling at constant speed. The idle fuel-consumption test showed that new vehicles consumed fuel at a higher rate than previously had been published. Acceleration and deceleration models were generated, which allowed a direct analysis of speed-change cycles for all driving situations. A new set of constant- speed fuel-consumption tables as a function of grade were generated and are presented in this paper. It was found during this research that pavement condition did not affect fuel economy over the range of conditions normally encountered in the United States. This resulted from testing on asphalt concrete pavements with a range in serviceability of 1.8-4.2 and testing on concrete and surface-treated pavements. This is a very significant finding in the economic and energy analysis of highway transportation systems, since it removes the fuel-based incentive for providing smooth pavements on highways.

Existing literature (1,2) shows a strong relation between vehicle operating cost and the roadway characteristics of grade, curvature, and roughness. Due to the dramatic increase in vehicle operating costs in the past decade, states that use these as inputs to the process of planning the construction, reconstruction, and maintenance of roadways found vehicle operating costs were a major influence on the selection process. In 1979, the Federal Highway Administration (FHWA) sponsored the Texas Research and Development Foundation to perform research on the relations between vehicle operating costs and roadway characteristics. This research included fuel-consumption measurements and a detailed analysis of oil consumption, tire wear, maintenance and repair, depreciation, accident rates, and vehicle emissions (3).

TEST VEHICLES

The fuel-consumption tests were performed with eight vehicles with characteristics representative of the general vehicle population, as described in Table 1. Four automobiles were included in the fleet: an economy car, two midsized cars, and a large luxury car.

The test fleet included two midsized cars so that the variance of the two identical automobiles could be used in the statistical analysis for significance factors. However, since the statistical tests on the effect of surface type showed no significance when tested with the repeat variance on tests, it was not necessary to use the variance between the repeat vehicles.

Four trucks were also tested: a pickup, a two-axle single-unit truck (2A-SU), a three-axle single-unit truck (3A-SU), and a four-axle semi (2-S2). All trucks had a minimum of 20 000 miles at the start of the test. All tests with the trucks, except the pickup, were run in the loaded condition. A load was selected that was typical for the model of truck being tested. In some cases, the typical weight for the truck tested was not representative of the vehicle class weight. In order to have a common vehicle weight basis for the operating cost tables, it was necessary to extrapolate the fuel-consumption data to different weight classes.

It was also necessary to extrapolate the fuel-consumption data collected with the 2-S2 to estimate fuel consumption for the six-axle semi (3-S2). Data from previous studies were used to make this extrapolation. Although extrapolating data is not a desirable situation, steps were taken to minimize the amount of extrapolation. This included the use of a test weight for the 2-S2 vehicle that was only 6500 lb less than the typical loaded weight for a 3-S2.

TEST SECTIONS

Test sections were selected to be homogenous with respect to grade, surface type, and roughness. The test section properties are summarized in Table 2. Grades were determined from as-constructed plan sheets on file at the Texas State Department of Highways and Public Transportation. Roughness was measured by Austin Testing Engineers by using a Maysmeter calibrated against the Texas calibration sections in the Austin, Texas, area.

A total of 12 test sections were used during the experiments. Tests to determine the effect of curvature were performed on a large parking lot with the economy car, the large car, and the pickup. The parking-lot owners prohibited further testing, fearing the test would damage the pavement. No other acceptable area was found to continue these tests with the other vehicles.

After the tests had been completed with three of the vehicles, two sections with more desirable characteristics were located. Test sections used with each vehicle are shown in Table 2. Section 10 was used to replace section 6, since the serviceability index of section 10 was closer to the middle of the range of serviceability. Section 11 replaced section 7, since the new section had a surface treatment in relatively good condition and would allow a direct comparison of the influence of surface type on fuel consumption.

EQUIPMENT

A Fluidyne 1214F fuel meter (4), a Lamar System fifth wheel (from Lamar Instruments, Redondo Beach, California), and two digital recorders were selected from the available hardware. The digital recorders were mounted in a black box, which also contained counters for the distance and fuel measurements, a crystal clock, and a microprocessor for data management. The recording unit had 12 thumbwheels that could be used to identify the test run and select the sample time interval. After initial testing, a sampling interval of 2 s was selected. An inverter was used to power the recording unit.

The fifth wheel had a design resolution of 50 counts/ft. However, careful tests of the distance measurements indicated that only 36 counts/ft were recorded. Furthermore, the number of counts per foot seemed to vary with speed and the tire-surface interaction. A gas spring of the type used on hatch-back doors of automobiles was mounted on a fabricated bracket to try to eliminate wheel bounce problems, but this did not correct the problem. Thus, the distance data were not as reliable as desired.

For the constant-speed tests, the vehicle speedometer was used to determine speed. Radar was used

Table 1. Test fleet characteristics.

Characteristic	1980 Ford Escort	1980[a] Ford Fairmont	1979 Oldsmobile Delta 88	1980 Ford Pickup	2A-SU GMC	3A-SU GMC (Brigadier)	2-S2 Freightliner
Road weight (lb)	2412	3006	4350	3678	17 120	35 870	56 000
Curb weight (lb)	2112	2706	4050	3378	10 720	15 760	24 680
Engine displacement (in^3)	98	200	350	350	366	426	855
No. of cylinders	4	6	8	8	8	6	6
Fuel type	Unleaded	Unleaded	Unleaded	Unleaded	Leaded	Diesel	Diesel
Frontal area (ft^2)	20.7	22.1	29.2	31.0	38.7	57.3	95.7
Transmission	Manual	Automatic	Automatic	Automatic	Manual	Manual	Manual
No. of forward gears	4	3	3	3	5	8	9
Body style	Station wagon	Sedan	Sedan	Box	Van	Dump	Flatbed
Options							
Air conditioning	Yes	Yes	Yes	Yes	No	Yes	Yes
Power steering	No	Yes	Yes	Yes	No	Yes	Yes
Power brakes	No	Yes	Yes	Yes	No	Yes	Yes
Steel-belted radials	Yes	Yes	Yes	Yes	No	No	No
Fuel consumption[b] (miles/gal)							
City cycle	28	18	-	16	NA	NA	NA
Highway cycle	44	24	-	18	NA	NA	NA
Combined	32	20	16	17	NA	NA	NA
Test vehicle number	1	2	4	5	6	7	8
Vehicle category	Small car	Medium car	Large car	Pickup truck	2A-SU truck	3A-SU truck	2-S2 semi

Note: NA = not applicable.

[a]Two vehicles with these characteristics were used.

[b]Fuel-consumption data from U.S. Environmental Protection Agency (EPA).

Table 2. Test section characteristics.

Location	Section No.	Surface Type[a]	Grade (%)	Length (mile)	Serviceability Index	Vehicles Tested[b]
US-281	1	AC	2.6	0.5	4.2	All
SH-71	2	AC	0	0.4	4.4	All
US-281	3	AC	5.6	0.4	4.5	All
FM-2222	4	AC	11	0.4	4.0	All
Old Highway 20	5	AC	~0	0.8	1.5	All
FM-973	6	AC	~0	0.8	3.8	1, 3, 4
FM-973	7	AC	~0	0.5	3.7	1, 3, 4
Burger Center	8	AC	~0	-[c]	1.5	1, 3, 4
I-10	9	PCC	~0	2.1	3.4	All
Littig Road	10	AC	~0	0.5	3.2	2, 5, 6, 7, 8
Hays County	11	ST	~0	0.6	3.5	2, 5, 6, 7, 8
CC-229	12	Gravel	~0	0.6	1.8	All

[a]Surface types are as follows: AC = asphalt concrete, PCC = portland cement concrete, and ST = surface treatment.
[b]See vehicle numbers in Table 1.
[c]Constant-speed-cycle tests.

to test the accuracy of the speedometer. Data from the fifth wheel were reliable enough to establish that the test was performed at constant speed. By reviewing these distance measurements, it was possible to eliminate test runs when there was a speed change.

In the acceleration and deceleration tests, it was necessary to use the distance data from the fifth wheel. In this case, the best estimate between distance recordings and actual distance traveled was used to establish the relation between speed and fuel consumption during acceleration and deceleration. Thus, these relations have an inherent source of unquantified error. Although this is an undesirable situation, it could not be avoided or altered with the resources available. Due to the lack of any better data source, the fuel-consumption relations for acceleration and deceleration seemed reasonable and were useful on this project.

A wood box was fabricated to hold the fuel meter. The meter was mounted to the front bumper of the vehicles with a bicycle rack, as shown in Figure 1. Quick-connects were used to attach the fuel lines so that the fuel meter could be removed when not in use. Mounting the fuel meter on the front bumper disrupts the aerodynamic design of the vehicle and hence alters fuel consumption. However, this was a constant factor in all tests, so it did not influence the effects investigated in this research.

METHODOLOGY

The first test performed after the fuel meter was installed in a vehicle was to measure idle fuel consumption. With gasoline vehicles, an exhaust analyzer was used to measure hydrocarbons and nitric oxide. This measure showed that all the vehicles were properly tuned.

The vehicles were driven a minimum of 12 miles to the test section. The equipment was installed, and the vehicle was idled for a minimum of 3 min while air temperature and wind were measured. Acceleration, deceleration, and coast-down tests were then performed. Finally, the constant-speed tests were performed.

Constant-speed tests were performed in the sequence 10, 30, 50, 70, 20, 40, and 60 mph in each direction. The sequence was repeated three times. Occasionally, traffic would require aborting a test before the end of the section. These occurrences were noted in a field book and were subsequently screened during data processing.

Vehicles with automatic transmissions were tested in Drive. The driver used his or her discretion for selecting the gear for manual shift vehicles. The gear used was always recorded in the log book.

By using the automatic data-recording box, one technician could both drive and operate the equipment for all vehicles except the two heaviest

Figure 1. Fuel meter mounted on vehicle.

trucks. Professional truck drivers were hired to operate these trucks while the technician operated the equipment.

RESULTS OF FUEL-CONSUMPTION EXPERIMENTS

Experiments were performed to determine the effect of speed, grade, pavement type, and roughness on fuel consumption at constant speed. In addition, fuel consumption was measured during idling, acceleration, and deceleration.

Fuel Consumption While Idling

Average fuel consumption per minute was calculated from the idling tests performed before each test session on the various sections. These values were converted to gallons per hour and are summarized in the table below (note, fuel consumption for the 3-S2 is assumed):

Vehicle	Fuel Consumption (gal/h)
Small cars	0.271
Medium cars	0.563
Large cars	0.563
Pickups	0.756
2A-SU	1.198
3A-SU	0.398
2-S2	0.470
3-S2	0.470

The fuel-consumption rates while idling, especially for automobiles, are substantially higher than the values reported by Winfrey (2). This is attributed to emission reduction equipment on the automobiles. Modern cars have a much higher idling speed than the vehicles used for the idling consumption rates reported by Winfrey.

Fuel Consumption During Acceleration and Deceleration

Fuel consumption was measured as the vehicles accelerated from a stop to 70 mph or the top speed of the vehicle and then decelerated back to a stop. These tests were started with the third vehicle, so no acceleration and deceleration data were collected for the heavy car and pickup truck.

The typical graphs of the raw data for a medium-class car during acceleration and deceleration tests are shown in Figures 2 and 3, respectively. The fuel-consumption tests during acceleration start with a low number of distance counts per time interval, or speed; as the number of counts per foot increases, the fuel consumption per unit time increases. Fuel consumption during deceleration (Figure 3) starts at a high speed, i.e., many distance counts per time interval, and then reduces to zero. The variance in Figures 2 and 3 is the result of repeated tests.

Acceleration Fuel-Consumption Models

Careful review of the acceleration data showed a linear relation between fuel consumption and speed in all cases. Due to time constraints, it was decided to generate these equations by visual inspection of the data. For the automobiles and pickup truck, linear equations, passing through the origin, did a good job of modeling the data. However, this type of model was not adequate for the trucks because it underestimated the fuel consumption at low speeds and overestimated consumption at high speeds. For trucks, a maximum fuel-flow rate during acceleration was identified at approximately 45 mph. In addition, a minimum fuel-flow rate was identified at low speeds. These equations are summarized in Table 3. The fuel-rate equations were integrated with respect to speed to obtain equations for estimating fuel consumption during acceleration.

The procedure used to generate the acceleration portion of fuel consumption for speed-change cycles was to calculate the volume of fuel required for each 5-mph increase in speed and then sum the appropriate values for acceleration phases of more than 5 mph. An example of this calculation procedure is shown in Table 4.

Acceleration Rate Models

A nonuniform model was used for calculating time and distance during acceleration (5). In the nonuniform acceleration model, the acceleration varies as a linear function of speed; that is,

$$\mathrm{ACCEL} = A - B(V) \tag{1}$$

where

ACCEL = acceleration at velocity V (ft/s^2),
A,B = constants, and
V = speed (ft/s).

By using this formulation, the time to change from speed V_o to V_1 is

$$t = \{\ln[A - B(V_1)] - \ln[A - B(V_o)]\}/-B \tag{2}$$

where t equals time (s).

The distance traveled in feet (X) over the time interval t from initial speed V_o can be expressed as follows:

$$X = (A/B)t - (A/B^2)(1 - e^{-Bt}) + (V_o/B)(1 - e^{-Bt}) \tag{3}$$

Thus, to quantify this model, only the two coefficients A and B need to be determined. Due to the formulation of this model, A represents the maximum acceleration and A/B is the maximum speed attainable. The values of A and B selected as representative of the vehicle classes used in this report are given below:

Vehicle	Coefficient A	Coefficient B
Automobile		
Small	7.2	0.060
Medium	8.60	0.076
Large	7.9	0.055
Truck		
Pickup	7.9	0.08
2A-SU	2.8	0.026
3A-SU	1.8	0.016
2-S2	1.8	0.016
3-S2	1.8	0.016

Figure 2. Fuel consumption of medium car during acceleration (raw data).

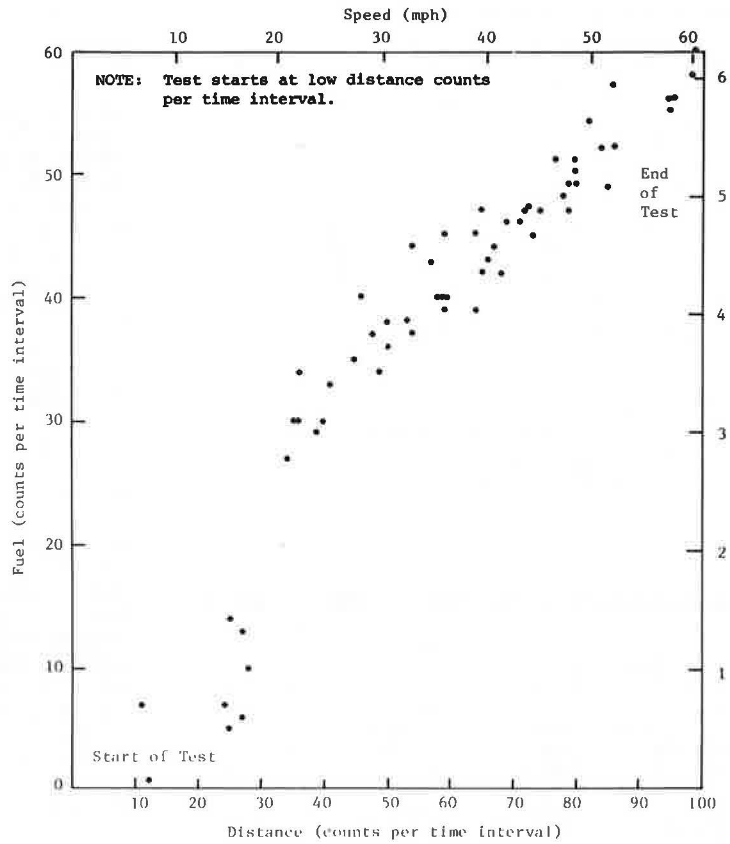

Figure 3. Fuel consumption of medium car during deceleration (raw data).

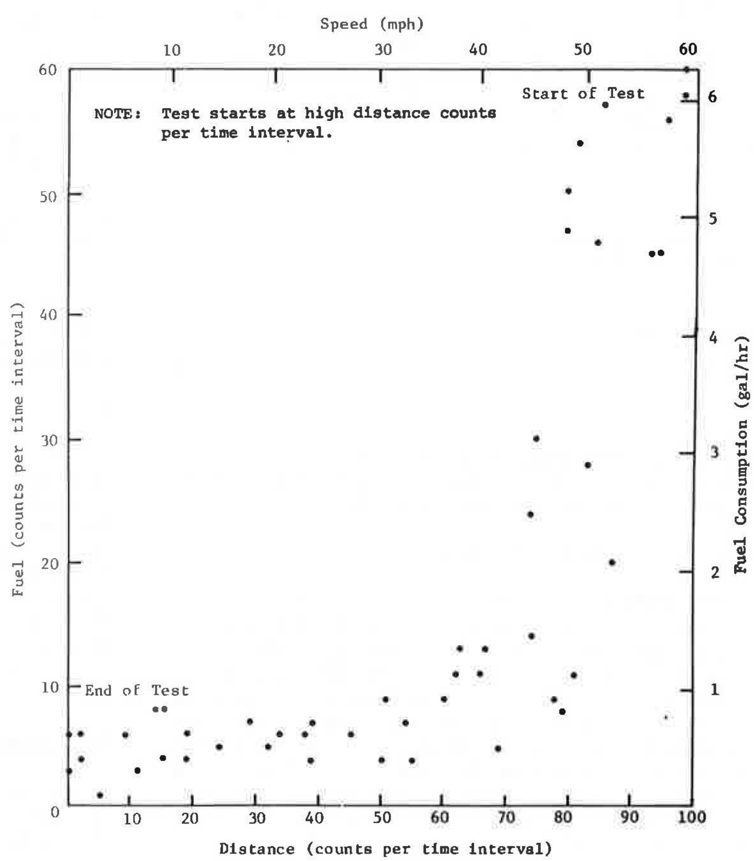

Table 3. Acceleration fuel-consumption models.

Vehicle	Coefficient A	Coefficient B	Maximum Speed (mph)	Maximum Fuel Rate (gal/h)
Small car	0.0	0.062	-	-
Medium car	0.0	0.102	-	-
Large car[a]	0.0	0.136	-	-
Pickup[a]	0.0	0.136	-	-
2A-SU	1.34	0.260	45	13.0
3A-SU	2.07	0.263	45	13.9
2-S2	6.20	0.180	45	14.6
3-S2[a]	6.80	0.240	45	17.6

Note: FR = A + BV; if V < max speed, FT = At + Bs; if V > max speed, FT = (FR max)t

where

FR = fuel rate (gal/h),
V = speed (mph),
FT = total fuel for acceleration (gal) obtained by integrating the equation for fuel rate,
t = time for acceleration (h), and
s = distance for acceleration (mile).

[a]Assumed.

Table 4. Example of fuel-consumption calculations for acceleration, large car.

Start Speed (mph)	Time for Acceleration: 5 mph (h) (t)[a]	Distance for Acceleration: 5 mph (mile) (s)[b]	Fuel for 5-mph Acceleration[c] (gal)	Cumulative Fuel (gal)
0	0.000 26	0.000 66	0.000 09	0.000 09
5	0.000 28	0.002 11	0.000 29	0.000 38
10	0.000 29	0.003 68	0.000 50	0.000 88
15	0.000 31	0.005 39	0.000 75	0.001 63
20	0.000 34	0.007 57	0.001 03	0.002 66
25	0.000 36	0.009 86	0.001 34	0.004 00
30	0.000 39	0.012 56	0.001 71	0.005 71
35	0.000 42	0.015 75	0.002 13	0.007 84
40	0.000 45	0.019 38	0.002 64	0.010 48
45	0.000 50	0.023 77	0.003 24	0.013 72
50	0.000 55	0.029 19	0.003 97	0.017 69
55	0.000 63	0.035 97	0.004 89	0.022 58
60	0.000 71	0.044 66	0.006 07	0.028 65
65	0.000 83	0.056 12	0.007 63	0.036 28

[a]$t = \{\ln[A - B(V_1C)] - \ln[A - B(V_oC)]\}/3600(-B)$

[b]$s = [(A/B)t - (A/B^2)(1 - e^{-Bt}) + (V_oC/B)(1 - e^{-Bt})]/5280$

where

A = 7.9,
B = 0.055,
C = 1.467,
V_o = initial speed (mph), and
V_1 = end speed (mph).

[c]FT = At + Bs

where

FT = total fuel (gal),
A = 0.0, and
B = 0.136.

The acceleration rates used in this project for automobiles are compared with the rates recommended by St. John and Kobett (6) in Figure 4.

Deceleration Fuel-Consumption Models

The fuel-consumption tests during deceleration showed that, for the automobiles, there was about a 6-s lag between the time the driver started deceleration and when the fuel consumption reached a steady-state condition. This may be attributed to dash pots (or vacuum-actuated switches) that are used on modern carburetors to keep the throttle from closing rapidly to reduce hydrocarbons. After this phase, fuel consumption during deceleration reached a steady-state condition. The two phases are clearly shown in Figure 3.

A two-step function was used to model the fuel data for deceleration. One step covers the transition phase of deceleration while the throttle is closing, as shown in Figure 3. Even though the data on this figure indicate that the fuel consumption during the transition phase is a function of speed, such a conclusion based on these data would be incorrect. The throttle is regulated such that it closes at a constant time rate that is not dependent on speed. It should be noted that the fuel supply to a diesel motor is completely shut off, as shown on Figure 5, whenever there is negative horsepower, such as during deceleration or on negative grades.

The models generated by analyzing the plots of the fuel data during deceleration are given in Table 5. In using these models to generate the speed-change fuel-consumption tables, the transition phase model was used for the first 6 s for automobiles and the first 3 s for trucks. The remainder of the time during deceleration was modeled with the steady-state models.

