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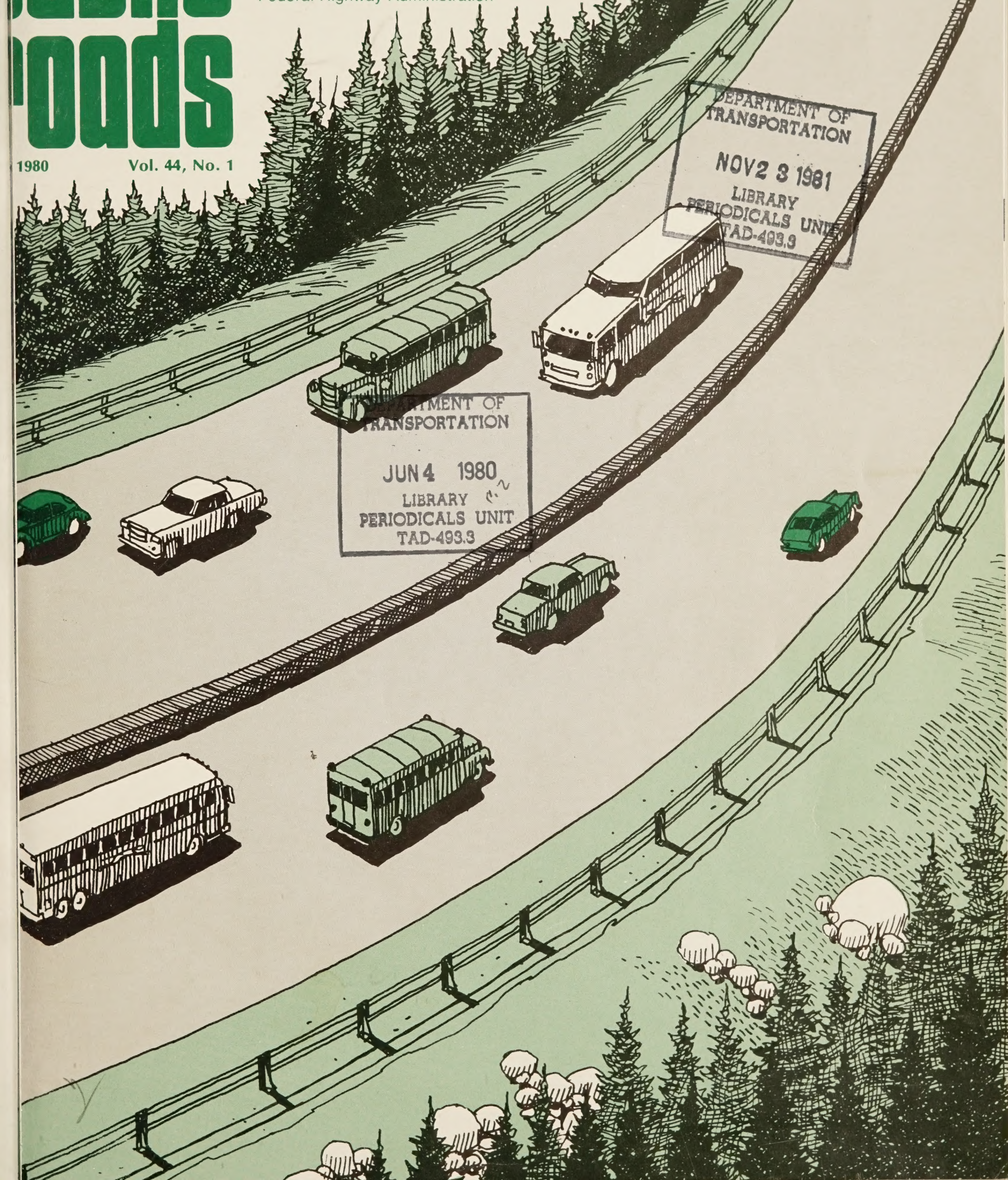

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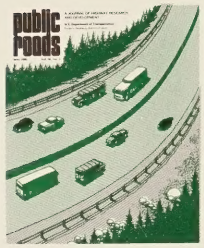

COVER:

Roadside safety devices in relation to the downsizing of vehicles.

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Articles

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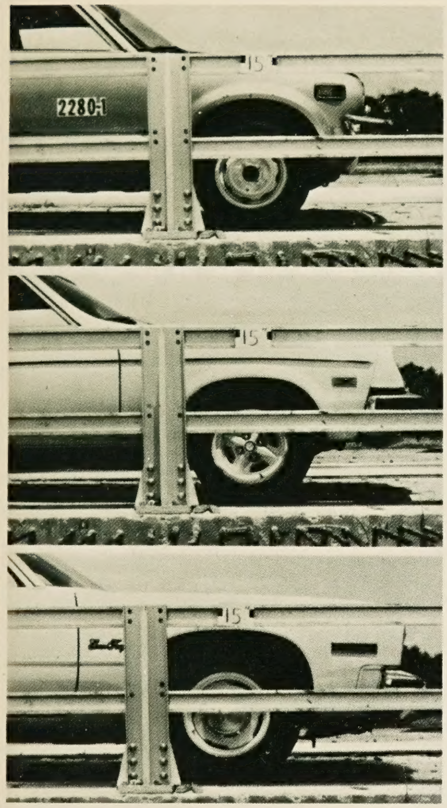


Vehicle Downsizing and Roadside Safety Hardware

by
John G. Viner[1]

Introduction

Downsizing of domestic passenger vehicles is expected to be a major element in the strategy to meet the fuel economy mandates of the Corporate Average Fuel Economy (CAFE) requirements of the 1975 Energy Policy and Conservation Act. The expected increase in mini-sized vehicles makes it necessary to reexamine existing roadside safety hardware design and performance criteria, which generally are based on vehicles weighing 1.02 to 2.04 Mg (2,250 to 4,500 lb). An analysis of future vehicle downsizing needs suggests that vehicles as light as 0.77 to 0.81 Mg (1,700 to 1,800 lb) must be considered.

As vehicle weight decreases, the severity of crashes involving vehicles and pole-like structures increases. High-speed offcenter impacts of small vehicles with such structures may result in violent multiple rollovers. Thus it is important to reexamine the safety performance criteria for breakaway signs and luminaire supports, base bending signs, and other poles and supports.

Vehicle geometry, inertial characteristics, and crash properties of mini-sized vehicles