Deceleration Rate Models

A uniform deceleration model was chosen for braking for two primary reasons. First, sliding friction is theoretically independent of the relative speed of the surfaces in contact. The second reason is more pragmatical, in that it is difficult to quantify a typical braking pattern for the population of vehicles on the road. Much of the existing research in the area has quantified braking performance into levels of constant deceleration. The constant deceleration model may be expressed as follows:

$$D = dV/dt \tag{4}$$

where D equals deceleration (ft/s^2) and dV/dt equals change in speed with time. The time to change from speed V_o to V_1 is

$$t = (V_o - V_1)/D \tag{5}$$

where

t = time (s),
V_o = initial speed (ft/s), and
V_1 = final speed (ft/s).

The distance traveled in feet (X) for changing from speed V_o to V_1 is

$$X = [1/(2D)](V_o^2 - V_1^2) \tag{6}$$

The distance traveled over time interval t from the initial speed V_o is

$$X = V_o t - (1/2)Dt^2 \tag{7}$$

In the above formulations, the deceleration has been expressed as a positive (+) quantity, i.e., negative acceleration equals positive deceleration. Based on information reported by Claffey (1) on normal deceleration rates used by drivers in traffic, a two-level deceleration model was used. For decelerations at speeds less than 30 mph, a 5.0-mph/s (7.33-ft/s^2) rate was used. For initial speeds greater than 30 mph, a 3.3-mph/s (4.84-ft/s^2) rate was used. These rates were used for all vehicle classes.

Fuel Consumption at Constant Speed

Three parameters were studied in the constant-speed fuel-consumption experiments: the effects of speed, grade, and pavement type and surface condition. The major emphasis in the fuel consumption was placed on the constant-speed experiments.

Figure 4. Comparison of acceleration rates of St. John and Kobett to rates developed in this study for cars.

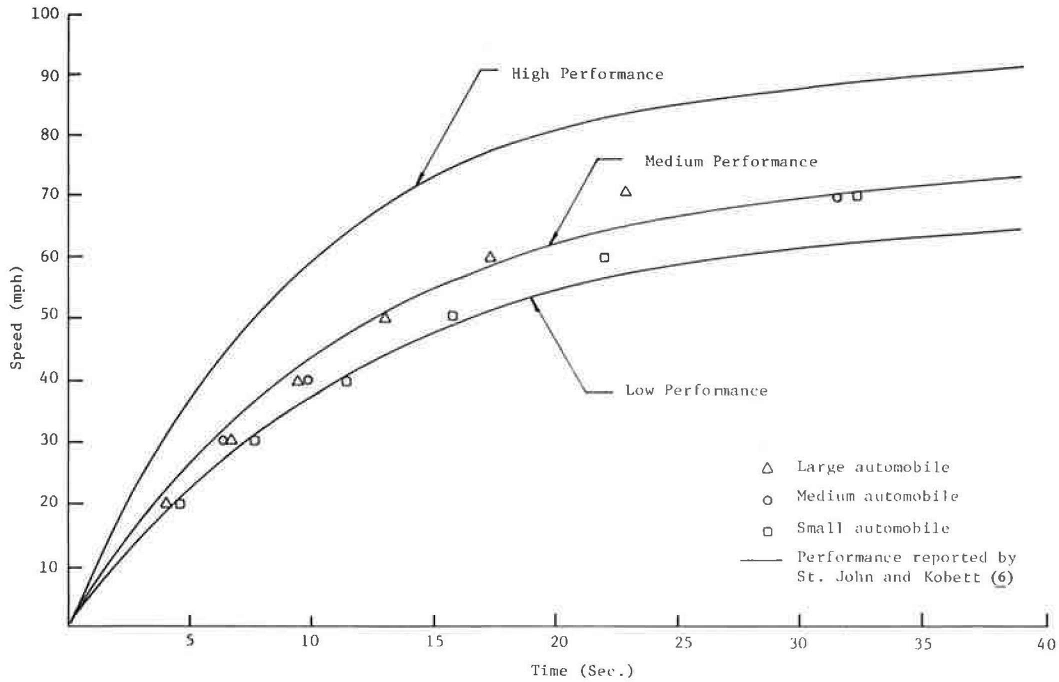

Figure 5. Fuel consumption of diesel motor during deceleration.

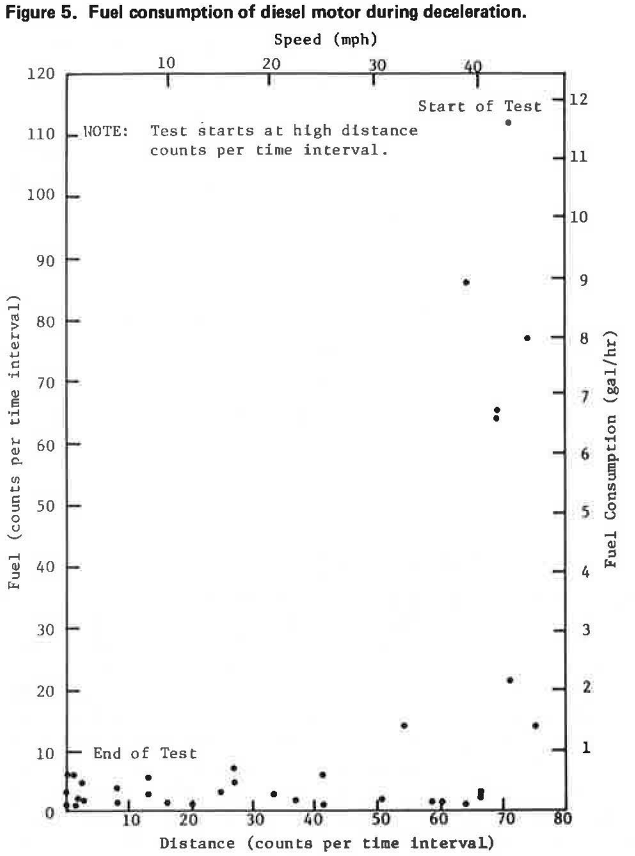

Table 5. Deceleration fuel-consumption model.

	Coefficient			Coefficient	
Item	C_1	C_2	Item	C_1	C_2
Small car	0.52	2.07	2A-SU	1.45	7.23
Medium car	0.72	3.62	3A-SU	0	7.23
Large car	0.93[a]	4.13[a]	2-S2	0	7.23
Pickup	0.93[a]	4.13[a]	3-S2	0	7.23[a]

Note: $f = [C_2 t_2 + C_1 (t - t_2)]/3600$

where

f = fuel consumption (gal);
C_2 = fuel consumption during initial deceleration;
t_2 = time of initial deceleration (s): for automobiles and pickups t_2 = min (6,t), and for other trucks t_2 = min (3, t);
C_1 = fuel consumption during stable deceleration, and
t = time of deceleration.

If $(t - t_2) < 0$, $(t - t_2) = 0$.

[a] Assumed.

Due to the fact that the fuel experiments with the 2A-SU and the 2-S2 were not tested at the typical vehicle weights, it was necessary to adjust the measured fuel consumption to typical fuel-consumption rates for the vehicle population. In addition, it was necessary to extrapolate the data from the 2-S2 experimental vehicle to the 3-S2. The best source of data for making these adjustments was developed by France (7) in a direct study of truck fuel economy on a dynomometer.

France tested several trucks at different test weights. One of these vehicles had the same type of motor as the 2-S2 used in this research. Graphs of fuel economy versus weight at each speed were plotted from France's data. These plots were then entered with the test weight and the desired typical weight for this research to determine fuel consumption at these weights for each speed. The ratio of the fuel consumption at the typical weight to the fuel consumption at the test weight was multiplied by the fuel consumption measured in this research to obtain the fuel consumption for the typical vehicles. The data used for these calculations are summarized in Table 6.

Effect of Speed and Grade on Fuel Consumption

Plots of fuel consumption versus grade and speed were generated from the data. These graphs were

Table 6. Fuel adjustment factors and consumption rates for 2A-SU, 2-S2, and 3-S2.

Item	Fuel-Adjustment Factors and Consumption Rates by Speed (mph)													
	5	10	15	20	25	30	35	40	45	50	55	60	65	70
2A-SU fuel consumption (gal/1000 miles)														
Estimated at 12 kips[a]	435	244	141	145		111		114			152			
Estimated at 17.1 kips[a]	444	256	152	156		121		127			172			
Ratio (12 kips/17.1 kips)	0.98	0.95	0.93	0.93	0.92	0.92	0.90	0.89	0.88	0.88	0.88	0.87	0.87	0.87
Measured at 17.1 kips	217	217	180	132	132	122	126	129	140	151	159	166	172	178
Adjusted to 12 kips	212	207	167	122	122	112	113	115	123	133	139	144	150	163
Semitruck fuel consumption (gal/1000 miles)														
Estimated at 62.5 kips[a]	408	353	227	204		196		210			250			
Estimated at 56.0 kips[a]	408	353	227	204		192		204			238			
Estimated at 40.0 kips[a]	404	350	225	202		186		188			209			
Ratio (62.5 kips/56.0 kips)	1.00	1.00	1.00	1.00	1.01	1.02	1.03	1.03	1.04	1.05	1.05	1.06	1.07	1.08
Ratio (40.0 kips/56.0 kips)	0.99	0.99	0.99	0.99	0.98	0.97	0.94	0.92	0.91	0.89	0.88	0.87	0.87	0.87
Measured at 56.0 kips	470	370	287	205	202	200	197	195	192	190	192	195	197	200
Adjusted to 62.5 kips (3-S2)	470	370	287	205	204	204	202	201	199	199	202	207	210	215
Adjusted to 40.0 kips (2-S2)	465	367	284	203	198	193	186	180	174	169	168	170	171	173

[a]Estimated from data collected by France (7).

Figure 6. Constant-speed fuel consumption (gal/1000 miles): small car.

GRADE	SPEED mph													
%	5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	87.00	85.00	65.00	52.50	46.80	41.00	44.30	47.50	47.50	47.50	52.30	57.00	61.80	66.50
7	85.00	83.00	52.50	48.00	42.50	37.00	39.80	41.50	42.00	42.50	46.30	50.00	55.50	61.00
6	83.00	81.00	50.50	47.00	42.00	36.00	37.00	38.00	39.00	40.00	43.00	46.00	51.00	56.00
5	82.00	79.00	51.50	46.00	41.50	35.00	36.00	37.00	37.80	38.50	41.00	43.50	47.80	52.00
4	80.00	78.00	57.50	44.00	40.00	34.00	35.30	36.50	36.80	37.00	39.50	42.00	45.20	48.30
3	77.00	76.00	61.00	43.00	39.80	33.50	34.50	35.50	35.80	36.00	38.00	40.00	42.50	45.00
2	75.00	73.00	59.50	42.00	38.00	32.00	32.50	33.00	33.00	33.00	35.00	37.00	39.50	42.00
1	70.00	70.00	55.50	41.00	34.50	28.00	29.00	30.00	28.30	27.50	30.00	32.50	35.50	38.50
0	67.00	64.00	50.00	36.00	29.50	27.00	24.80	24.00	24.80	25.00	25.50	28.00	31.50	35.00
-1	57.00	57.00	42.00	27.00	23.00	19.00	21.00	23.00	21.50	20.00	22.30	24.50	27.80	31.00
-2	49.00	49.00	35.00	22.00	19.00	16.00	18.50	21.00	19.00	17.00	19.00	21.30	23.50	26.00
-3	43.00	43.00	32.00	20.00	17.50	15.00	17.30	19.50	17.50	15.50	17.30	19.00	21.50	24.00
-4	38.00	38.00	29.00	20.00	17.40	14.80	16.90	19.00	16.50	14.00	15.80	17.50	20.30	23.00
-5	35.00	35.00	27.50	20.00	17.00	14.50	16.50	18.50	15.30	12.00	14.30	16.50	19.30	22.00
-6	34.50	34.50	26.80	19.00	16.50	14.00	15.80	17.50	14.50	11.50	13.50	15.50	18.30	21.00
-7	34.50	34.50	26.00	17.50	15.50	13.50	15.20	17.00	13.80	10.50	12.30	14.00	16.50	19.00
-8	35.00	35.00	25.50	16.00	14.50	13.00	14.50	16.00	12.50	9.00	11.00	13.00	15.50	18.00

then used to generate fuel-consumption tables for each grade level as given. Figures 6-13 give the typical fuel consumption for the eight vehicle classes.

Effect of Pavement Type and Condition on Fuel Consumption

Measurements were taken on PCC, AC, ST, and gravel sections to determine if surface type had an influence on fuel consumption. Three AC sections were used to test for the influence of surface condition on fuel consumption. Student's t-test values were computed for each of the individual combinations of speed and section to determine if there were any significant differences on fuel consumption. In general, there were no statistically significant differences at the 95 percent level between the fuel consumption on the paved sections. Fuel consumption on the unpaved section was slightly higher then the fuel consumption on the paved sections.

Estimation of Fuel Consumption on Curves

Unfortunately, it was not possible to measure fuel consumption as a function of curvature. Previous researchers have shown a definite correlation between degree of curvature and fuel consumption for small radius curves (1,2). However, this relation is generally not significant for the curves encountered on rural roads, which was the primary concern in this research. Therefore, the effect of curvature on fuel consumption was approximated by using horsepower calculations. The procedure used to estimate fuel consumption on curves was as follows:

1. Compute the horsepower (h_c) required to transverse the curve at a constant speed,
2. Determine the grade that could be climbed with h_c at the same constant speed, and
3. Use interpolation to determine the fuel consumption at the grade level determined in step 2 from Figures 6-13 for the vehicle type.

Figure 7. Constant-speed fuel consumption (gal/1000 miles): medium car.

GRADE %	SPEED mph 5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	91.00	91.00	83.30	75.00	76.00	77.00	82.00	86.50	90.00	93.50	102.00	110.00	112.00	113.00
7	82.00	82.00	75.50	68.50	68.30	68.00	71.80	75.50	80.00	84.00	93.50	103.00	104.00	106.00
6	77.50	77.50	70.80	64.00	63.00	62.00	65.30	68.50	73.00	77.30	87.00	96.00	97.80	99.50
5	74.00	74.00	67.50	61.00	59.80	58.50	61.30	64.00	68.50	72.50	80.30	87.50	91.50	95.50
4	73.00	73.00	66.50	60.00	57.80	55.50	58.30	60.50	64.50	68.00	73.00	77.50	84.80	92.00
3	71.50	71.50	64.50	57.50	55.50	53.50	56.30	58.80	61.00	63.00	66.00	68.50	78.30	87.50
2	68.00	68.00	60.80	53.50	52.30	50.50	53.00	55.50	56.80	58.00	60.50	62.50	72.50	82.50
1	61.50	61.50	54.30	47.00	46.30	45.00	46.00	46.50	49.30	51.50	54.50	57.00	64.50	72.00
0	55.40	55.40	47.30	38.70	38.00	37.30	37.60	38.00	40.50	43.00	47.90	52.80	57.60	62.70
-1	52.00	52.00	41.80	31.00	30.30	29.50	31.80	33.50	34.80	36.00	41.00	45.50	51.00	56.00
-2	50.80	50.80	39.70	28.00	25.80	22.50	26.30	29.50	30.30	31.00	34.80	38.50	45.00	51.50
-3	51.30	51.30	38.90	26.90	23.70	20.50	23.30	25.50	26.50	27.80	31.50	35.00	41.30	47.30
-4	52.00	52.00	39.90	27.30	24.00	20.30	20.70	21.00	23.00	25.00	28.80	32.00	37.50	42.50
-5	53.00	53.00	40.00	27.30	23.80	20.00	19.30	18.50	20.50	22.80	26.00	29.50	34.50	39.00
-6	53.50	53.50	40.60	27.30	23.50	19.80	18.00	16.30	18.90	21.00	29.00	27.00	30.80	34.00
-7	54.30	54.30	40.70	27.30	23.80	19.50	17.30	15.00	17.30	19.00	21.30	23.50	26.80	30.00
-8	54.50	54.50	41.20	27.30	23.00	19.30	16.80	14.30	15.40	16.50	18.30	20.00	22.80	25.50

Figure 8. Constant-speed fuel consumption (gal/1000 miles): large car.

GRADE %	SPEED mph 5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	84.00	84.00	82.30	80.50	81.30	82.00	86.00	90.00	111.00	131.00	131.00	131.00	161.00	190.00
7	83.50	83.50	81.50	79.50	78.80	78.00	81.50	85.00	97.50	110.00	113.00	115.00	133.00	150.00
6	83.00	83.00	80.80	78.50	76.30	74.00	77.30	80.50	86.30	92.00	96.00	100.00	113.00	125.00
5	82.20	82.20	79.60	77.00	74.00	71.00	73.30	75.50	78.50	81.50	85.80	90.00	97.50	105.00
4	81.30	81.30	77.90	74.50	70.30	66.00	68.30	70.50	71.00	71.50	75.80	80.00	85.50	91.00
3	79.80	79.80	75.70	71.50	65.80	60.00	62.30	64.50	64.20	63.80	67.80	71.80	76.60	81.30
2	77.00	77.00	71.50	66.00	59.00	52.00	55.30	58.50	58.00	57.50	61.30	65.00	70.00	75.00
1	70.00	70.00	64.00	58.00	51.80	45.50	48.80	52.00	52.50	53.00	56.00	59.00	64.20	69.40
0	59.00	59.00	51.10	43.20	41.80	40.30	42.30	44.30	46.50	48.70	52.50	56.30	61.60	66.90
-1	56.50	56.50	45.00	33.50	35.00	36.50	37.80	39.00	42.50	46.00	50.00	54.00	60.00	66.00
-2	54.80	54.80	42.90	31.00	32.00	33.00	33.50	34.00	38.50	43.00	47.50	52.00	58.00	64.00
-3	54.00	54.00	41.30	28.50	29.30	30.00	29.50	29.00	34.30	39.50	44.30	49.00	55.40	61.80
-4	53.50	53.50	40.70	27.80	27.20	26.50	25.50	24.50	30.30	36.00	40.30	44.50	50.80	57.00
-5	53.00	53.00	40.00	27.20	25.00	23.00	21.60	20.20	26.00	31.80	34.40	37.00	42.30	47.50
-6	52.50	52.50	39.80	27.00	23.50	20.00	18.00	16.00	22.00	28.00	27.50	27.00	29.50	32.00
-7	52.50	52.50	39.50	26.50	22.00	17.50	13.90	10.20	16.60	23.00	22.50	22.00	23.50	25.00
-8	52.50	52.50	39.50	26.50	20.50	14.50	11.30	8.50	13.30	18.00	18.00	18.00	19.00	20.00

COMPARISON WITH PREVIOUS RESEARCH

There have been four prior studies on the effect of roadway characteristics on fuel consumption (1,8-10). The effect of grade and speed were investigated by Claffey (1) and Zaniewski and others (8) with results similar to the findings of this research. Because the findings of this research are for the current vehicle fleet and essentially agree with prior results, it is recommended that the current results be used in future economic analyses.

The findings of this research relative to the effect of pavement roughness are in direct conflict with the findings of Claffey and Zaniewski, where pavement roughness was found to influence fuel consumption by as much as 30 and 10 percent, respectively. However, the rough paved sections in each of these studies were badly broken, potholed, and patched and thus are not representative of realistic operating conditions in the United States. Use of these data require interpolation between the extreme conditions of pavement roughness. In Kenya (9), no effect of pavement roughness on fuel consumption was found, reportedly because the range of roughness was too small.

Ross (10) studied the fuel consumption of three automobiles at 55 mph on five bituminous test sections with a range in roughness of 0.9 to 4.4 on the serviceability index (SI) scale. Ross reported that, for a practical range of roughness (1.5-4.5

Figure 9. Constant-speed fuel consumption (gal/1000 miles): pickup truck.

GRADE %	SPEED mph 5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	137.00	137.00	129.00	120.00	111.00	101.00	109.00	116.00	132.00	147.00	155.00	162.00	170.00	178.00
7	127.00	127.00	108.00	108.00	100.00	92.00	97.50	103.00	119.00	135.00	144.00	152.00	162.00	172.00
6	118.00	118.00	109.00	100.00	93.50	87.00	90.00	93.00	108.00	123.00	133.00	143.00	155.00	166.00
5	112.00	112.00	104.00	95.00	89.50	84.00	85.80	87.50	99.80	112.00	124.00	135.00	145.00	155.00
4	107.00	107.00	97.50	88.00	85.50	83.00	83.50	84.00	93.00	102.00	114.00	125.00	137.00	148.00
3	103.00	103.00	92.50	82.00	81.50	81.00	81.00	81.00	86.00	91.00	103.00	114.00	128.00	142.00
2	97.00	97.00	85.00	73.00	75.00	77.00	77.00	77.00	79.50	82.00	91.00	100.00	116.00	131.00
1	86.00	86.00	75.00	64.00	66.50	69.00	69.50	70.00	71.30	72.50	78.80	85.00	98.00	111.00
0	77.90	77.90	64.80	51.60	52.30	53.00	54.00	55.00	59.70	64.40	67.50	70.60	81.90	93.20
-1	72.00	72.00	55.00	38.00	37.80	37.50	39.30	41.00	47.00	53.00	56.50	60.00	66.50	73.00
-2	70.00	70.00	52.30	34.50	33.30	32.00	33.00	34.00	37.50	41.00	44.50	48.00	51.50	57.00
-3	68.00	68.00	51.30	34.50	31.30	28.00	29.00	30.00	33.00	36.00	39.50	43.00	47.50	52.00
-4	68.00	68.00	52.00	36.00	31.50	27.00	27.50	28.00	30.50	33.00	36.50	40.00	45.00	50.00
-5	68.50	68.50	52.80	37.00	31.50	26.00	26.80	27.50	29.80	32.00	35.00	38.00	44.00	50.00
-6	70.00	70.00	54.20	38.30	32.20	26.00	26.60	27.00	28.50	30.00	33.50	37.00	42.00	47.00
-7	71.00	71.00	54.80	38.50	32.30	26.00	26.00	26.00	26.80	27.50	30.80	34.00	38.00	42.00
-8	72.50	72.50	55.50	38.50	32.30	26.00	25.50	25.00	25.00	25.00	28.00	31.00	32.50	34.00

Figure 10. Constant-speed fuel consumption (gal/1000 miles): 2A-SU.

GRADE %	SPEED mph 5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	416.00	406.00	330.00	263.00	247.00	231.00	238.00	248.00	223.00	203.00	202.00	201.00	204.00	216.00
7	389.00	380.00	306.00	242.00	230.00	219.00	228.00	240.00	217.00	197.00	198.00	198.00	201.00	214.00
6	362.00	354.00	288.00	230.00	220.00	212.00	221.00	232.00	211.00	194.00	194.00	194.00	198.00	211.00
5	343.00	335.00	228.00	222.00	213.00	206.00	210.00	218.00	201.00	188.00	179.00	189.00	194.00	206.00
4	324.00	317.00	263.00	215.00	206.00	198.00	200.00	204.00	192.00	182.00	176.00	182.00	187.00	200.00
3	305.00	298.00	249.00	206.00	197.00	189.00	187.00	187.00	179.00	174.00	171.00	174.00	180.00	192.00
2	281.00	275.00	230.00	192.00	182.00	173.00	170.00	169.00	165.00	163.00	163.00	166.00	171.00	184.00
1	249.00	243.00	202.00	166.00	151.00	138.00	141.00	146.00	151.00	150.00	153.00	156.00	161.00	174.00
0	212.00	207.00	167.00	132.00	121.00	112.00	113.00	115.00	123.00	133.00	139.00	144.00	150.00	163.00
-1	150.00	147.00	126.00	108.00	101.00	94.60	86.10	79.30	93.10	108.00	120.00	130.00	135.00	147.00
-2	121.00	118.00	109.00	101.00	91.20	82.60	73.10	65.10	73.80	83.70	99.70	115.00	122.00	135.00
-3	121.00	118.00	107.00	97.80	86.60	76.20	65.90	57.10	64.10	72.20	88.30	103.00	110.00	121.00
-4	124.00	121.00	109.00	98.80	84.80	71.60	60.50	50.80	58.00	66.10	79.10	92.00	98.50	110.00
-5	127.00	124.00	110.00	99.70	83.00	67.00	56.50	47.20	53.20	59.90	71.70	83.30	90.70	102.00
-6	130.00	127.00	112.00	101.00	81.10	62.40	53.40	45.50	50.10	55.50	66.40	76.30	83.70	94.30
-7	131.00	128.00	112.00	98.80	78.40	58.80	50.20	42.80	45.70	49.30	60.30	70.30	79.40	91.60
-8	133.00	130.00	110.00	94.20	74.30	55.10	47.50	41.00	42.60	44.90	56.80	67.70	75.90	87.90

SI), fuel consumption is 1.5 percent higher on the rough section. In developing this conclusion, Ross used very strict criteria for eliminating outlaying data, and hence the variance of the data used in the analysis was much smaller than would be anticipated in the real world. Ross found that the measured fuel consumption on a section with an SI of 2.1 was less than the fuel consumption on sections with SIs of 4.4 and 3.6 for all three vehicles. Because of these apparent anomalous measures, these data were removed from the final analysis. Considering the fact that Ross eliminated so much data from the final analysis and still only found a very minor influence of roughness on fuel consumption, it is believed that an analysis of the complete data base would support the findings that roughness does not have a measurable effect on real-world fuel economy.

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Figure 11. Constant-speed fuel consumption (gal/1000 miles): 3A-SU.

GRADE %	SPEED mph 5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	885.00	705.00	525.00	344.00	351.00	357.00	360.00	363.00	378.00	393.00	392.00	390.00	399.00	407.00
7	789.00	640.00	491.00	342.00	348.00	353.00	357.00	360.00	373.00	386.00	384.00	382.00	389.00	397.00
6	694.00	576.00	458.00	340.00	344.00	347.00	351.00	354.00	365.00	375.00	374.00	372.00	379.00	386.00
5	596.00	510.00	424.00	338.00	339.00	339.00	342.00	345.00	353.00	361.00	360.00	359.00	366.00	372.00
4	505.00	448.00	391.00	333.00	331.00	328.00	331.00	333.00	339.00	344.00	344.00	343.00	349.00	355.00
3	412.00	383.00	354.00	324.00	318.00	312.00	314.00	315.00	318.00	320.00	321.00	322.00	328.00	333.00
2	330.00	323.00	316.00	308.00	297.00	285.00	282.00	278.00	278.00	278.00	287.00	295.00	300.00	304.00
1	274.00	270.00	272.00	273.00	254.00	235.00	230.00	225.00	225.00	225.00	238.00	250.00	255.00	260.00
0	236.00	217.00	198.00	179.00	168.00	156.00	153.00	149.00	149.00	149.00	153.00	156.00	163.00	169.00
-1	196.00	159.00	122.00	85.00	79.00	73.00	68.00	62.00	72.00	82.00	79.00	75.00	75.00	79.00
-2	153.00	119.00	85.00	50.00	45.00	40.00	30.00	24.00	30.00	37.00	37.00	37.00	37.00	38.00
-3	126.00	96.00	66.00	36.00	32.00	28.00	16.50	15.00	18.50	22.00	25.00	28.00	28.00	30.00
-4	110.00	82.00	54.50	27.00	23.50	20.00	15.50	11.00	15.00	19.00	21.80	24.60	25.00	26.00
-5	94.00	70.00	46.00	22.00	18.50	15.00	11.50	8.00	13.00	18.00	19.80	21.50	22.00	23.00
-6	78.00	58.00	38.00	18.00	14.50	11.00	9.00	7.00	12.00	17.00	17.80	18.50	19.00	20.00
-7	64.50	48.00	31.50	15.00	11.00	7.00	6.50	6.00	11.00	16.00	15.60	15.10	15.00	16.00
-8	46.50	35.00	23.50	12.00	8.50	5.00	5.00	5.00	10.00	15.00	13.70	12.30	12.00	14.00

Figure 12. Constant-speed fuel consumption (gal/1000 miles): 2-S2.

GRADE %	SPEED mph 5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	742.00	655.00	564.00	475.00	477.00	478.00	406.00	337.00	296.00	258.00	286.00	318.00	373.00	428.00
7	683.00	605.00	524.00	446.00	448.00	449.00	384.00	323.00	281.00	240.00	271.00	305.00	353.00	402.00
6	643.00	575.00	500.00	426.00	426.00	425.00	370.00	318.00	276.00	236.00	267.00	301.00	340.00	381.00
5	604.00	546.00	529.00	416.00	402.00	415.00	365.00	318.00	276.00	236.00	267.00	301.00	336.00	372.00
4	594.00	531.00	467.00	406.00	404.00	410.00	364.00	318.00	276.00	236.00	267.00	301.00	334.00	368.00
3	574.00	516.00	455.00	396.00	399.00	400.00	359.00	318.00	276.00	236.00	267.00	301.00	330.00	359.00
2	534.00	481.00	423.00	366.00	372.00	376.00	340.00	305.00	263.00	222.00	254.00	288.00	312.00	337.00
1	505.00	436.00	366.00	297.00	309.00	318.00	274.00	231.00	206.00	182.00	199.00	218.00	252.00	285.00
0	465.00	367.00	284.00	203.00	198.00	193.00	186.00	180.00	174.00	169.00	168.00	170.00	171.00	173.00
-1	346.00	258.00	178.00	102.00	100.00	96.50	93.50	89.50	87.00	111.00	111.00	113.00	129.00	130.00
-2	228.00	129.00	74.20	0.00	0.00	0.00	0.00	0.00	0.00	53.40	54.30	56.70	86.80	86.50
-3	98.90	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	43.40	43.30
-4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Discussion

Paul Claffey

My comments on Zaniewski's work refer specifically to the conclusion expressed in the abstract of his paper, that the highway pavement and its condition do not affect fuel economy over the range of conditions normally encountered in the United States. Certainly, many miles of road surface in this country, whether of concrete, asphalt, or stabilized gravel, have characteristics compatible with good motor vehicle fuel economy. Nevertheless, there are still hundreds of miles of primary roads with surfaces that have a deleterious effect on fuel economy. It has been found, from large-scale studies of motor vehicle fuel consumption relative to road surface conditions for all types of vehicles (especially large trucks), that fuel economy drops sharply for operation on road surfaces that (a) allow wheel slippage (loose surface material), (b) force tire indentations (exposed imbedded stones and/or a spalled surface condition), and/or (c) provide a coarse-sandpaper kind of surface (certain stone surface treatments). For example, a fully loaded 2-S2 tractor-semitrailer truck combination traveling at 50 mph will use more than 50 percent more fuel on a badly spalled concrete road surface as compared with operation over a smooth high-type road surface. I have observed many miles of such

Figure 13. Constant-speed fuel consumption (gal/1000 miles): 3-S2.

GRADE	SPEED mph													
%	5	10	15	20	25	30	35	40	45	50	55	60	65	70
8	756.00	679.00	599.00	523.00	553.00	588.00	528.00	463.00	426.00	389.00	424.00	445.00	460.00	490.00
7	694.00	627.00	557.00	489.00	518.00	551.00	498.00	443.00	402.00	360.00	399.00	424.00	432.00	456.00
6	655.00	593.00	531.00	467.00	491.00	520.00	478.00	434.00	393.00	350.00	388.00	413.00	411.00	426.00
5	615.00	563.00	562.00	455.00	462.00	505.00	468.00	431.00	389.00	347.00	384.00	407.00	400.00	410.00
4	604.00	547.00	495.00	443.00	462.00	496.00	463.00	426.00	384.00	342.00	378.00	400.00	390.00	396.00
3	581.00	531.00	481.00	430.00	453.00	480.00	451.00	420.00	378.00	335.00	370.00	390.00	375.00	377.00
2	543.00	494.00	444.00	395.00	418.00	444.00	419.00	392.00	350.00	306.00	340.00	361.00	342.00	340.00
1	512.00	447.00	381.00	315.00	339.00	365.00	325.00	285.00	261.00	239.00	252.00	258.00	260.00	271.00
0	470.00	370.00	287.00	205.00	204.00	204.00	202.00	201.00	199.00	199.00	202.00	207.00	210.00	215.00
-1	0.00	0.00	0.00	0.00	0.00	34.10	49.90	58.20	63.80	89.90	91.70	90.90	93.80	89.10
-2	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	12.10	24.20
-3	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-4	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-5	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-6	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-7	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
-8	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

spalled concrete road surface in the United States, including mileage on the Interstate system.

The road surfaces used in the test operations reported on by Zaniewski are described in Table 2. They are identified as AC, PCC, ST, or gravel without any indication of the looseness of surface material, the amount of exposure of imbedded stone, or the roughness characteristics of the surface stone. Because fuel consumed by the test vehicles was about the same for all of the test sections, each probably had a firm, smooth surface without exposed surface stone. The SI (given in Table 2 for each test road section) is no help. It could vary over the range shown (from less than 1.0 to more than 4.0) because of surface undulations, even though the road surface is about the same as far as vehicle fuel consumption is concerned. A road with a low SI does not necessarily cause excess fuel to be consumed by vehicles that use the road.

Zaniewski refers to the work of Ross (10). In the Ross study, passenger car fuel consumption was measured for operation on each of a series of level, straight road sections that have relatively smooth undulating surfaces. The SI was 0.9 for the road in the poorest condition and 4.4 for the road in the best condition (similar to the test roads used by Zaniewski). Ross found, as did Zaniewski, that the fuel consumption per unit distance for each test car was about the same for each of the roads in his study.

I had occasion to observe the test roads used in the Ross study, and it was obvious why fuel economy was about the same for each road. None of the test sections had loose surface material, exposed imbedded stones, or coarse-sandpaper roughness. Despite the wide range of SI represented in the study roads, the surfaces were all pretty much alike relative to fuel consumption.

There are many sections of primary road in this country as well as in other countries where surface conditions cause excessive fuel consumption. It would be most unwise to conclude from studies conducted only on roads with surfaces conducive to good fuel economy that the highway pavement and its condition do not affect fuel economy.

Author's Closure

The points made by Claffey concerning alternative measures of road surface characteristics that may influence the fuel consumption of vehicles are well taken and probably correct. The generalized statement made in the abstract of the paper, "pavement condition does not affect fuel consumption over the range of conditions normally encountered in the United States", should have explicitly referred to pavement condition as defined by the SI.

This is a very significant finding, in that many states use either the SI or an alternative measure of roughness to quantify the condition of roadways. These states were erronously using previous research, which indicated that fuel consumption on a good pavement surface was less than fuel consumption on a bad pavement, to compute the economic and energy benefits of improving the SI, or roughness, of a road.

The three types of surface condition measures identified by Claffey as affecting fuel consumption are all candidates for further research into the influence of surface characteristics on fuel economy. Specifically, any future research should include not only the measurement of fuel, but engineering measures of the condition of the road surface with respect to looseness and microtexture. In order to be useful to highway engineers, terms like "badly spalled concrete" and "smooth high pace type" road surfaces must be replaced with reproducable measures of the extent of spalling. Unwise conclusions concerning the interrelations between pavement condition and fuel consumption can only be avoided by measuring both the dependent and independent variables in the relations. The research performed by Ross (10) and myself has clarified and resolved the situation with respect to one very important measure of road surface condition.

REFERENCES

1. P.J. Claffey. Running Costs of Motor Vehicles as Affected by Road Design and Traffic. NCHRP, Rept. 111, 1971, 97 pp.

2. R. Winfrey. Economic Analysis for Highways. International Textbook Company, Scranton, PA, 1969.
3. J.P. Zaniewski, B.C. Butler, Jr., G. Cunningham, G.E. Elkins, and R. Machemehl. Vehicle Operation Costs, Fuel Consumption, and Pavement Type and Condition Factors. FHWA, Final Report, June 1982.
4. 1214D/1228 in Vehicle Diesel Fuel Economy Measurement System. Fluidyne Instrumentation, Oakland, CA, Feb. 1978.
5. D.R. Drew. Traffic Flow Theory and Control. McGraw-Hill, New York, 1968.
6. A.D. St. John and D.R. Kobett. Grade Effects on Traffic Flow Stability and Capacity. NCHRP, Rept. 185, 1978, 110 pp.
7. C. France. Fuel Economy of Heavy Duty Vehicles. U.S. Environmental Protection Agency, Ann Arbor, MI, Sept. 1976.
8. J.P. Zaniewski, B.K. Moser, P.J. de Morais, and R.L. Kaesehagen. Fuel Consumption Related to Vehicle Type and Road Conditions. TRB, Transportation Research Record 702, 1979, pp. 328-334.
9. H. Hide. An Improved Data Base for Estimating Vehicle Operating Costs in Developing Countries. Transport and Road Research Laboratory, Crowthorne, Berkshire, England, TRRL Supplementary Rept. 223 UC, 1976.
10. F.R. Ross. The Effect of Pavement Roughness on Vehicle Fuel Consumption. Research Unit, Materials Section, Division of Highways and Transportation Facilities, Wisconsin Department of Transportation, Madison, June 1981.

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Impact of Two-Way Left-Turn Lanes on Fuel Consumption

ZOLTAN A. NEMETH, PATRICK T. McCOY, AND JOHN L. BALLARD

Two-way left-turn lanes (TWLTLs) serve to eliminate conflict between midblock left turns and through traffic moving in the same direction. The purpose of this study was to evaluate the potential fuel savings generated by TWLTLs through reduced stops and delays. In the first part of the paper, the results of two earlier studies are examined and related to fuel efficiency and fuel savings. In the second part, the results of a simulation study are presented. The simulation study estimated annual fuel savings generated by the introduction of TWLTLs on sections of two-way two-lane and two-way four-lane arterials under various combinations of driveway density, average daily traffic, and left-turn frequency. The magnitude of the benefits to be derived from TWLTLs obviously depends on the magnitude of the existing midblock left-turn conflicts. On two-lane roadways, potential fuel savings can be substantial even at relatively low volumes. Fuel savings on four-lane roadways compared favorably with fuel savings estimated to result from another energy conservation method, the right-turn-on-red policy.

Urban streets must serve two distinct and conflicting functions, namely, the movement of traffic and the provision of access to abutting properties. The operating characteristics of a given street are largely determined by the compromises involved in serving these two functions. Those streets that are designated to favor one of these functions present relatively little problem to the transportation engineer. For example, freeways serve well the movement function, and local streets provide easy access to all properties. Most arterials, however, serve both movement and access. Even arterials, which were originally intended to serve the movement function, eventually attracted commercial, industrial, or high-density residential developments, i.e., the high accessibility resulted in an increased intensity of land use. The nature and the intensity of these developments often created left-turn demands to driveway entrances between intersections that led to conflicts between left turners and through traffic.

In many cases, the conventional median with left-turn pockets is a good solution. There are instances, however, when the need for access to abutting properties from both directions is there, but the pattern of location of the driveways makes left-turn pockets impractical. The prohibition of left turns would eliminate the conflict between through traffic and turning traffic, but it seriously limits the accessibility of the properties and would therefore be often unacceptable. Median two-way left-turn lanes (TWLTLs) may offer a solution.

A TWLTL is a single lane identified by pavement markings and signs and reserved for the exclusive use of left-turning traffic from either direction. Left turns can be made from any point along the median lane.

The major function of this lane is to provide a deceleration and waiting lane for left turns to minor traffic generators (major traffic generators are better served by one-way left-turn pockets), including abutting properties and minor streets. Secondary functions include the separation of opposing traffic flows, an acceleration lane for vehicles turning left into the arterials from minor streets and driveways, an emergency lane in case of temporary lane closures due to maintenance or accidents, and a lane for use by emergency vehicles, especially during peak hours.

A TWLTL can simultaneously improve access to land use and increase the speed of through traffic by eliminating the conflict between left-turning vehicles and through traffic moving in the same direction. Left-turning vehicles can wait in safety for appropriate gaps in the opposing through traffic.

Initial concerns with the potential hazard of head-on collisions between left-turning vehicles that enter from the opposite direction have been proved unfounded (1,2). Several studies have shown TWLTLs to be beneficial by reducing both left-turn-related accidents and delays. Guidelines have been published regarding the application and design of TWLTLs (3,4). One of the more specific guidelines states (5):

> The two-way left-turn lane is operationally warranted on arterial highways that have average daily traffic (ADT) volumes higher than 10 000 and traffic speeds faster than 48 km/h (30 mph). The number of driveways should exceed 60 in 1.6

km (1 mile), and there should be fewer than 10 high-volume driveways. Left-turn driveway maneuvers in 1.6 km should total at least 20 percent of the through traffic volume during peak periods. High rates of accidents that involve left-turn maneuvers can also warrant this technique.

PURPOSE OF STUDY

The purpose of this study is to investigate the potential of TWLTLs to save fuel in urban areas by reducing stops and travel delays. In the first part, results of past delay studies reported in the literature are related to some widely accepted methods of estimating fuel savings.

In the second part, a simulation study is conducted to evaluate the relative magnitude of potential fuel savings under various conditions. Results are compared with fuel savings from another energy-conservation measure.

REVIEW OF LITERATURE

A recent Federal Highway Administration (FHWA) publication (6) suggests that average transient speed as measured by travel time is a good composite parameter for reflecting stops and slowdowns and is closely related to fuel consumption. By using data from a previous study conducted by General Motors (7), the following relation is suggested for urban conditions where speeds are usually at or below 35 mph:

$$\text{Fuel consumption (gallons per vehicle-mile)} = 0.0362 + (0.746/\bar{V}) \quad (1)$$

where $\bar{V}$ is in miles per hour. The equation will be used to relate the results of past delay studies to fuel consumption.

Ohio Study

The Ohio study clearly demonstrated that the impact of TWLTLs, in terms of delay reduction, depend very much on the pre-TWLTL condition (1). The three before-and-after case studies showed results ranging from negative to statistically significant positive impact on speeds. Average running speeds were computed by eliminating from the travel time all delays that were not related to midblock left turns. Test vehicles were used to collect travel-time data. Each data point represents approximately 40 runs.

Site 1 involved the restriping of a 36-ft-wide four-lane roadway as a three-lane roadway. The ADT was 16 320 vehicles. As could be expected at this volume level, the elimination of one through lane in each direction offset the beneficial effects of the TWLTL and average speeds as well as fuel consumption (expressed in miles per gallon) dropped: i.e.,

Direction	Period	Avg Speed (mph)	Fuel Efficiency (miles/gal)
Eastbound	Before	34.5	17.3
	After	30.9	16.6
Westbound	Before	33.2	17.0
	After	28.5	16.0

In conclusion, at site 1, the conversion of two through lanes into a TWLTL improved the access function of the roadway at the expense of the through movement function.

At site 2, a 59-ft-wide four-lane arterial was restriped as a five-lane roadway. The ADT was 17 610 vehicles. The intensity of the strip development varied along the 1-mile-long site, but was not high. Consequently, the improvement was not statistically significant:

Direction	Period	Avg Speed (mph)	Fuel Efficiency (miles/gal)
Eastbound	Before	32.3	16.9
	After	33.4	17.1
Westbound	Before	33.6	17.1
	After	34.4	17.3

Although there was a reduction in the number of conflicts between left-turning and through vehicles, the total numbers were not high, and therefore the impact on speeds was not significant.

Site 3 involved the widening of a 31-ft two-lane roadway to 36 ft and restriping it as a three-lane roadway. The ADT was 14 070 on the north half of this study section and 12 940 on the south half. Development intensity was especially low on the south half. The results for the two sections are given below:

Direction	Period	Avg Speed (mph)	Fuel Efficiency (miles/gal)
North half			
Northbound	Before	35.2	17.4
	After	38.6	18.0
Southbound	Before	38.5	18.0
	After	39.8	18.2
South half			
Northbound	Before	29.3	16.2
	After	32.6	16.9
Southbound	Before	30.1	16.4
	After	32.9	17.0

The left-turn conflicts, expressed as the number of brakings, were relatively high at this site (1327 compared with 614 and 575 at sites 2 and 3, respectively), and consequently the reduction in conflict had the most significant impact on average speeds and fuel consumption at this site.

In conclusion, the potential of TWLTLs as a way to reduce fuel consumption obviously depends very much on the nature and extent of the problem TWLTLs are introduced to solve.

Midwest Research Institute Study

The subject of the Midwest Research Institute study was the control of direct access to arterial highways and included TWLTLs as one of the median treatments (5). Level of development and highway ADT were defined as follows:

Item	Definition
Level of development (driveways/mile)	
Low	<30
Medium	30-60
High	>60
Highway ADT	
Low	<5000
Medium	5000-15 000
High	>15 000

The effectiveness of the TWLTL in reducing delay was estimated by assuming a value for the increase in average running speed. The following assumptions were made to estimate reductions in delay for typical four-lane highways:

1. Arterials with low traffic volumes or low levels of development would not experience any increase in running speed.

Table 1. Reduction in stops, two-lane roadway, 1000-ft section.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/h)	Reduction in Stops (no./h) by Left-Turn Volume[c] 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	350	23	36	45
	700	98	157	290
	1000	186	612	982
60	350	0	14	29
	700	69	189	206
	1000	140	804	1216
90	350	18	27	48
	700	74	206	244
	1000	326	814	1630

[a]Total number of driveways on both sides of street.
[b]Volume in each direction, including left turns.
[c]Volume in each direction.

Table 2. Reduction in delay, two-lane roadway, 1000-ft section.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/h)	Reduction in Delay (min/h) by Left-Turn Volume[c] 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	350	4.1	8.8	11.4
	700	13.7	16.8	43.8
	1000	19.4	44.2	79.8
60	350	0	3.8	9.0
	700	5.3	24.1	30.3
	1000	16.1	75.6	123.6
90	350	1.8	6.5	14.4
	700	6.9	30.2	37.6
	1000	47.3	83.6	271.1

[a]Total number of driveways on both sides of street.
[b]Volume in each direction, including left turns.
[c]Volume in each direction.

2. Average running speeds on arterials without median treatments are assumed to be as given below:

Highway ADT	Level of Development	Avg Running Speed (mph)
Medium	Medium	35
	High	30
High	Medium	30
	High	25

3. For a medium level of development, there is an increase of 5 mph in average running speed during the 2 h of each day that show the highest traffic volume. These hours are assumed to include 20 percent of all through vehicles.

4. For a high level of development, there is an increase of 5 mph in average running speed during the 4 h of each day when traffic volume is highest. These hours are assumed to include 35 percent of all through vehicles.

The estimated reduction in delay that results from the introduction of a 1-mile TWLTL is given below, including the corresponding reduction of fuel consumption. The fuel consumption was calculated by Equation 1, based on the above assumptions made by the Midwest Research Institute:

Level of Development	ADT	Annual Reduction per Mile in Delay (h)	Fuel Consumption (gal)
Low	Low	0	0
	Medium	0	0
	High	0	0
Medium	Low	0	0
	Medium	2 628	1 960
	High	6 935	5 175
High	Low	0	0
	Medium	6 059	4 520
	High	17 046	12 715

Table 3. Reduction in stops, four-lane arterial, 2000-ft section.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/h)	Reduction in Stops (no./h) by Left-Turn Volume[c] 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	350	8	6	24
	700	13	59	87
	1050	100	78	599
60	350	5	9	24
	700	17	45	105
	1050	98	237	-[d]
90	350	5	7	26
	700	12	38	114
	1050	88	271	589

[a]Total number of driveways on both sides of street.
[b]Volume in each direction, including left turns (divided equally in each lane).
[c]Volume in each direction.
[d]Jammed flow in no-TWLTL case. Simulation incomplete.

Table 4. Reduction in delay, four-lane arterial, 2000-ft section.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/h)	Reduction in Delay (min/h) by Left-Turn Volume[c] 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	350	1.5	1.1	2.7
	700	0.8	6.8	8.7
	1050	8.4	58.6	112.6
60	350	0.2	1.9	5.9
	700	0.8	4.6	23.9
	1050	6.8	36.3	-[d]
90	350	0.4	0.9	4.1
	700	0.4	3.0	22.2
	1050	5.8	44.6	142.3

[a]Total number of driveways on both sides of street.
[b]Volume in each direction, including left turns (divided equally in each lane).
[c]Volume in each direction.
[d]Jammed flow in no-TWLTL case. Simulation incomplete.

SIMULATION STUDY

Computer simulation models of sections of two-way two-lane (1000-ft) and two-way four-lane (2000-ft) arterials with and without TWLTLs were developed at the University of Nebraska by using the General Purpose Simulation System (GPSS) language. The simulation models are described in more detail elsewhere (8).

The output of the simulation study included reduction in stops and reduction in delay during 1-h simulated periods and under various combinations of driveway densities, traffic volumes, and left-turn volumes. Tables 1-4 summarize the results of the simulation study.

The next step involved the conversion of the stops and delay reduction into hourly fuel savings by using the same relation that is used by the Signal Operations Analysis Package (9):

$$\text{Excess fuel (gal)} = 0.01 \times \text{stops} + 0.6 \text{ delay (h)} \quad (2)$$

Tables 5 and 6 summarize the hourly fuel savings on two- and four-lane roadways that result from the introduction of TWLTLs under various traffic and development density conditions.

The next step involved the estimation of annual fuel savings. This required that an assumption be made regarding the impact of TWLTLs during the day. If the simulated hourly volumes represent peak flows on a given arterial, then the assumptions made by Midwest Research Institute offer a simple method to estimate ADTs and annual fuel savings. These previously listed assumptions say that

1. For a medium level of development, the TWLTL is assumed to affect traffic during the 2 h of each day that show the highest traffic volume. These hours are assumed to include 20 percent of all through vehicles. Driveway densities of 30 and 60/mile are in this category, and annual savings were calculated accordingly (i.e., annual savings = hourly savings x 2 x 365).
2. For a high level of development, the impact is assumed to be significant during the 4 h of each day when the traffic volume is highest. These hours are assumed to include 35 percent of all through vehicles. Annual savings were calculated accordingly at driveway densities of 90/mile.

Tables 7 and 8 summarize the estimated annual fuel savings. It needs to be emphasized that these savings are related to very short sections only, and since typical volumes on TWLTLs are considerably higher (all three test sites in the Ohio study were close to 1 mile long), fuel savings would also be higher.

CONCLUSION

The benefits to be derived from the introduction of TWLTLs depend very much on the type and intensity of existing conflicts created by midblock left turns. The Ohio field studies illustrated this point fairly clearly. The simulation study provided further quantitative evidence.

The estimated reductions in stops, delays, and fuel savings are very high on two-lane roadways (Tables 1, 2, 5, and 7). It needs to be pointed out, though, that the highest simulated volumes (20 000-23 000 ADT) are not likely to occur in reality. However, potential fuel savings are also substantial in the 7000-16 000 ADT range, even at the lowest level of development.

Estimated fuel savings on four-lane roadways (Table 8) are much lower in comparison. This is to be expected, since the simulated four-lane roadway carried practically the same ADT volumes as the two-lane roadway. The driveway densities and turning volumes, as well as through traffic volumes, are well within the range of realistic expectations. Annual fuel savings range from 38 to 5338 gal. In order to place these savings in the proper perspective, it was necessary to compare these quantities with fuel savings from some other energy-conservation method.

Table 5. Reduction in fuel consumption, two-lane roadway, 1000-ft section.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/h)	Reduction in Fuel Consumption (gal/h) by Left-Turn Volume[c]: 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	350	0.271	0.448	0.564
	700	1.117	1.738	3.338
	1000	2.054	6.562	10.618
60	350	0.000	0.178	0.380
	700	0.743	2.131	2.363
	1000	1.564	8.796	13.396
90	350	0.198	0.335	0.624
	700	0.809	2.362	2.816
	1000	3.733	8.976	19.011

[a] Total number of driveways on both sides of street.
[b] Volume in each direction, including left turns.
[c] Volume in each direction.

Table 6. Reduction in fuel consumption, four-lane arterial, 2000-ft section.

Driveway Density[a] (no./mile)	Traffic Volume[b] (vehicles/h)	Reduction in Fuel Consumption (gal/h) by Left-Turn Volume[c]: 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	350	0.095	0.071	0.267
	700	0.138	0.652	0.957
	1050	1.084	1.366	7.216
60	350	0.052	0.109	0.299
	700	0.178	0.496	1.289
	1050	1.048	2.733	-[d]
90	350	0.054	0.079	0.301
	700	0.124	0.410	1.362
	1050	0.938	3.156	7.313

[a] Total number of driveways on both sides of street.
[b] Volume in each direction, including left turns (divided equally in each lane).
[c] Volume in each direction.
[d] Jammed flow in the no-TWLTL case. Simulation incomplete.

Table 7. Annual reduction in fuel consumption, two-lane roadway, 1000-ft section.

Driveway Density[a] (no./mile)	ADT	Reduction in Fuel Consumption (gal) by Left-Turn Volume[b]: 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	7 000	200	325	410
	14 000	815	1 270	2 440
	20 000	1500	4 800	7 750
60	7 000	0	130	280
	14 000	540	1 555	1 725
	20 000	1140	6 420	9 780
90	8 000	290	490	910
	16 000	1180	3 450	4 110
	23 000	5450	13 100	27 800

[a] Total number of driveways on both sides of street.
[b] Volume in each direction.

Table 8. Annual reduction in fuel consumption, four-lane arterial, 2000-ft section.

Driveway Density[a] (no./mile)	ADT	Reduction in Fuel Consumption (gal) by Left-Turn Volume[b]: 35 vehicles/h/1000 ft	70 vehicles/h/1000 ft	105 vehicles/h/1000 ft
30	7 000	69	52	195
	14 000	101	476	699
	21 000	791	997	5268
60	7 000	38	80	218
	14 000	130	362	940
	21 000	765	1195	-[c]
90	8 000	39	58	220
	16 000	90	299	994
	24 000	685	2304	5338

[a] Total number of driveways on both sides of street.
[b] Volume in each direction.
[c] Jammed flow in the no-TWLTL case. Simulation incomplete.

Table 9. Reduction in fuel consumption, four-lane arterial.

Driveway Density[a] (no./mile)	ADT	Reduction in Fuel Consumption (gal/vehicle) by Left-Turn Volume[b]: 35 vehicles/ h/1000 ft	70 vehicles/ h/1000 ft	105 vehicles/ h/1000 ft
30	7 000	0.000 14	0.000 10	0.000 38[c]
	14 000	0.000 10	0.000 47	0.000 68
	21 000	0.000 52	0.000 65	0.003 44
60	7 000	0.000 07	0.000 16	0.000 43[c]
	14 000	0.000 13	0.000 35	0.000 92
	21 000	0.000 50	0.000 78	-[d]
90	8 000	0.000 04	0.000 06	0.000 22[c]
	16 000	0.000 04	0.000 15	0.000 49
	24 000	0.000 45	0.001 50	0.003 48

[a] Total number of driveways on both sides of street.
[b] Volume in each direction.
[c] Quantities under these lines exceed 0.000 29 gal/vehicle, the estimated savings from RTORAS.
[d] Jammed flow in the no-TWLTL case. Simulation incomplete.

The Institute of Transportation Engineers (ITE) published a report in 1980 by Wagner (10) on fuel-conservation impacts of various transportation improvement measures. The most widely implemented measure included in the report is the introduction of right-turn-on-red-after-stop (RTORAS). Fuel savings from RTORAS were calculated for a hypothetical area of 1 million population. It was estimated that an annual traffic volume of 5530 million vehicles will save 1.62 million gal. This corresponds to saving 0.000 29 gal/vehicle. Because RTORAS is considered to be a significant energy-conservation policy, the selection of the above 0.000 29 gal/vehicle fuel savings could be considered a valid yardstick with which to evaluate fuel savings from TWLTLs.

The annual fuel savings shown in Table 8 were therefore recalculated in gallons per vehicle, as shown in Table 9. The steplike heavy line within the table separates the quantities that exceed the savings from RTORAS from those that do not. The following observations can be made from the data presented in Table 9:

1. At a given driveway density, potential fuel savings increase as total volumes (ADT) increase. This increase is more rapid at higher left-turn volumes.

2. Fuel savings change as driveway density changes at a given ADT level and given left-turn volume. This is not unexpected, since changing driveway density changes the average left-turn volumes per driveway. However, no clear pattern can be identified. More research is needed in this area.

REFERENCES

1. Z.A. Nemeth. Two-Way Left-Turn Lanes: State-of-the-Art Overview and Implementation Guide. TRB, Transportation Research Record 681, 1978, pp. 62-69.
2. C.M. Walton, R.B. Machemehl, T. Horne, and W. Fung. Accident and Operational Guidelines for Continuous Two-Way Left-Turn Median Lanes. TRB, Transportation Research Record 737, 1979, pp. 43-54.
3. Z.A. Nemeth. Development of Guidelines for the Application of Continuous Two-Way Left-Turn Median Lanes. Ohio State Univ., Columbus, Final Rept. EES 470, July 1976.
4. Committee 4A-2. Design and Use of Two-Way Left-Turn Lanes. Technical Council Information Rept., ITE Journal, Feb. 1981.
5. D.W. Harwood and J.C. Glennon. Selection of Median Treatments for Existing Arterial Highways. TRB, Transportation Research Record 681, 1978, pp. 70-77.
6. J. Raus. A Method for Estimating Fuel Consumption and Vehicle Emissions on Urban Arterials and Networks. FHWA, Rept. FHWA-TS-81-210, April 1981.
7. L. Evans and R. Herman. Automobile Fuel Economy of Fixed Urban Driving Schedule. Transportation Science, Vol. 12, No. 2, May 1978, pp. 137-152.
8. P.T. McCoy, J.L. Ballard, and V.H. Wijaya. Operational Effects of Two-Way Left-Turn Lanes on Two-Way Two-Lane Streets. Civil Engineering Department, Univ. of Nebraska, Lincoln, Res. Rept. TRP-02-003-81, 1981.
9. Signal Operations Analysis Package. FHWA, July 1979.
10. F.A. Wagner. Energy Impacts of Urban Transportation Improvements. ITE, Washington, DC, Aug. 1980.

Effect of Bus Turnouts on Traffic Congestion and Fuel Consumption

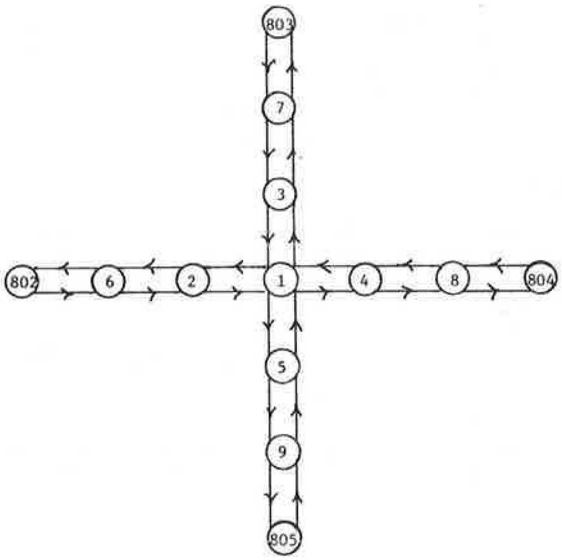
S.L. COHEN

The NETSIM simulation model was employed to determine the energy impacts of using bus turnouts. Two sets of computer runs were made. The first one consisted of 80 runs of a single intersection with different values of independent variables. The second consisted of six runs of three different networks. The result was that bus turnouts were found to have some potential for improving the fuel efficiency of the general traffic stream but only at high values of volume-to-capacity ratios, high bus volumes, and long bus-loading times.

There have been a number of papers written in the past few years concerning the effect of various traffic engineering alternatives on automotive fuel consumption. One set of measures studied include such traffic-flow improvements as right-turn-on-red (1), improved signalization (1,2), one-way versus two-way street patterns (2), cycle length (3), and exclusive turn bays (4).

Another set of traffic engineering alternatives that have potential for fuel savings involves changes in bus operations. These alternatives include such measures as near side versus far side

Figure 1. Single intersection.



stops, bus priority signal strategies, and bus turnouts. The issue of near side versus far side bus stops has been studied by a number of authors (5-7), who arrived at contradictory conclusions. Bus priority signal strategies have also been studied in both field tests (8) and simulations (9). Results indicate that bus service may be slightly improved at the expense of greater delay to the other traffic. By comparison, bus turnouts, the subject of this paper, have received very little attention. Only two rather limited studies (10,11) have been done. Pignataro and others (10) used the NETSIM (formerly UTCS-1) simulation model (12) to study the effect of bus turnouts on general traffic. However, the results of this study may be questionable because it is not made clear whether the authors recognized that the action of a bus pulling out of a turnout is not modeled in NETSIM. In the report by Richards (11), a number of bus turnout designs were studied in the field to determine their effect on bus exit times. This study, however, was limited to two-lane roads (one lane in each direction) with the bus stop not in the vicinity of traffic signals. Thus, it would not be particularly applicable to urban bus operations.

METHODOLOGY

This study used the NETSIM model (12), which was suitably modified, as described in the next section. The use of simulation has a number of advantages. A large number of comparisons that involve before-and-after studies can be performed at a relatively low cost. The cost of performing such studies in field situations would be quite prohibitive. In addition, simulation experiments have repeatability, which is very difficult, if not impossible, to achieve in the field.

Two experiments were performed. In the first, a single intersection with four approaches, in which each approach had one bus stop and a different number of lanes, was simulated under different passenger car volumes, bus volumes, bus stop locations, and bus loading-time conditions. In the second, three urban networks, one in Washington, D.C., and two in Chicago, were simulated on NETSIM. The purpose of the first experiment was to compare the condition of a bus turnout with the condition of no bus turnout under a limited set of independent variables. The purpose of the second experiment was to indicate what might be expected to occur in an actual street network if unprotected bus stops were replaced with bus turnouts.

NETSIM CHANGES

NETSIM does not model the exit of a bus from a turnout by using a gap acceptance algorithm. Instead, the user is supposed to input a value of mean dwell time that accounts for time spent in a turnout in addition to passenger loading time. At the end of this dwell time, the bus is then inserted back into the traffic stream whether or not a gap exists. This procedure has two drawbacks from the point of view of this study:

1. The value of dwell time is not known if one is modeling a turnout where one does not exist, and
2. Inserting the bus back into the traffic stream without checking for a gap will cause an overestimate of the delay to the other traffic because, in the model, the bus interferes with the stream, whereas in the real world the bus would wait until there was sufficient room to exit the turnout.

For these reasons, the NETSIM model was modified so that a bus turnout looks for a gap in traffic in the adjacent through lane. Unfortunately, no specific data are available on gap acceptance distributions for buses in turnouts. Richards (11) found that the excess dwell time (i.e., time spent after passenger loading and waiting for a gap) was dependent on the shape of the bay. Observations of some turnouts in the Washington area indicated that the access dwell time is time dependent. Bus drivers appeared more aggressive about forcing their way into the through traffic stream in the morning peak than in the evening peak or off peak. In the absence of any data, the following criterion was used. It was assumed that a bus will exit a bay if a space gap of one bus length plus 2 ft is available in the target lane. If, in fact, larger minimum gaps are observed, the consequence is that turnouts will cause a greater increase in bus travel times than predicted in this work. On the other hand, larger reductions in delay to the other traffic will occur because of less disruption due to slowly accelerating buses.

SINGLE INTERSECTION SETUP

The first set of experiments involved a total of 80 simulation runs of the single intersection network shown in Figure 1. Approach links 2-1, 3-1, 4-1, and 5-1 had 2, 3, 4, and 5 lanes, respectively. There were four bus routes, one on each of the four directions, and each had a stop on the approach link to node 1. Nodes 1, 2, 3, 4, and 5 were signalized with an 80-s cycle and equal splits. Nodes 6, 7, 8, and 9 were unsignalized. Volumes on each of the entry links were chosen so that the volume-to-capacity ratios (V/C's) on all approach links to 1 were approximately equal for each volume level.

The independent variables--V/C, bus loading time, bus volume, and station position--were selected as likely to have the greatest bearing on the effect of turnouts. Thus, one would expect no effect if volumes are low and/or if bus volumes are low and/or if loading times are short. It would also be expected that the effect of a turnout at the stop line could differ from midblock because, in the former case, buses may be blocked from exiting by a standing queue.

A total of 80 simulation runs were performed.

Table 1. Tabulation of statistical results, turnouts versus no turnouts.

Test No.	V/C	Bus Stop Location	Bus Load Time (s)	Bus Volume (buses/h)	Mean Time Delay (s/vehicle mile)	Mean Passenger Car Fuel Difference (miles/gal)	Mean Bus Speed Difference (miles/h)
1	All	All	All	All	-35.3[a]	+0.26[a]	-0.21[a]
2	All	All	30	40	-80.1[a]	+0.52[a]	+0.043
3	All	All	15	40	-75.0[a]	+0.32[a]	-0.142[a]
4	All	All	30	20	-18.1[a]	+0.18[a]	-0.373[a]
5	All	All	15	20	-3.2	+0.037	-0.387[a]
6	0.95	All	All	All	-51.5[a]	+0.33[a]	-0.21[a]
7	0.95	All	30	40	-123.5[a]	+0.70[a]	-0.18[a]
8	0.95	All	15	40	-57.8[a]	+0.40[a]	-0.033
9	0.95	All	30	20	-25.6[a]	+0.22[a]	-0.42[a]
10	0.95	All	15	20	+0.729	-0.021	-0.54[a]
11	0.75	All	All	All	-10.8[a]	+0.17[a]	-0.22[a]
12	0.75	All	30	40	-15.0[a]	+0.22[a]	-0.16
13	0.75	All	15	40	-12.3[a]	+0.20[a]	-0.29[a]
14	0.75	All	30	20	-6.9	+0.11[a]	-0.29[a]
15	0.75	All	15	20	-9.1[a]	+0.12[a]	-0.15
16	All	Near	All	All	-48.1[a]	+0.30[a]	-0.21[a]
17	All	Near	30	40	-119.1[a]	+0.65[a]	+0.22
18	All	Near	15	40	-55.1[a]	+0.40[a]	+0.005
19	All	Near	30	20	-21.9[a]	+0.19[a]	-0.53[a]
20	All	Near	15	20	+3.86	-0.030	-0.54[a]
21	All	Midblock	All	All	-22.4[a]	+0.22[a]	-0.22[a]
22	All	Midblock	30	40	-41.0[a]	+0.38[a]	-0.14[a]
23	All	Midblock	15	40	-24.1[a]	+0.25[a]	-0.28[a]
24	All	Midblock	30	20	-14.4[a]	+0.16[a]	-0.21[a]
25	All	Midblock	15	20	-10.3[a]	+0.10[a]	-0.24[a]

[a]Indicates significant difference at 5 percent level.

Table 2. Tabulation of statistical results, turnout versus no turnout.

		Relative Differences		
Test No.	Test Data Description	Mean Difference of Delay Difference (s/vehicle mile)	Mean Difference of Passenger Car Fuel Difference (miles/gal)	Mean Difference of Bus Speed Difference (mph)
1	High volume versus low volume	+40.3[a]	-0.13	-0.01
2	Near side versus far side	+24.3	-0.06	-0.03

[a]Indicates significant difference at 5 percent.

They consisted of the following:

1. A total of 32 runs for two V/C levels, two bus volume levels, two bus loading-time levels, and two bus station locations for the two choices of turnout or no turnout;
2. Thirty-two replicate runs (by using different random number seeds) of 1; and
3. Sixteen replication runs for the higher volume levels in 1.

The V/C levels chosen were V/C = 0.95 and 0.75. Bus volume levels chosen were 40 and 20 buses/h. Loading-time levels chosen were 30 and 15 s. Near side stops located at the stop line and midblock stops located 180 ft from the stop line were tested. The high level of 40 buses/h and 30-s loading times was chosen by using the data from the three networks described in the next section.

Four measures of effectiveness (MOEs) were chosen for comparing the turnout and no turnout situations:

1. Total delay per vehicle mile in seconds per vehicle mile--all vehicles;
2. Fuel efficiency in miles per gallon--passenger cars;
3. Average route speed in miles per hour--buses; and
4. Fuel efficiency in miles per gallon--buses.

STATISTICAL ANALYSIS OF SINGLE INTERSECTION EXPERIMENTS

In order to make the most efficient use of the available computer runs, it was decided to use the matched-pairs signed-rank test described by Wilcoxon (13). This test is particularly useful in cases where the effect of a single treatment (here the use of a bus turnout) is being determined and where varying values of independent variables (e.g., bus loading times) are not expected to reverse the direction of the effect (i.e., it is not expected that a bus turnout would improve traffic at long loading times and make it worse at short loading times). Details of the test are as follows. The Wilcoxon matched-pairs test consists of the following:

1. Arrange the data in two columns: column 1, which consists of data from the turnout case, and column 2, which consists of data from the no-turnout case.
2. Pair off the elements of columns 1 and 2 based on identical values of independent variables.
3. Subtract, pairwise, column 2 from column 1 and rank the differences independently of the signs.
4. Form the rank sums of the positive differences and the rank sums of the negative differences.
5. If the number of matched pairs that have nonzero differences is less than 16, compare the

Table 3. Network results.

Network	Protected Stops	Networkwide Delay per Vehicle Mile (s/vehicle mile)	Networkwide Passenger Car Fuel Efficiency (miles/gal)	Networkwide Bus Delay (s/vehicle mile)	Networkwide Bus Fuel Efficiency (miles/gal)
K Street	No	215.4	7.54	118.2	4.23
	Yes	202.8	7.69	121.1	4.23
North Michigan Avenue	No	237.6	7.58	183.6	4.07
	Yes	231.2	7.75	203.4	4.08
South Halsted Street	No	61.2	12.91	73.2	4.76
	Yes	55.8	13.27	76.2	4.81

smaller of the positive and negative rank sums against the table values in Snedecor and Cochran (13).

6. If the number of matched pairs that have nonzero differences is 16 or greater, compare the statistic Z given by the following equation with the standard normal table, i.e.,

$$Z = [|\mu - T| - (1/2)]/\sigma \quad (1)$$

where

$$\mu = n(n+1)/4 \quad (2)$$

and

$$\sigma = [\mu(2n+1)/6]^{1/2} \quad (3)$$

where n is the number of pairs that have nonzero differences and T is the smaller of the positive or negative rank sums.

Two types of analysis were made:

1. A determination of whether the presence of a bus turnout had a statistically significant effect on MOEs for different sets of values of the independent variables, and
2. A determination of whether the magnitude of the effects of bus turnouts differed significantly from one set of independent variables to another.

Table 1 gives the results for tests of significance of bus turnout effects. It should be noted that a positive sign for a mean difference indicates that the value of the MOE for the case with a turnout is larger than the value for the case without a turnout. Thus, a negative value for a delay difference indicates that bus turnouts improved delay; a positive value for a fuel-efficiency difference indicates that bus turnouts improved fuel efficiency.

Table 2 gives the results for tests of significance for the effect of independent variables on the relative effects of bus turnouts. Here, for example, we try to discern whether bus turnouts give a greater reduction in delay at higher rather than lower V/C's.

DISCUSSION OF SINGLE INTERSECTION RESULTS

A number of conclusions can be drawn from looking at Tables 1 and 2:

1. There is some potential for fuel savings for nonbus traffic where bus turnouts are used. However, the savings are rather small, as the overall saving of 0.26 mile/gal is about 4 percent of the mean fuel efficiency experienced by passenger car traffic.
2. The V/C has a substantial effect on the benefits obtained from turnouts. The mean delay reduction of all traffic went from 51.5 s/vehicle-mile at V/C = 0.95 to 10.3 s/vehicle-mile at V/C = 0.75. The fuel-efficiency benefit for passenger cars went from 0.33 mile/gal at V/C = 0.95 to 0.17 mile/gal at V/C = 0.75.
3. Benefits derived from turnouts are strongly dependent on high bus volumes and loading times. No significant improvement in MOEs was found at the lowest combined bus volume and loading-time level.
4. No significant difference was found in benefits derived from turnouts at either near side or midblock stops.
5. Turnouts decreased average bus speed; however, the reduction was small, being only about 1-3 percent.

NETWORK DEMONSTRATION RUNS

Although the intersection results described in the previous sections are of interest, it was felt that practitioners would profit from demonstrations of the effects of bus turnouts in actual urban street systems. Three networks with actual bus data were available in the form of input cards to the NETSIM program. They included a network from Washington, D.C., centered around K Street, N.W. (shown in Figure 2); a network from Chicago centered around North Michigan Avenue (shown in Figure 3); and another network from Chicago centered around South Halstead Street (shown in Figure 4). These three networks include a wide variety of conditions with differing values for the independent variables tested in the previous sections.

Six runs were performed, one of each network for each situation: all bus stops with turnouts or no bus stops with turnouts. Results are given in Table 3.

DISCUSSION OF NETWORK RESULTS

A number of conclusions can be drawn from looking at Table 3. In particular, it is evident that the results obtained through use of a turnout for the single intersection hold up when aggregated over a network. Thus, there is some improvement in fuel efficiency, although small, and some reduction in delay for the nonbus traffic. Again little, if any, change in bus fuel efficiency was seen, but generally a small increase in bus travel times was observed.

CONCLUSIONS

From the results of this work, it can be concluded that installing turnouts for buses has the potential to reduce traffic congestion and thus increase fuel efficiency. However, the congestion reductions become substantial only at high V/C's, high bus volumes, and high passenger loading times. It can also be concluded that the relative improvement due to turnouts is not dependent on the location of the bus

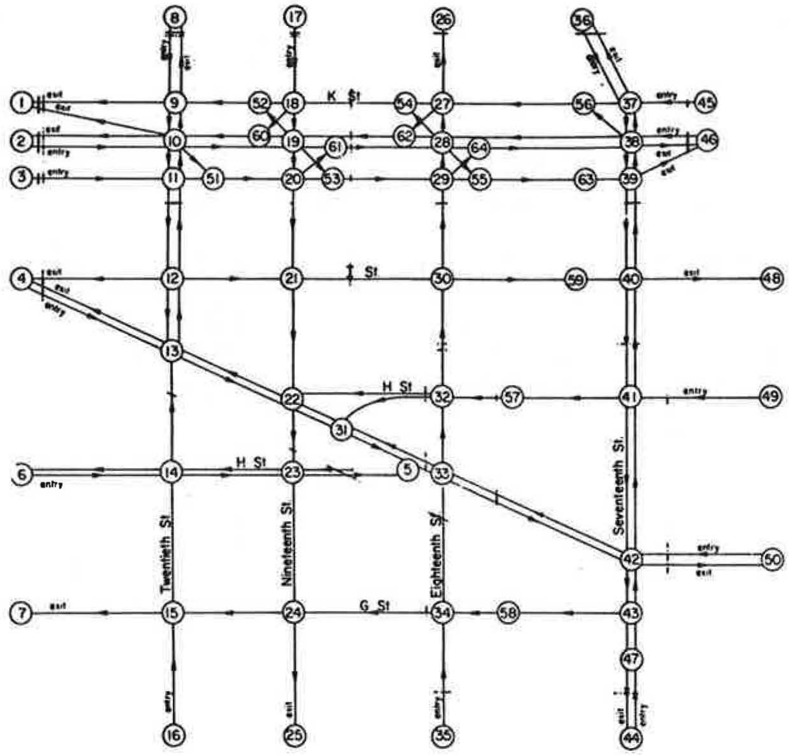


Figure 2. K Street network.

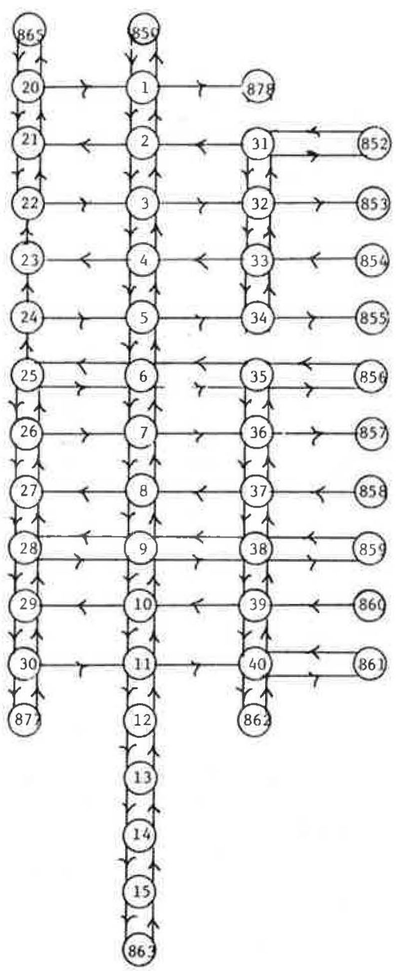


Figure 3. North Michigan Avenue network.

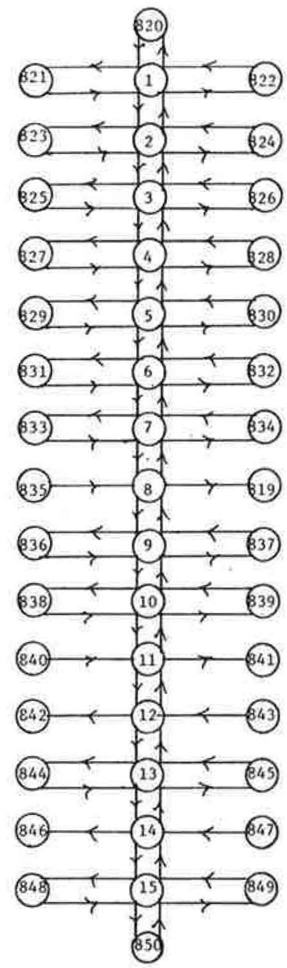


Figure 4. South Halsted Street network.

factors, as discussed elsewhere (5-7).

REFERENCES

1. E.B. Lieberman and S.L. Cohen. New Technique for Evaluating Urban Traffic Energy Consumption and Emissions. TRB, Transportation Research Record 599, 1976, pp. 41-45.
2. Honeywell Traffic Management Center. Fuel Consumption Study. FHWA, Rept. FHWA-RD-76-81, Feb. 1976.
3. S.L. Cohen and G. Euler. Signal Cycle Length and Fuel Consumption and Emissions. TRB, Transportation Research Record 667, 1978, pp. 41-48.
4. J.W. Hall. Traffic Engineering Improvement Priorities for Energy Conservation. Univ. of New Mexico, Albuquerque, April 1980.
5. W.H. Kraft and T.J. Boardman. Location of Bus Stops. Journal of Transportation Engineering, ASCE, Vol. 98, Feb. 1972, pp. 103-116.
6. D.S. Terry and G.J. Thomas. Far Side Stops Are Better. Traffic Engineering, Vol. 41, March 1971, pp. 21-29.
7. N.S. Ghoneim and S.C. Wirasinghe. Near Side or Far Side Bus Stops: A Transit Point of View. TRB, Transportation Research Record 761, 1980, pp. 69-75.
8. J.L. Kay, J.C. Allen, and J.M. Bruggeman. Evaluation of First Generation UTCS/BPS Control Strategy--Volume 1. FHWA, Rept. FHWA-RD-75-27, 1975.
9. J.C. Ludwick. Bus Priority Systems: Simulation and Analysis. U.S. Department of Transportation, Rept. UMTA-VA-06-0026-76-1, 1976.
10. L.J. Pignataro and others. Traffic Control in Oversaturated Street Networks. NCHRP, Rept. 194, 1978, 152 pp.
11. M.J. Richards. The Performance of Bus Bays. Institute of Technology, Cranfield, England, Cranfield CTS Rept., Jan. 1976.
12. R.D. Worrall and E.B. Lieberman. Network Flow Simulation for Urban Traffic Control Systems, Phase II. FHWA, May 1973.
13. G.W. Snedecor and W.G. Cochran. Statsistical Methods. Iowa State Univ. Press, Ames, 1976.

Queuing at Drive-up Windows

W. GORDON DERR, THOMAS E. MULINAZZI, AND BOB L. SMITH

The development of drive-up windows may cause serious problems, where (a) additional fuel is consumed and pollutants are generated by vehicles, in queue, waiting to be served; (b) serious off-site operational problems may occur due to queued vehicles extending onto adjacent streets; and (c) queued vehicles often interfere with the use of on-site parking spaces. Queuing theory is used in the development of a procedure to identify and quantify the magnitude of the problems associated with drive-up windows. Estimated arrival rates and service rates are used to predict a failure rate, i.e., the percentage of time in which a queue length will exceed a selected length of queue. The average time of vehicles in the system, estimated from arrival and service rates, is used to calculate the amounts of air pollutants generated and the fuel consumed. An example problem is included. A brief description of applicable queuing theory is included. Some geometric design guidelines are suggested for the efficient on-site operation of drive-up windows.

The fast-food industry is booming. New businesses are springing up everywhere, and older establishments are adding services to stay competitive. One area where this affects the transportation system is in the development of drive-up windows. Many of these businesses seem to feel it is their inalienable right to have a drive-up window. The traffic engineer and planner have had very few tools to combat drive-up-window developments, which may have an adverse impact on area air quality, fuel consumption, smooth flow of vehicles on site and off, and/or use of on-site parking. Although many businesses, new and old, have adequately sized and shaped lots so that the drive-up-window queues can be handled without harm to other area activities, some do not.

The standing of operating vehicles in queue can cause problems, including consuming fuel and producing air pollutants. In addition, the cars in queue may stack up to the point that they block the flow on site. Of greater concern are those times when the queues extend into area streets. An ancillary problem is the loss of use of those on-site parking spaces adjacent to the area of the queue buildup.

The purpose of this paper is to provide tools that will quantify the impacts in terms that will be understandable to decisionmakers and to provide some guidelines for geometric design of facilities. This paper should assist in the development of local guidelines that will help in determining the situations where drive-up windows should or should not be allowed.

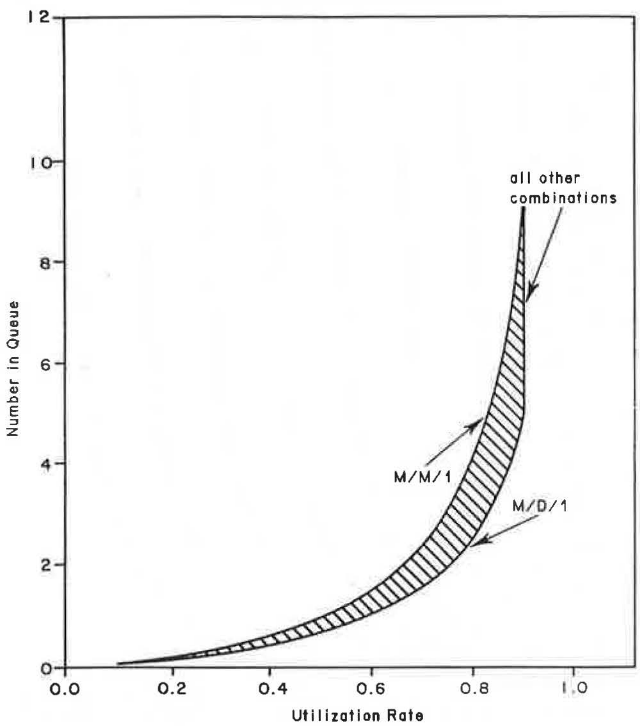
QUEUING THEORY

Queuing theory involves the mathematical study of waiting lines. [General discussions of queuing theory may be found elsewhere (1-3).] Customers show up at some arrival rate (λ) and then stand in the queue until assisted by a server. Service occurs at some service rate (μ). Both arrival rate (λ) and service rate (μ) are expressed as the number of activities per hour. The utilization factor (ρ) is the ratio of the arrival rate (λ) to the service rate (μ).

Every queuing situation can be described by using six descriptors. They are

1. Arrival time distribution--a mathematical description of the times between arrivals;
2. Service time distribution--a mathematical description of the times taken to serve customers;
3. Number of servers--in this case, number of service lanes;
4. Service discipline--in what order will the customers be served; the most popular is first come, first served;
5. System storage capacity--how many vehicles can fit in the area where the queue is located; and
6. Size of the customer population--how many vehicles there are in the community.

Figure 1. Utilization rate versus number in queue M/M/1 and M/D/1.



To simplify the procedures to be discussed, the following assumptions were made:

1. The arrival of customers at the queue is totally random. Each arrival is completely independent from every other.
2. There is a single service lane.
3. The service discipline is first come, first served.
4. The system storage is very large. In fact, it is of medium size. This variable applies to those situations where the area is critical.
5. The customer population is large enough to not affect the results, i.e., there are always cars available to join the queue.

Notation

The notation usually used is A/B/C, where A = arrival distribution, B = service distribution, and C = number of servers.

Distributions

There are a large number of distributions that can be used to describe the arrival and service rates. The primary ones include

M = Markovian (random),
E = Erlang,
H = hyperexponential,
D = deterministic (constant, every service takes the same time), and
G = general (includes those mentioned above plus all of the other possible distributions).

The study of queuing has found that, with Markovian (random) arrivals, the distributions for service rates that bound all of the rest are the Markovian and the deterministic (constant). Figure 1 shows a plot of the utilization factor (ρ) versus the average number of cars in the queue. Because the distribution of the service rates is unknown, the M/M/1 case is used, as it represents the worst case estimated of the queue length. The narrow width of the band between the M/M/1 line and the M/D/1 line provides confidence that the findings should be realistic. The margin of error is likely within that of the accuracy of the estimates of the arrival rates (λ) and service rates (μ).

Design Chart Development

The design charts were developed by using queuing theory for the M/M/1 case. M/M/1 denotes random arrivals and random service times with a single server lane. Figure 2 is a chart that will predict the number of cars in the queue from the utilization rate (ρ) and the percentage of time the system will be allowed to fail, i.e., the percentage of time the queue will be equal to or longer than the number shown. The chart was formed by using the following equation:

$$P(\geq k) = \rho^k \qquad (1)$$

where P ($\geq$ k) is the probability of there being k or more vehicles in the queue, and ρ is the utilization factor. This form is a discrete function that has been converted to a continuous function for ease of use.

Figure 3 is a chart that uses the arrival rate (λ) and the service rate (μ) to find the average time the car will be in the system. It was made by using the following equation:

$$T = (60\ \text{min/h})/(\text{service rate} - \text{arrival rate}) \qquad (2)$$

where T is the average time in queue (min).

PROCEDURE

This section outlines a suggested procedure for using Figures 2 and 3 to find queue lengths, quantity of pollutants generated, and the amount of fuel used.

Step 1: Data Collection

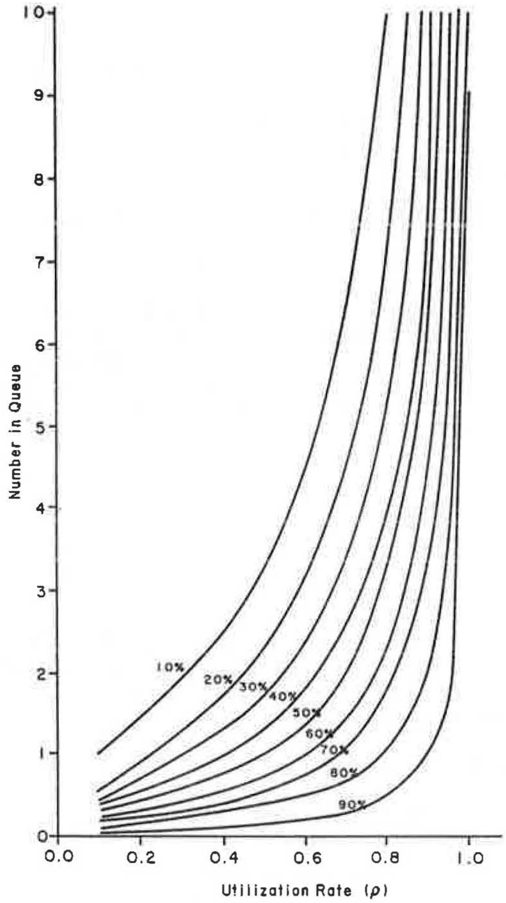
The design charts require the arrival rate (λ) and the service rate (μ) as inputs. Usually, the drive-up window under consideration has no history; therefore, data can only be estimated. If a similar establishment exists in the area, data can be taken at that site. Caution should be used when estimates of these values are provided by the developers, as they would most probably be more oriented to the furtherance of their case rather than the definition of reality.

Data should be collected for two time periods--one being the peak service use and the other being during the peak use of the surrounding streets. Data should be taken in 1- or 2-h blocks with subtotals every 15 min. Although the data are used as arrivals per hour, if the arrival rate varies greatly through the hour, a shorter time period may be required. For example, if the hourly volume is 30 cars, but during one 15-min period 12 cars arrived, the 12 cars/15 min should be expanded to 48 cars/h.

Step 2: Parameter Calculation

The determination of the service rate (μ) is dependent on the form of drive-up service. This paper is concerned with the single-service-lane case, but there may be multiple stations that deal with the lane. The two categories in the single-lane case

Figure 2. Utilization rate versus number in queues for various failure rates—M/M/1.

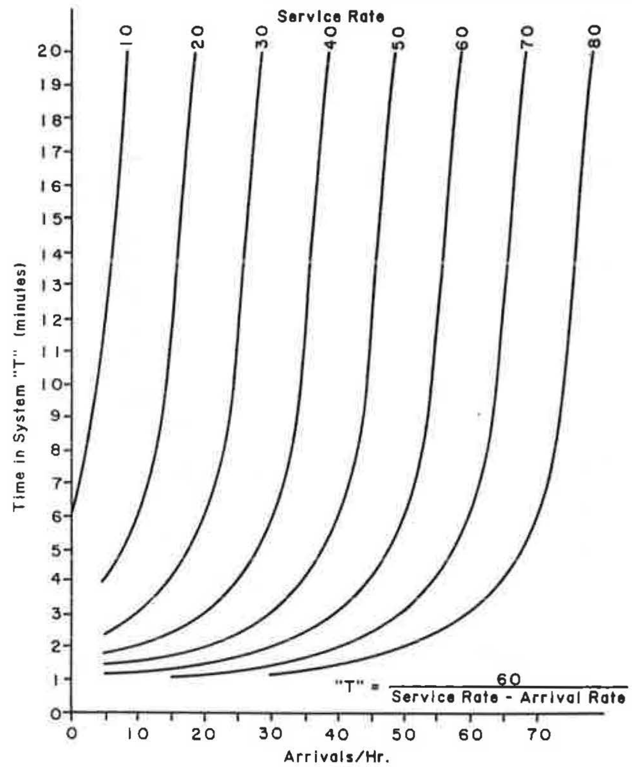


are the single-window and the multiple-window-in-series systems.

In the single-window system, the service rate may be found by taking the inverse of the window average time in minutes found while the system is working at full capacity. This answer would be in services per minute. Multiplying the answer by 60 min in the hour would provide the number of services possible in an hour.

The multiple-window system is actually a number of queues linked together. The provided equations and procedures will not be as accurate for this case, but the answers will be "ballpark" figures. Almost all of the existing systems have two stations. The first station is a menu board where the order is taken. At the second station the money is paid and the food is picked up. At least one national franchise is experimenting with a three-station operation: menu board, pay window, and pickup window. In either situation, the service rate is then used in the analysis. For example, the menu board rate is 120 services/h, and the window works at the rate of 70 services/h. The limiting factor is the window, where 70 services/h would be used as the service rate and the queue lengths found would extend back from the pickup window. If the numbers were reversed and the window was 90 services/h and the menu board rate was 60 services/h, the 60 services/h should be used in the analysis. In the case of the menu board being the limiting factor, the average time in queue found from the figure will be an underestimate. The desired queue length found in the figure will extend from the menu board.

Figure 3. Arrival rate versus time in system for various service rates—M/M/1.



Utilization Factor

The utilization factor (ρ) is found by dividing the arrival rate (λ) by the service rate (μ). For example, if the arrival rate is 45 arrivals/h and the service rate is 60 services/h, the utilization rate would be 45/60 = 0.75.

Failure Rate

The failure rate is a measure of the quality of the service provided. A failure is defined as any length of queue longer than the provided lane. A failure rate of 10 percent would mean that in every hour of operation the line will be longer than was provided for--one-tenth of an hour or 6 min. The failure rate chosen for a location should be related to the problems that would be caused by a failure. For example, if the failure of the queue will back onto an arterial during a peak driving period, a 1 percent failure rate may be appropriate. On the other hand, if failure just makes some of the parking spaces harder to use, a higher failure rate value could be used.

Step 3: Determining Queue Length

Queue length is found by using Figure 2. The values needed are the utilization rate (ρ) and the failure rate. The utilization rate is located along the bottom scale. A vertical line is extended until the selected failure curve is reached. A horizontal line is extended to the left until the vertical scale is intersected. The queue length may then be read. This value will be in a decimal form, like 3.7. Because 3.7 cars are hard to find, the value should be rounded up so that 4 cars are designed for.

If the queue length is known, and the utilization rate may be found or estimated, then this figure may also be used to determine the failure rate of the system.

Step 4: Determining Average Time in System

Average time in the system is used to estimate fuel consumption and the amounts of pollutants generated. It is found by using Figure 3. The input values are arrival rate and service rate. The arrival rate is located on the bottom scale, and a line is extended to the curve that corresponds to the service rate. The horizontal projection of that point gives the average time in the system in minutes per vehicle. Multiplication of this value by the arrival rate will give the total vehicle minutes per hour of standing. Division of this value by 60 will give the vehicle hours of idling per peak hour.

Step 5: Computation of Fuel Consumption and Emissions

The vehicle hours of idling per peak hour drawn from Figure 3 provide the basis for computing fuel consumption and emissions. This value is multiplied by the factors shown in the table below (4):

Item	Amount per Vehicle Hour
Fuel	0.65 gal
Carbon monoxide (CO)	2.43 lb
Hydrocarbons (HO)	0.16 lb
Nitrogen oxide (NOX)	0.05 lb

EXAMPLE PROBLEM

1. Step 1 (data collection): Data were taken at the site for 1 h. The resultant information is given below:

Vehicle No.	Arrival Time	Service Time (min)
1	12:00.5	0.9
2	12:05.2	0.3
3	12:05.9	1.5
4	12:18.7	4.4
5	12:27.5	4.5
6	12:29.6	2.6
7	12:30.4	2.0
8	12:37.3	3.9
9	12:38.4	3.0
10	12:45.4	0.5
11	12:48.7	0.4
12	12:49.4	0.9
--	13:00.5	--
Total		24.9

2. Step 2: The parameter calculations are as follows:

Mean service time = 2.075 min = 24.9/12.

Service rate (μ) = 28.92 vehicles/h = 60/2.075.

Arrival rate (λ) = 12 vehicles/h.

Utilization rate (ρ) = 0.415 = 12/28.92.

3. Step 3: The queue length determinations from Figure 2 are given below:

Failure Rate (%)	Figure 2 Value	No. of Spaces Needed
>50	0.85	1
30	1.4	2
20	1.9	2
>10	2.7	3
5	3.4	4
>1	5.3	6

4. Step 4: The time in the system determinations from Figure 3 are as follows: average time in system = 3.7 min, and total time in system = 44.4 vehicle-min/h.

5. Step 5: The fuel use and emission quantity calculations from the table in the previous section are as follows:

Item	Amount per Vehicle Hour
Fuel consumed	0.48 gal
CO	1.8 lb
HO	0.12 lb
NOX	0.04 lb

GEOMETRIC DESIGN CONSIDERATIONS

A search of the literature has found nothing concerning the geometric design considerations for single-lane drive-up windows at fast-food restaurants, banking institutions, etc. Some authors have discussed the trip generation rates for fast-food restaurants (5,6). Other studies have looked at multilane banking systems (7,8). Surely, some experience-based guidelines must have been written, but they have not made it into the standard traffic engineering references. Therefore, a list of general design considerations follow that are aimed at providing operational efficiency for single-lane drive-ups:

1. When facing the establishment, the drive-up should be located on the left side of the building. This location will result in a counterclockwise flow pattern with the maximum use of the available space and allow the longest queue. (The major problem with wrapping the queue around the building is the conflict with the pedestrians who use the facility.)
2. The drive-up-window operation should have at least two stations, one for ordering and the other for delivery.
3. Storage lengths for each station should be based on the arrival rate and service at that station. If the menu board is the critical activity in the system, then the queue storage for that area should be designed by using the outlined procedure. If the service window is the critical element, the combined service and menu queue length should be checked.
4. It should be noted that a drive-up facility may result in a reduction in the number of effective parking spaces on the existing property due to the queue blocking the parking spaces. Additional land might have to be purchased to meet the parking requirements in the subdivision and/or zoning regulations.
5. There should be a bypass lane or another convenient exit to an existing street so that vehicles not wanting to use the drive-up facility can leave the premises without passing through the drive-up window.
6. The drive-up-window lane should be a minimum of 12 ft wide from face-of-curb to face-of-curb.
7. The turning radius should not be less than 15 ft on any curve used in the drive-up operation.
8. The minimum vertical clearance should be 9 ft to accommodate recreational vehicles and vans.
9. Parking spaces located beyond the end of the drive-up window should be designated for use by those drive-up patrons whose orders are long in preparation. The driver would be told to park and the order would then be brought out to patron's vehicle.

NEEDED RESEARCH

The further testing and refining of the application of queuing theory to both single- and multiple-lane service systems is needed. Further development of the geometric design of parking and queuing areas is needed so that the interference of queued vehicles with the use of parking spaces and/or pedestrians can be minimized. Additional information for estimating arrival rates and service times is needed. The development of a microcomputer program to carry out the analysis would be desirable.

CONCLUSION

This paper has presented some tools and guidelines to help traffic engineers and planners understand the impacts of drive-up windows, as well as to suggest ways in which the negative impacts may be reduced.

REFERENCES

1. F.S. Hillier and G.J. Lieberman. Introduction to Operations Research. Holden-Day, Inc., San Francisco, 1980, pp. 400-484.
2. ITE. Transportation and Traffic Engineering Handbook. Prentice-Hall, Englewood Cliffs, NJ, 1976, pp. 302-388.
3. D.L. Gerlough and M.J. Huber. Traffic Flow Theory. TRB, Special Rept. 165, 1976, pp. 137-173.
4. Procedure for Estimating Highway User Costs, Fuel Consumption, and Air Pollution. FHWA, 1980.
5. R.B. Shaw. Traffic Generation and Fast-Food Restaurants. Traffic Engineering, March 1975, pp. 36-37.
6. R.H. Lopata and S.J. Jaffe. Fast-Food Restaurant Trip Generation: Another Look. ITE Journal, April 1980, pp. 28-31.
7. D.L. Woods and C.J. Merser. Design Criteria for Drive-In Banking Facilities. Traffic Engineering, Dec. 1970, pp. 30-37.
8. P.N. Scifres. Traffic Planning for Drive-In Financial Institutions. Traffic Engineering, Sept. 1975, pp. 21-24.

Influence of Arterial Access Control and Driveway Design on Energy Conservation

JOHN M. MOUNCE

Driveway design standards influence turning maneuver performance and are most critical on arterial streets. The speed disparity between outside lane arterial vehicles and driveway right-turn entry vehicles directly affects both operations and safety. This study used fuel consumption as a measure of effectiveness between minimum, typical, and desirable driveway design standards for the driveway right-turn entry maneuver. A simplistic model analysis illustrated the differences in fuel consumption incurred by arterial vehicles in the outside lane traveling at a given speed (i.e., 35 mph), which are forced to negotiate a deceleration-acceleration speed-change cycle due to right-turning vehicles that enter driveways that exhibit various levels of design standards. The results for the stated condition of 35-mph arterial speed indicate little difference in annual fuel consumption as influenced by design at an arterial-driveway hourly volume product of less than 100 000 vehicles. Between the 100 000 and 500 000 arterial-driveway hourly volume product range there is demonstrated fuel savings incurred through the institution of desirable versus minimum driveway design standards. Above a 500 000 arterial-driveway hourly volume product, the fuel savings are substantial and warrant the application of desirable driveway standards on all such facilities, with special consideration given to parallel deceleration right-turn lanes. Further research is needed to fully simulate and quantify the arterial-driveway traffic operational interaction for the right-turn driveway entry maneuver.

Ordinances that manifest regulatory policy and procedures for access control have been instituted in most U.S. cities with populations greater than 25 000 persons. These statutory guidelines have been based on safety and operational criteria that have served as the measures of effectiveness for driveway design on urban streets and highways.

Studies in Texas (1) have indicated that there exists a great inconsistency in the objectives of driveway regulations (safety, operations, etc.) and a general lack of uniformity in design standards and specifications. Table 1 (1) presents a summary of both commercial and residential driveway design standards from 34 Texas municipalities. As shown, there is a considerable range in both the importance associated with a specific driveway design element under regulation and the standard values designated to any particular element. Many cities assign absolute minimum and/or maximum design limits but do not state desirable design criteria. Most cities do not recognize the interaction between driveway design features. This seems to be reflective of national trends as well.

There is a need to relate the individual and interactive effects of standards for driveway design elements to a single measure of effectiveness. In recent years, energy conservation has become increasingly important as a measure of effectiveness to various federal agencies, as can be seen by the Emergency Energy Conservation Act of 1979 and Executive Order 12185 of the Federal Highway Administration (FHWA), December 17, 1978. The objective of this paper is to assess driveway design standards on arterial streets in terms of the affected operational speed differential between arterial vehicles and vehicles turning right into driveways of various design standards. Fuel consumption of arterial vehicles forced to decelerate due to driveway entry vehicles is calculated and compared for various design standards.

OPTIMAL DRIVEWAY DESIGN

Optimal driveway design, and subsequent turning maneuver performance, is extremely critical on arterial streets. Arterial streets constitute those streets without full access control that carry traffic entering, leaving, or passing through an urban area or intra-area traffic between the central business district and outlying residential areas, between major inner city communities, or between major suburban centers. Primary arterial streets serve very high traffic volumes at moderate speeds and are

Table 1. Summary of commercial and residential driveway design standards in 34 Texas cities.

	Commercial		Residential	
Design Feature	Range of Values (ft)	Cities with no Guidelines (%)	Range of Values (ft)	Cities with no Guidelines (%)
Minimum throat width	12-25	37	9-20	44
Maximum throat width	24-45	3	12-40	9
Minimum curb return radius	2-15	9	1.5-10	15
Maximum curb return radius	5-50	44	4-50	43
Minimum spacing between driveways	10-45	18	8-30	26
Minimum spacing to intersections	20-100	20	20-100	23

Table 2. Arterial driveway design standards—curb return radius.

	Curb Return Radius (ft)					
	One-Way Driveways			Two-Way Driveways		
Driveway Type	Typical	Desirable	Minimum	Desirable	Maximum	Minimum
Residential	5	13	2.5	5	15	2.5
Commercial	20	30	10	20	30	10
Industrial	30	-	15	30	-	15

Table 3. Arterial driveway design standards—throat width.

	Width (ft)						
	One-Way Driveways						
	Typical				Two-Way Driveways		
Driveway Type	Entry Drive	Exit Drive	Desirable	Minimum	Desirable	Maximum	Minimum
Residential	15	15	20	12	20	25	12
Commercial	18	16	20	15	30	35	25
Industrial	20	20	25	15	35	40	30

vital transportation links within an urban area. In most jurisdictions, every land parcel abutting an arterial is guaranteed access. Conflicts between through and turning vehicles pose a major operational and safety problem.

Bochner (2) reported that the capacity of a four-lane arterial street is reduced 1 percent for each 2 percent of the traffic that turns between the right lane and unsignalized access points. For example, if a street carries 1200 vehicles/h in a given direction and 120 vehicles turn into driveways and 120 turn out of driveways (20 percent turns), then the capacity in that direction will be reduced by 10 percent. He also stated that, "as the level of design of the driveway is increased (allowing turns to be made at higher speeds), the capacity loss is reduced and level of service on the arterial is maintained."

Various studies (3,4) have supported the fact that speed differential is the major cause of rear-end accidents associated with the driveway turn maneuver. From the standpoint of safety, it has been suggested that the speed differential between the average speed of through traffic and vehicles entering and leaving the arterial be limited to 10 mph or less.

It is desirable that driveway design standards minimize the speed disparity between arterial and driveway turning movements to optimize operational and safety performance. Maximizing driveway turning speed is a viable measure of effectiveness for driveway design elements and may be expressed in terms of fuel consumption (energy conservation).

DRIVEWAY DESIGN ELEMENTS

The Texas Transportation Institute conducted proving-ground studies (5) that assessed the effects of various driveway geometric design elements on speed in the turn maneuver. Recommended values and corresponding effects on turning speeds are discussed independently by design element.

Throat Width and Curb Return Radius

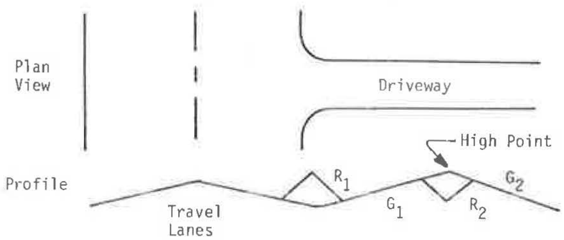
Driveway width and curb return radius interact to affect vehicle speed and path. The selection of an appropriate width must be coordinated with curb return radii selection to achieve desirable driveway operation and safety. Tables 2 and 3 present width and curb return radius requirements for one- and two-way driveways. The desirable values shown should be used whenever possible. If variation from these values is required because of site conditions, the width and radii selected should be as close as possible to the desirable values. The use of both a small width and curb return radius should be avoided. Generally, if the width must be greatly reduced, then curb return radius should be increased, and vice versa. Throat width and curb return radius may individually impact turning speed by ±2 mph.

Angle

The angle at which a driveway intersects the street affects the speed and path of vehicles that use the driveway. Approach angles interact with other design features (e.g., width, curb return radius, throat length, channelization, etc.) to influence driveway operation and maneuvers. Recommended standards are presented in the table below. However, no assessment of impact on turning speed was possible with available data.

Figure 1. Profile key to guidelines for typical driveways on a curbed street.

1) Profile Key



Driveway Type	Angle of Intersection (°) Maximum	Desirable	Typical
Residential	60	70	90
Commercial	45	70	90
Industrial	30	70	90

In regard to the above table, note that all two-way driveways and one-way driveways with unrestricted turning movements should intersect the street at a 90° angle. If site conditions (e.g., terrain, lot size and shape, etc.) will not permit a 90° approach angle, the angle may be reduced, but not below these values: 70° for commercial and industrial driveways and 60° for residential driveways.

At one-way driveways where only right turns are permitted (e.g., one-way driveway pair on a divided street), it may be desirable to flatten the approach angle below 90° to increase entry and exit speeds. Under these conditions, an angle of approximately 60° is recommended, with the following exceptions:

1. At driveways where sidewalk pedestrian traffic is heavy, the approach angle should not be reduced below 70°. Lesser angles encourage high vehicle speed and a pedestrian safety problem may result.
2. If an acceleration or right-turn lane is provided at an exit driveway, the angle may be reduced down to 45°.
3. At industrial driveways that service large trucks, the angle may be reduced to as low as 30° to facilitate driveway operation. Angles less then 30° result in severe visibility limitations and are discouraged.

Profile

Driveway profile is a critical element of driveway design. It influences the speed and path of driveway users and therefore affects driveway operation and safety. It is difficult to recommend a single set of standards for driveway profile, since site conditions vary greatly. Currently, there are no standards available that have received widespread acceptance. Some general profile guidelines for typical driveways on a curbed street are given in the table below (note, the profile key for this table is shown in Figure 1):

Profile Parameter	Desirable	Typical	Minimum
R_1 (ft)	75	50	25
G_1 (in/ft)	0.125	0.25	0.5
R_2 (ft)	100	75	50
G_2 (in/ft)	1	1.75	2.5

Variations from typical values may impact turning speed by ±1 mph.

Other Driveway Design Elements of Influence

Throat Length

Even if existing vehicle storage requirements are minimal, throat length should be as great as practical in order to (a) move the parking and circulation area conflict point away from the driveway entrance and (b) encourage proper use of the driveway, in the case of two-way driveways, by exiting traffic. A minimum throat length of 25 ft is suggested.

Number of Driveways

As greater numbers of driveways are constructed along a street, the accident rate increases and roadway capacity decreases. Therefore, every development (or land parcel) should have only the minimum number of driveways needed to efficiently handle the traffic volumes generated by the development.

Spacing

Driveways should be spaced far enough apart so that conflicting movements at adjacent driveways do not overlap, thus increasing accident potential and/or reducing roadway capacity. Desirable minimum driveway spacings for arterials with a speed range of 35-40 mph is 200 ft measured from driveway throat to driveway throat.

Summary

The following general statements summarize the individual and combined effects of driveway design elements on right-turn entry speed:

1. Right-turn entry speed decreases as the available width and/or curb return radius decreases;
2. Right-turn entry speed increases as the angle of entry is decreased to an optimum level; further decreases promote excessive driveway speeds; and
3. Right-turn entry speed increases as relative disparities in profile are minimized.

Estimates of the quantifiable effects of driveway design standards on right-turn entry speeds may be drawn from the field investigations by the Texas Transportation Institute (5). For a driveway constructed under typical standards, which exhibits a nominal turning speed of approximately 10 mph, the comparative cumulative speed effect between minimum and desirable standards is given in the table below (note, the nominal turning speed under typical standards is assumed to be 10 mph, and the cumulative turning speeds for the various levels are as follows: minimum = 5 mph, typical = 10 mph, and desirable = 15 mph):

Design Element	Effect on Turning Speed (mph) Minimum	Typical	Desirable
Throat width	-2	0	+2
Radius	-2	0	+2
Angle	0	0	0
Profile	-1	0	+1

IMPACT ON FUEL CONSUMPTION

The evaluation of the impact of specified levels of driveway design standards on fuel consumption consists of a limited, simplistic model of the driveway right-turn entry maneuver. Arterial operating speed is assumed to be 35 mph with volume ranges from 400 to 1600 vehicles/h in the outside lane during the peak hour. Driveway right-turn entry volumes represented 5, 10, and 20 percent of arterial volume under turning speeds of 5 mph (minimum), 10 mph (typical), and 15 mph (desirable), respectively, as dictated by design.

Fuel consumption is calculated based on the disparity between arterial operating speed and driveway

Table 4. Summary of arterial traffic volume impacted by driveway right-turn entry volume.

Peak-Hour Arterial Through Street Volume (vehicles/h)	Driveway Right-Turn Entries (%)	Driveway Design Standards: Minimum (5 mph) P_T	EFV	V_T	Typical (10 mph) P_T	EFV	V_T	Desirable (15 mph) P_T	EFV	V_T
1600	5	0.048	4	308	0.048	3	231	0.048	2	154
	10	0.099	4	632	0.099	3	474	0.099	2	316
	20	0.160	4	1024	0.160	3	768	0.160	2	512
1200	5	0.048	3	174	0.047	2	112	0.047	1	56
	10	0.098	3	354	0.098	2	236	0.097	1	116
	20	0.159	3	573	0.158	2	380	0.157	1	188
800	5	0.047	2	76	0.046	1	37	0.044	1	35
	10	0.097	2	156	0.095	1	76	0.091	1	73
	20	0.156	2	250	0.153	1	122	0.148	1	118
400	5	0.039	1	16	0.036	1	14	0.018	1	7
	10	0.081	1	32	0.073	1	29	0.037	1	15
	20	0.130	1	52	0.118	1	47	0.059	1	24

Note: P_T = total probability of any following vehicle in the outside lane of an arterial being impacted by a driveway right-turn entry, EFV = number of equivalent following vehicles impacted by a driveway right-turn entry, and V_T = total number of arterial vehicles in the outside lane impacted per hour for the total number of driveway right-turn entries specified.

right-turn entry maneuver speed. This follows as a function of arterial volumes and driveway right-turn volume. An estimate of the number of arterial vehicles impacted by driveway turning vehicles for conditions of volume and speed may be formulated through a series of probability statements.

The first probability involved in calculating speed impact is the probability that the lead vehicle of any two vehicles will actually be a turning vehicle (P_{Turn}). This binomial condition may be calculated for each driveway turn by the relation

$$P_{Turn} = Q_{Turn}/Q_A \quad (1)$$

where Q_A is the arterial volume (vehicles/h) in the outside lane, and Q_{Turn} is the driveway turning volumes (vehicles/h).

The second probability involves the determination of the time headway between two vehicles such that, given designated operating conditions, no impact of reduction in through arterial speed will be incurred by a vehicle (P_{Impact}). Physically, this is the time headway between vehicles sufficient that, as the lead vehicle decelerates to negotiate the driveway turning maneuver, the following vehicle can maintain its operating speed unimpeded to within a minimum 2-s headway of the lead vehicle.

Any time headway less than the derived values, under specified conditions (by design constraints), will generate a speed impact. These critical headways, which assume a deceleration rate of -3 ft/s² nominal, are calculated as T_5 = 16.67 s (minimum standard, 5 mph driveway turn), T_{10} = 14.23 s (typical standard, 10 mph driveway turn), and T_{15} = 11.78 s (desirable standard, 15 mph driveway turn).

By substituting the critical deceleration time headway (T) for all specified conditions, the following equation allows the calculation of the probability of two consecutive vehicles that exhibit a gap headway of size T or less, which results in an impact to the following vehicle:

$$P_{Impact} = 1 - e^{-q_A T} \quad (2)$$

where

P_{Impact} = probability of speed impediment to following vehicle,
q_A = arterial volume (vehicles/s) in the outside lane,
T = critical gap headway (s), and
e = logarithmic constant = 2.718 28.

A third probability--the probability that the following vehicle of any two vehicles will be a straight or through vehicle (P_{Thru})--must also be considered. This binomial condition may be calculated by

$$P_{Thru} = 1 - P_{Turn} \quad (3)$$

The probability that any two vehicles on the arterial will be involved in a turn maneuver, such that a speed-reduction cycle is imposed on the following through vehicle under given operating conditions, is the multiplicative function of the three previously discussed probabilities (P_T). The equation is as follows:

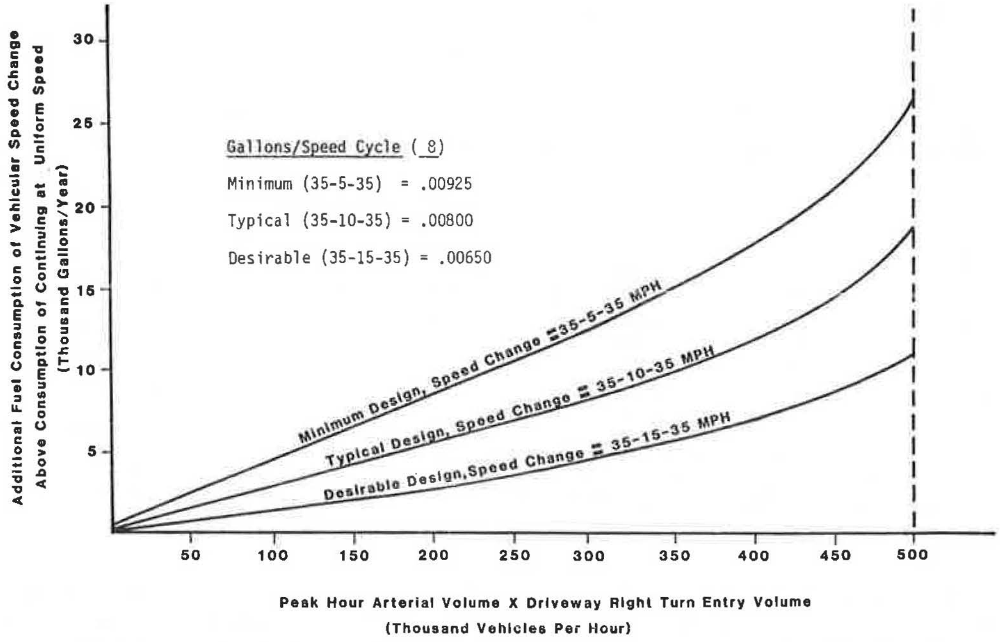
$$P_{Total} = P_{Turn} \times P_{Impact} \times P_{Thru} \quad (4)$$

The total probability function multiplied by the arterial volume in the outside lane provides an estimate of the number of immediate vehicles impacted per hour. This does not give an indication of the extent of impact (degree of imposed deceleration/acceleration speed cycle) on this vehicle or on subsequent vehicles. An estimate of the equivalent following vehicles (EFV) impacted may be derived by a cursory comparison of the mean gap headway for designated arterial volumes with critical impact gap as established by design levels. The extent of impact is assumed to be an average between the first and last vehicles impacted.

Table 4 summarizes those calculated values for the total probability (P_T) of any arterial vehicle in the outside lane being impacted by a driveway right-turn entry, the number of EFV subsequently impacted by a driveway right-turn entry, and the total number of arterial vehicles (V_T) in the outside lane impacted per hour for levels of driveway design standards and arterial-driveway volume combinations. The total vehicles impacted are converted to gallons of fuel consumed by referenced standards (6) for the driveway turn speed cycles specified. Figure 2 depicts the annual fuel consumption plotted against the parameters of arterial street volume in the outside lane multiplied by the driveway right-turn entry volume for each specified design level. Conversion from peak-hour fuel consumption to annual fuel consumption was accomplished by using a peak-hour factor of 0.10 multiplied by 5 days/week and 52 weeks/year.

It should be noted that the curves shown on this graph are for only one condition of arterial operat-

Figure 2. Annual additional fuel consumption as affected by driveway design standard levels and arterial-driveway hourly volume product.



ing speed--35 mph. A more complete analysis requires the extension of this methodology to produce a family of curves that represent a range of arterial operating speeds.

CONCLUSIONS

Figure 2 reveals there to be little apparent difference in the effect on fuel consumption between the previously designated minimum and desirable driveway standards below the 100 000 arterial-driveway hourly volume product level. This represents an arterial facility with a maximum of approximately 750 vehicles/h in the outside lane and 150 driveway entry right turns per hour.

Between the 100 000 and 500 000 arterial-driveway hourly volume product there is demonstrated fuel savings incurred through the institution of desirable over minimum driveway design standards. At the maximum combined volume level of approximately 1600 vehicles/h on an arterial in the outside lane and more than 300 vehicles/h driveway entry right turns, additional annual fuel consumption, which represents the measure of effectiveness between driveway design standards, approaches 15 000 gal/year.

Above the 500 000 arterial-driveway hourly volume product level, the annual fuel savings to be gained through employing desirable driveway design standards are obvious and continue to increase dramatically as volumes increase. Also, above this volume level, if generated by a single driveway, serious consideration should be given to the construction of parallel deceleration right-turn lanes, which can be justified through fuel savings that approach 30 000 gal annually.

It should be stated again when reviewing these values that they are the result of a very simplistic modeling analysis with various assumptions subject to question. However, the generalized direction and magnitude of the effect is reasonable and supportive of the specified conclusions. Also, this analysis made no mention of the documented safety benefits derived when speed disparity between through and turning vehicles is minimized as a result of desirable driveway design standards.

RECOMMENDATIONS

There is indication of the potential for substantial fuel savings through the upgrading of driveway design standards for the right-turn entry maneuver. Tentative arterial-driveway design volume product values for application of standards are designated; however, the limited scope of this study precludes the use of these results.

Further research is needed beyond that previously cited (5) to establish in more detail the specific operational effects of individual and combined driveway design elements. Also, there is a need to collect data associated with the arterial-driveway traffic operational interaction needed to calibrate a simulation model for depicting, in more quantifiable detail, the effects on the traffic stream of those vehicles impacted by a right-turn driveway entry maneuver.

REFERENCES

1. S.H. Richards. Guidelines for Urban Driveway Regulation, Volume 3. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. 5183-3, Oct. 1980.
2. B.S. Bochner. Regulations of Driveway Access to Arterial Streets. Public Works, Oct. 1978.
3. D. Soloman. Accidents on Main Rural Highways Related to Speed, Driver, and Vehicle. Public Works, July 1964.
4. W.W. McGuirk and G.T. Satterly, Jr. Evaluation of Factors Influencing Driveway Accidents. TRB, Transportation Research Record 601, 1976, pp. 66-72.
5. S.H. Richards. Driveway Design and Operations, Volume 3. Texas Transportation Institute, Texas A&M Univ., College Station, Rept. 5183-2, April 1980.
6. C.W. Dale. Procedure for Estimating Highway User Costs, Fuel Consumption, and Air Pollution. FHWA, March 1980; revised April 1981.

Effect of Left-Turn Bays at Signalized Intersections on Fuel Consumption

JOHN R. TOBIN AND PATRICK T. McCOY

A consequence of the reductions in delay and stops that result from the provision of left-turn bays is a reduction in fuel consumption. Less delay and fewer stops cause less fuel to be consumed by vehicle-idling and speed-change cycles. The objective of this research was to estimate the effect of the addition of left-turn bays on fuel consumption on the approaches to two-phase, signalized intersections. These effects were evaluated over a range of approach, opposing, and left-turn volumes for both left-turn bays with and without a protected left-turn phase. However, the scope of the study was limited to isolated intersections of two-way, two-lane streets. A procedure based on the critical-movement-analysis method presented in Transportation Research Board Circular 212 was used to evaluate the effect of left-turn bays. With this technique, the fuel savings that result from the addition of left-turn bays with and without protected phases for 3360 combinations of traffic conditions were computed. These fuel savings ranged from 0.3 to 10 gal/h for traffic on the street on which the left-turn bays were added. A multiple regression analysis of these data determined that the fuel savings that result from the addition of left-turn bays with and without protected phases were curvilinear functions of the initial critical lane volume, opposing volume, and left-turn percentages. Nomographs of these relations were constructed to facilitate the calculation of potential fuel savings that result from the addition of left-turn bays at signalized intersections.

Left-turn bays are provided on approaches to signalized intersections to increase the capacity of the intersections and improve the efficiency of traffic flow through them. The primary function of the left-turn bay is to remove the deceleration and storage of left-turning vehicles from the through lanes and thus enable through and right-turning vehicles to move past them without conflict and delay. Among the benefits derived from the provision of these left-turn bays are reductions in delay and stops. Previous research (1) has indicated that the amounts of the reductions are functions of the approach, opposing, and left-turn volumes.

A consequence of the reductions in delay and stops that result from the provision of left-turn bays is a reduction in fuel consumption. Less delay and fewer stops cause less fuel to be consumed by vehicle-idling and speed-change cycles. The objective of this study was to estimate the effect of the provision of left-turn bays on the approaches to signalized intersections on fuel consumption. These effects were evaluated over a range of approach, opposing, and left-turn volumes for both left-turn bays with and without an exclusive signal phase. However, the scope of the study was limited to isolated intersections of two-way, two-lane streets with approach speeds of 30 mph. This paper presents the procedure, findings, and conclusions of this study.

PROCEDURE

Previous studies (2) of traffic operations at isolated signalized intersections have shown delay and stops to be functions of signal timing as well as lane configuration. Therefore, in an effort to isolate the effect of left-turn bays, the critical-movement-analysis method presented in Transportation Research Board (TRB) Circular 212 (3) served as the basis of the procedure used in this study.

For each combination of approach, opposing, cross-street, and left-turn volumes considered in this study, the critical-movement-analysis method was used to compute the intersection volume-to-capacity ratios with and without left-turn bays on both approaches of one of the two intersecting streets. Each of these volume-to-capacity ratios was then expressed in terms of stopped-time delay per vehicle by applying the relation between stopped-time delay and volume-to-capacity ratio presented in TRB Circular 212. The results are given in the table below (3):

Volume-to-Capacity Ratio	Stopped-Time Delay (s/vehicle)
0.00-0.60	0.0-16.0
0.61-0.70	16.1-22.0
0.71-0.80	22.1-28.0
0.81-0.90	28.1-35.0
0.91-1.00	35.1-40.0

Next, the following empirical relation determined by Reilly and others (4) was used to compute the percentage of vehicles stopped from the stopped-time delay values:

$$PVS = \log_{10}(1.3 \times STD) \times 55 - 14 \quad (1)$$

where PVS is the percentage of vehicle stopped, and STD is the stopped-time delay (s/vehicle).

The difference in the stopped-time delay and percentages of vehicles stopped with and without left-turn bays were then computed. These differences were multiplied by the approach volumes on the street on which the left-turn bays had been added to determine the reductions in vehicle hours of delay and number of vehicles stopped that were caused by the provision of left-turn bays. The resultant savings in fuel consumption were then computed by using the following equation, which was based on fuel-consumption data for light-duty vehicles presented by Dale (5):

$$f = 0.65D + (9.3\ S_t/1000) + (1.3\ S_\ell/1000) \quad (2)$$

where

- f = fuel savings for traffic on street on which left-turn bays were added (gal/h),
- D = reduction in stopped-time delay on street on which left-turn bays were added (vehicle-h/h),
- S_t = reduction in through vehicle stops on street on which left-turn bays were added (stops/h),
- S_ℓ = reduction in left-turn vehicle stops on street on which left-turn bays were added (stops/h),
- 0.65 = fuel-consumption rate of idling vehicle (gal/vehicle-h),
- 9.3 = stop-go fuel-consumption rate for stop from 30 mph (gal/1000 stop-go cycles), and
- 1.3 = stop-go fuel-consumption rate for stop from 10 mph (gal/1000 stop-go cycles).

As indicated in Equation 2, through vehicles were assumed to stop from and return to a speed of 30 mph and left-turning vehicles were assumed to stop from and return to a speed of 10 mph.

In this study, two basic cases were examined: (a) the addition of left-turn bays without a protected left-turn phase and (b) the addition of left-turn bays with a protected left-turn phase. In both cases, the street on which the left-turn bays were added initially had no protected left-turn phase. For each of the two basic cases, 1680 combinations of initial-condition (i.e., without left-turn bays) values of the following five variables were analyzed:

1. Initial level of service (C, D, and E),
2. Initial critical lane volume on the street on which left-turn bays were added (level-of-service E = 900-1500 passenger cars/h at 100 passenger cars/h intervals, level-of-service D = 800-1400 passenger cars/h at 100 passenger cars/h intervals, and level-of-service C = 700-1300 passenger cars/h at 100 passenger cars/h intervals),
3. Equivalent lane volume opposing initial critical lane volume on the street on which left-turn bays were added expressed as a percentage of the initial critical lane volume (30-100 percent at 10 percent intervals),
4. Percentage of left turns on the approach with the initial critical lane volume (10-50 percent at 10 percent intervals), and
5. Number of signal phases on the cross street (one and two).

Given the above initial conditions without left-turn bays, the critical-movement-analysis procedure was used to compute the peak-period passenger car volumes that would have yielded the initial level of service. Then, starting with these passenger car volumes, left-turn bays were added and a new level of service was computed. As explained previously, the volume-to-capacity ratios associated with this new level of service and the initial level of service were then used to compute the fuel savings that result from the addition of the left-turn bays. In the analysis of each case, it was assumed that (a) the percentage of left turns on the opposing approach was equal to that on the critical approach, (b) all lanes were 12 ft wide, (c) the peak-hour factor was 1.00, (d) the percentage of trucks and buses was zero, and (f) there were no right turns.

FINDINGS

The fuel savings that result from the provision of left-turn bays for the 3360 cases described above range from 0.3 to 10 gal/h on the street on which the left-turn bays were added. These data were analyzed to determine the relation between the fuel savings and the initial conditions. A stepwise multiple linear regression analysis was conducted with fuel savings as the dependent variable and the initial conditions as the independent variables. The regression analysis was applied to the data for each of the basic cases and to their combined data sets.

As a result of the regression analysis, the following relations were found to be statistically significant (α = 0.10). For left-turn bays without a protected phase,

$$f = -7.41 + (5.71V/10^3) + (2.48V_0/10^3) + 3.17 \log_{10} PLT - (1.24PLT^3/10^5) \quad (3)$$

For left-turn bays with a protected phase,

$$f = -8.60 + (6.12V/10^3) + (1.70V_0/10^3) + 3.591 \log_{10} PLT - (1.01PLT^3/10^5) \quad (4)$$

where

f = fuel savings for traffic on street on which left-turn bays were added (gal/h),
V = initial critical lane volume on street on which left-turn bays were added (passenger cars/h),
V_0 = equivalent lane volume opposing initial critical lane volume on street on which left-turn bays were added (passenger cars/h), and
PLT = percentage of left turns on initial critical-lane-volume approach.

These equations accounted for 90 percent of the variance of the fuel savings. Although these two equations contain the same independent variables with similar coefficients, it was determined that they are statistically different at the 10 percent level of significance.

The relations expressed in Equations 3 and 4 are consistent with the expectations that fuel savings should increase with increasing approach volumes and that fuel savings should increase with increasing opposing volumes. Also, as expected, the influence of the opposing volume is less in the case of left-turn bays with a protected phase than it is in the case of left-turn bays without a protected phase because, in the critical-movement-analysis method, the left-turn volume adjustment for protected left-turn movements is independent of the opposing volume (3).

Fuel savings would also be expected to be positively correlated with the percentage of left turns. But, in Equations 3 and 4, the percentage of left turns has both a positive and a negative influence on fuel savings. The negative influence is due to the fact that, in the critical-movement-analysis method, the actual approach volumes that correspond to a given equivalent volume adjusted for left turns on a one-lane approach decrease with an increase in the percentage of left turns. Therefore, since the actual approach volumes were used to compute the fuel consumption, the fuel savings will tend to also decrease with increased left-turn percentages. Thus, the fuel savings reflect the combined effects of a higher left-turn percentage increasing the volume-to-capacity ratio and decreasing the actual volume used in the calculation of fuel consumption.

To confirm this explanation of the dual effects of the left-turn percentage, a regression analysis of percent fuel savings versus the initial conditions was conducted. Because percent fuel savings is primarily sensitive to changes in the volume-to-capacity ratio, it was expected that the results of this regression analysis would only show a positive influence of left-turn percentage. The following statistically significant (α = 0.10) relations were the result of this regression analysis. For left-turn bays without a protected phase,

$$\%f = -37.8 + (15.3V/10^3) + (14.2V_0/10^3) + 31.7 \log_{10} PLT \quad (5)$$

For left-turn bays with a protected phase,

$$\%f = -49.9 + (22.9V/10^3) + (9.02V_0/10^3) + 34.0 \log_{10} PLT \quad (6)$$

where %f is the percent fuel savings for traffic on a street on which left-turn bays were added. As expected, no negative influence of left-turn percentage is found in Equations 5 and 6.

Over the entire range of conditions examined in this study, the fuel savings that result from the addition of left-turn bays without a protected left-turn phase were always greater than those that result from the addition of left-turn bays with a protected left-turn phase under the same set of conditions. There were two primary reasons for this occurrence. First, the addition of a left-turn phase does reduce the capacity of the intersection,

Figure 1. Fuel savings for left-turn bays without protected left-turn phase.

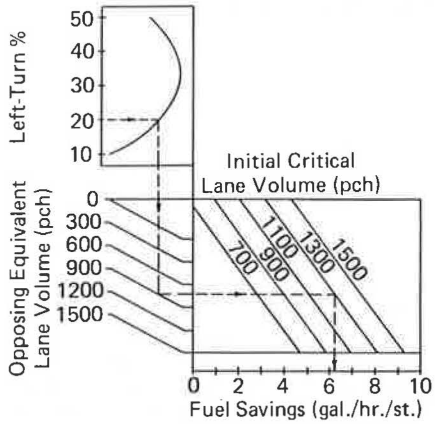
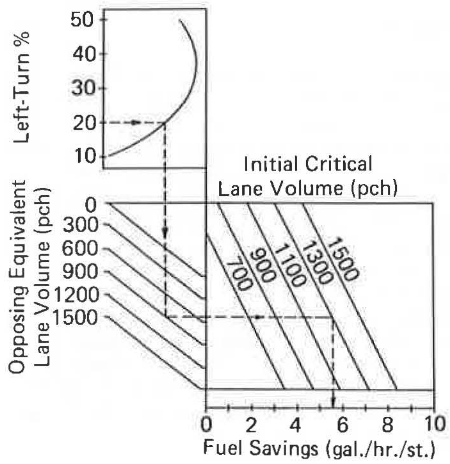


Figure 2. Fuel savings for left-turn bays with protected left-turn phase.



which tends to reduce the fuel savings to be realized. Second, in this study, the left-turn percentages on the opposing and critical-lane-volume approaches were equal for all conditions. Thus, the potential advantage of a left-turn phase tended to be negated as the left-turn percentage was increased because with an increased left-turn percentage, the actual approach volumes, which yielded the assumed equivalent lane volumes, tended to be decreased. As these volumes were decreased, so were the passenger car equivalents used to calculate the equivalent lane volumes of the unprotected left-turn movements. This effect tended to make the unprotected phasing more favorable. If instead an approach with a high left-turn percentage would have been opposed by an approach with a high volume and a low left-turn percentage, this effect would not have been present and the protected phasing would probably have been more favorable. Therefore, the limitation of this study to cases of equal left-turn percentage should be recognized when using its results.

To facilitate the calculation of potential fuel savings that result from the addition of left-turn bays at signalized intersections, Equations 3 and 4 are presented as nomographs in Figures 1 and 2, respectively. These nomographs represent the fuel savings on the approaches to which the left-turn bays were added over the range of conditions investigated in this study. Fuel savings realized on the cross street due to the improved level of service are not included in these figures. Also, the fuel savings in these figures are subject to the assumptions of this analysis.

CONCLUSIONS

The findings of this study indicate that the addition of left-turn bays at isolated, two-phase, signalized intersections of two-way, two-lane streets provide fuel savings for traffic on the street on which the left-turn bays are added. The fuel savings were found to range from 0.3 to 10 gal/h as a curvilinear function of the initial critical lane volume, the volume opposing the initial critical lane volume, and the percentage of left turns. In addition, on streets with equal left-turn percentages on its approaches, the addition of a left-turn bay without a protected left-turn phase always provided greater fuel savings than did the addition of a left-turn bay with a protected left-turn phase because of the combined effects of the lower intersection capacity caused by the addition of a phase and the reduction in the opposing through volume that accompanied an increase in the equal left-turn percentages. Unfortunately, only cases with equal left-turn percentages were considered in this study. Therefore, the fuel-savings breakeven point between protected and unprotected left-turn bays was not determined.

Although the procedures of this study are generally applicable, the application of the regression equations and nomographs developed in this study should be limited to the range of conditions examined in this study. Likewise, their application should recognize the following assumptions on which their development was based: (a) equal left-turn percentages, (b) 12-ft lanes, (c) peak-hour factor of 1.00, (d) no trucks or buses, (e) no right turns, and (f) 30-mph approach speeds. According to the critical-movement-analysis method (3) and the procedure of this study, fuel savings higher than those found in this study would be expected in cases with higher approach speeds and/or trucks and buses because of the higher fuel-consumption rates associated with these conditions. However, lower fuel savings would be expected on approaches with lower peak-hour factors, narrower lanes, and right turns because, for a given level of service, the higher passenger car equivalents associated with these conditions result in lower actual traffic volumes that consume fuel.

REFERENCES

1. D.B. Frambro, C.J. Messer, and D.A. Andersen. Estimation of Unprotected Left-Turn Capacity at Signalized Intersections. TRB, Transportation Research Record 644, 1977, pp. 113-119.
2. P.J. Tarnoff and P.S. Parsonson. Selecting Traffic Signal Control at Individual Intersections. NCHRP, Rept. 233, June 1981, 133 pp.
3. Interim Materials on Highway Capacity. TRB, Transportation Research Circular 212, Jan. 1980, 276 pp.
4. W.R. Reilly and others. A Technique for Measurement of Delay at Intersections. JHK & Associates, San Francisco, Sept. 1976.
5. C.W. Dale. Procedure for Estimating Highway User Costs, Fuel Consumption, and Air Pollution. FHWA, March 1980.

Effect of Left-Turn Bays on Fuel Consumption on Uncontrolled Approaches to Stop-Sign-Controlled Intersections

DENNIS V. DVORAK AND PATRICK T. McCOY

Associated with the reductions in delay and stops that result from the provision of left-turn bays is a reduction in fuel consumption. Less delay and fewer stops mean less fuel consumed by vehicle idling and speed-change cycles. The objective of this research was to estimate the effect of the provision of left-turn bays on fuel consumption on uncontrolled approaches to stop-sign-controlled intersections over a range of volumes, approach speeds, and truck percentages on two-way, two-lane roadways. A series of paired computer simulation runs was conducted by using the NETSIM traffic simulation model to evaluate the fuel consumption of the traffic on the uncontrolled approaches with and without left-turn bays. A pairwise comparison of the NETSIM fuel-consumption output from these runs provided the measure of fuel savings due to left-turn bays. Over the range of conditions studied, the fuel savings varied from zero to more than 20 gal/h for traffic on the approach. The amount of the fuel savings was a complex function of approach volume, opposing volume, left-turn percentage, free-flow approach speed, and truck percentage. Graphs and adjustment factors were developed to describe this relation and provide a means of estimating the fuel savings associated with left-turn bays.

Left-turn bays are provided on uncontrolled approaches to stop-sign-controlled intersections to improve the safety and efficiency of traffic operations on these approaches. The primary function of these left-turn bays is to remove the deceleration and storage of left-turning vehicles from the through lanes and thereby enable through and right-turning traffic to move by them without conflict and delay. Thus, the benefits derived from the provision of these left-turn bays are reductions in accidents, delay, and stops. Previous research (1-3) has found the amounts of these reductions to be functions of the approach, opposing, and left-turn volumes.

Associated with the reductions in delay and stops that result from the provision of left-turn bays is a reduction in fuel consumption. Less delay and fewer stops mean less fuel consumed by vehicle idling and speed-change cycles. The objective of this study was to estimate the effect of the provision of left-turn bays on fuel consumption on the uncontrolled intersections. This effect was evaluated over a range of volumes, approach speeds, and truck percentages. However, the scope of the study was limited to approaches on two-way, two-lane roadways. This paper presents the procedure and findings of this study.

PROCEDURE

A series of computer simulation runs was conducted by using the NETSIM traffic simulation model (4) to simulate traffic operations at a four-legged intersection of two, two-way, two-lane roadways. The intersection was controlled by stop signs on the approaches of the minor roadway. One set of simulation runs was made with left-turn bays on the uncontrolled approaches of the major roadway, and a second set of runs was made without left-turn bays on these approaches. Both sets of runs were made over the same range of volumes, approach speeds, and truck percentages. The effect of the left-turn bays on fuel consumption was then determined by a pairwise comparison of the NETSIM fuel-consumption output from the two sets of runs for identical combinations of volumes, approach speeds, and truck percentages. Thus, for a given combination of these conditions, the effect of a left-turn bay on an approach was computed as the difference between the two runs in the amount of fuel consumed by traffic in the direction of the approach. The results of these computations were then analyzed to examine the relation between the effect of left-turn bays on fuel consumption and traffic conditions.

Intersection Description

The basic intersection used in this study was a four-legged intersection of two, two-way, two-lane roadways with stop sign control on the minor roadway. One configuration of this simulated intersection had left-turn bays on the approaches of the major roadway, and the other simulated configuration had no left-turn bays on these approaches. These two configurations are shown in Figure 1.

Also shown in Figure 1 is the link-node representation of the intersection that was input to the NETSIM model. Links 1-5 and 3-5, which represented the approaches on the major roadway, were coded with and without the left-turn bays. The approach volumes on these links were generated according to the shifted exponential headway distribution contained in the NETSIM model.

Simulation Runs

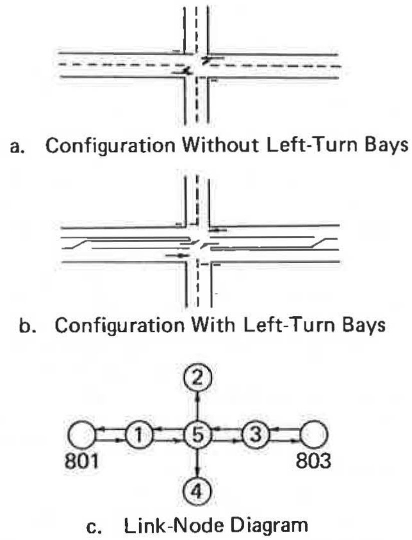
Simulation runs with and without left-turn bays on the major roadway approaches were made over a range of volumes for three approach speeds and three truck percentages. The free-flow speeds on the major roadway approaches were 30, 45, and 50 mph. The truck percentages used were 0, 10, and 20 percent.

For each of the nine combinations of approach speed and truck percentage, volumes were varied over ranges similar to those used by Lee (5) to develop design guidelines for left-turn lanes at priority intersections. The volumes on the study approach were varied over a range of 100-1500 vehicles/h. The volumes used on the opposing approach were equal to, one-half of, and twice the volume on the study approach. The percentage of left turns was varied from 1 to 50 percent of the vehicles entering on the approach. The percentage of left turns on the opposing approach was always equal to that on the study approach. The right-turn percentage was zero in every case. It was assumed that the provision of left-turn bays on the major roadway would have a negligible effect on the fuel consumed by traffic on the stop-sign-controlled approaches of the minor roadway. Therefore, the volumes on the minor roadway were always set equal to zero in order to minimize the computer time required to conduct the simulation runs.

A 30-min period of time was simulated during each run. Prior to the 30-min period, about 10 min of simulation time was required to achieve steady-state conditions.

The runs were initially chosen by using a modi-

Figure 1. Intersection studied.



fied response surface design (6,7). This type of experimental design chooses five points for each variable. The points selected are close to the end points, the midpoint, and the 20th and 80th percentiles. This was done for the variables of approach volume and left-turn percentage.

Then the points chosen by the response surface design were run for all combinations of approach speed, truck percentage, and the approach-opposing volume relation. These points were simulated both with and without a left-turn bay on the major approaches. After analysis of these runs, it was determined that there were some major gaps in the data. Therefore, some more combinations were run to fill these gaps. A total of 723 combinations were run. A summary of the runs made is given in the table below [note: for each approach volume, the left-turn percentage combination shown was run for each of the following 27 combinations of opposing volume, free-flow approach speed, and truck percentage: (a) opposing volume equal to approach volume, 0.5 x approach volume, and 2 x approach volume; (b) free-flow approach speed at 30, 45, and 50 mph; and (c) truck percentage at 0, 10, and 20 percent]:

Approach Volume (vehicles/h)	Left-Turn Percentage
100	1,10,20,30,40,50
300	1,10,20,30,40,50
500	1,10,20,30,40,50
700	1,10,20,30,40,50
850	1,5,10,15,20,30,40,50
1000	1,3,5,8,10,15,20,30
1200	1,3,5,8,10,15,20
1500	1,2,5,8,10,15

Data Analysis

The output of all the NETSIM runs was first examined to determine if congestion had occurred on the approach. When congestion had occurred, infinite queues began to form on the approach, which made it impractical to compute the effect on fuel consumption of the left-turn bay. Therefore, the output of runs during which congestion had occurred was eliminated from the analysis.

By using the NETSIM outputs of gallons of fuel consumed and number of entering vehicles for each combination of volumes, approach speed, and truck percentage, the number of gallons of fuel consumed per vehicle by traffic in the direction of the approach (i.e., the fuel consumed on links 1-5 and 5-3 in Figure 1) was computed for both with and without a left-turn bay. The difference between these two values was then multiplied by the hourly approach volume to obtain the gallons of fuel saved per hour on the approach by the provision of a left-turn bay. The fuel savings computed for all combinations of volumes, approach speed, and truck percentage were then analyzed to determine relations between fuel savings and these conditions.

FINDINGS

Initial review of the output of the NETSIM runs determined that congestion occurred on runs with approach and opposing volumes equal to or greater than 1000 vehicles/h. Congestion was also found on runs with approach volumes greater than 500 vehicles/h and opposing volumes greater than 1000 vehicles/h. After these runs were eliminated from the analysis, 473 combinations of volumes, approach speed, and truck percentage remained.

A regression analysis of the fuel savings due to left-turn bays for the remaining 473 combinations was conducted by using linear and polynomial terms. But the results of the regression analysis were not able to provide a relation between fuel savings and traffic conditions that accounted for a satisfactory amount of the variation in fuel savings. However, a comparison of mean fuel savings, conducted at a five percent level of significance, determined that the means for approach speed and truck percentage were significant.

Therefore, since the regression analysis did not provide an acceptable description of the relation between fuel savings and traffic conditions, it was decided to show this relation graphically. Three graphs, one for each approach-opposing volume relation, that show the relation among fuel savings, approach volume, and left-turn percentage were prepared for the combinations of a 45-mph approach speed and 20 percent trucks, which was the approach speed, truck percentage combination that provided the greatest fuel savings. These graphs are shown in Figures 2, 3, and 4.

Based on the mean fuel savings of each of the nine combinations of approach speed and truck percentage, the set of adjustment factors shown in the table below was derived:

Truck Percentage	Adjustment Factors by Approach Speed 30 mph	45 mph	50 mph
0	0.1	0.5	0.6
10	0.2	0.7	0.7
20	0.7	1.0	0.9

The adjustment factor for each combination represents the average portion of the fuel savings of the 45-mph, 20 percent truck combination that is realized with the combination to which the particular factor applies. Thus, to estimate the fuel savings that would result from the provision of a left-turn bay on an approach with an approach speed, truck percentage combination other than 45 mph and 20 percent, the fuel savings found from the appropriate graph (Figure 2, 3, or 4) are multiplied by the appropriate adjustment factor from the above table. For example, if the fuel savings found from the appropriate graph were 10 gal/h and the approach had a 50-mph speed and 10 percent trucks, the fuel savings that would result from providing a left-turn bay on the approach would be 6.0 (10 x 0.6) gal/h for traffic in the direction of the approach.

The adjustment factors in the above table show that the fuel savings at all approach speeds in-

Figure 2. Fuel savings: opposing volume equal to approach volume.

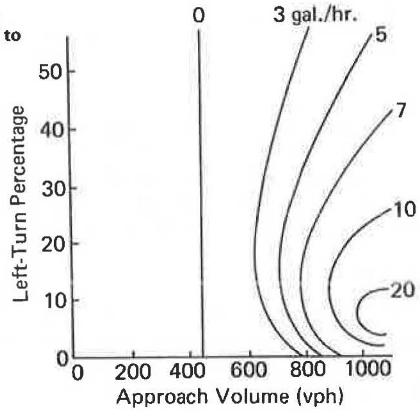


Figure 3. Fuel savings: opposing volume equal to one-half approach volume.

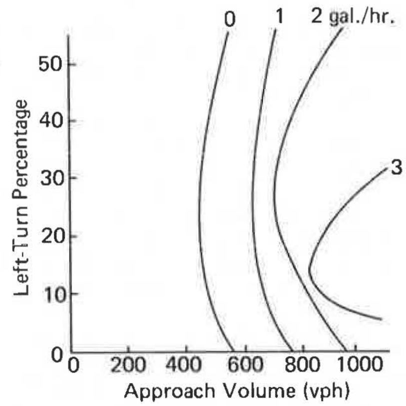
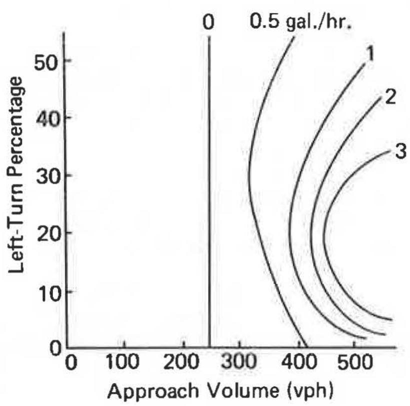


Figure 4. Fuel savings: opposing volume equal to two times approach volume.



crease with an increase in truck percentage, especially as the truck percentage increases from 10 to 20 percent. This increase resulted primarily from the more frequent occurrence of queues created by trucks during simulation runs without left-turn bays. These factors also indicate that the greatest fuel savings are realized for approaches with speeds of 45 mph and 20 percent trucks. The fuel savings for approaches with speeds of 50 mph are about the same as those for approaches with speeds of 45 mph. The fuel savings for approaches with 30-mph speeds average about one-half those on 45-mph approaches as a result of considerably lower fuel-consumption rates associated with speed changes at 30 mph.

Comparison of the graphs shown in Figures 2, 3, and 4 indicates that the pattern of fuel savings is similar on all three graphs, with the greatest fuel savings realized on the graph for opposing volumes equal to approach volumes (Figure 2). On all three graphs, fuel savings increase as approach volumes increase. However, in the case of equal opposing and approach volumes (Figure 2), no fuel savings are realized for approach volumes of less than 450 vehicles/h. In the other two cases, zero fuel saving is realized at approach volumes less than 450 vehicles/h when the opposing volume is one-half of the approach volume (Figure 3) and less than 250 vehicles/h when the opposing volume is twice the approach volume (Figure 4).

Also, on all three graphs, fuel savings increase with left-turn percentage up to a point and then decrease with further increases in the left-turn percentage. And, on all three graphs, the left-turn percentages for maximum fuel savings decrease as approach volumes increase. But, for a given approach volume, the left-turn percentage for maximum fuel savings in the case of equal opposing and approach volumes (Figure 2) is always lower than those of the other two cases (Figures 3 and 4). This was due to the fact that, in this study, the opposing left-turn percentage was equal to the approach left-turn percentage in all cases. Therefore, as the left-turn percentage increased, the opposing through volume actually decreased. Also, the right-turn percentage was zero in every case.

As an example to illustrate the application of the results of this study, consider the addition of a left-turn bay on an uncontrolled approach to a stop-sign-controlled intersection. The volume on the approach is 800 vehicles/h with 10 percent left turns and 10 percent trucks. The approach speed is 30 mph, and the opposing volume is equal to the approach volume. A fuel savings of 7 gal/h is found in Figure 2 for equal approach and opposing volumes of 800 vehicles/h with 10 percent left turns. However, this savings is for a 45-mph approach speed and 20 percent trucks. Therefore, an adjustment factor of 0.2 is found in the adjustment factor table presented earlier for a 30-mph approach speed and 10 percent trucks. The fuel saving of 7 gal/h found in Figure 2 is multiplied by this adjustment factor of 0.2 to obtain the fuel savings of 1.4 gal/h. Thus, the fuel savings that would result from the addition of the left-turn bay would be estimated to be about 1 gal/h.

CONCLUSIONS

In this study, the fuel savings that result from the provision of left-turn bays on the uncontrolled approaches to two-way stop-sign-controlled intersections on two-way, two-lane roadways ranged from zero to more than 20 gal/h per approach. The amount of the fuel savings was dependent on a complex relation among the following approach conditions: (a) approach volume, (b) opposing volume, (c) left-turn percentage, (d) free-flow approach speed, and (e) truck percentage. The greatest fuel savings were found on approaches with equal approach and opposing volumes with more than 950 vehicles/h, 5-10 percent left turns, 45-mph free-flow approach speeds, and 20 percent trucks. However, zero fuel savings were found on these approaches when the approach volume was less than 450 vehicles/h.

The lowest fuel savings were found on approaches with 30-mph free-flow approach speeds and no trucks. The fuel savings on these approaches was one-tenth of that found on approaches with 45-mph free-flow approach speeds and 20 percent trucks. Overall, the fuel savings on approaches with 30-mph free-flow approach speeds averaged about one-half those on approaches with 45-mph free-flow approach speeds. Also, the fuel savings on approaches with 10 percent trucks or less were about 50 percent of those found on approaches with 20 percent trucks.

Considerably lower fuel savings (usually less than 3 gal/h) were found on approaches where the opposing volumes were not equal to the approach volumes. However, under all conditions, the fuel savings on an approach increased with an increase in left-turn percentage up to a point beyond which further increases in left-turn percentage resulted in lower fuel savings. The left-turn percentage at which this point occurred decreased as approach volume increased.

The findings of this study can be used to estimate the fuel savings that would result from the provision of left-turn bays on approaches similar to those considered in the study. However, in using these findings, it should be noted that they apply to uncongested flow conditions with equal opposing and approach left-turn percentages and zero right-turn percentages. In addition, the fuel-consumption rates used in this study were those embedded in the NETSIM model (4), which represent weighted composite 1971 vehicles.

REFERENCES

1. R.B. Shaw and H.L. Michael. Evaluation of Delays and Accidents at Intersections to Warrant Construction of a Median Lane. HRB, Highway Research Record 257, 1968, pp. 17-33.
2. S.L. Ring and R.L. Carstens. Guidelines for the Inclusion of Left-Turn Lanes at Rural Highway Intersections. HRB, Highway Research Record 371, 1971, pp. 64-79.
3. J. Lee and T. Mulinazzi. Design of Left-Turn Lanes for Priority Intersections. TRB, Transportation Research Record 575, 1980, pp. 33-40.
4. E. Lieberman and others. NETSIM Model: Volume 4--User's Guide. FHWA, Rept. FHWA-RD-77-44, Oct. 1977.
5. J. Lee. Left-Turn Lane Design Guidelines for Priority Intersections in the State of Kansas. Center for Research, Inc., Univ. of Kansas, Lawrence, Rept. FHWA-KS-78-1, Feb. 1978.
6. W.G. Cochran and G.M. Cox. Experimental Designs. Wiley, New York, 1957.
7. R.H. Myers. Response Surface Methodology. Allyn and Bacon, Inc., Boston, 1971.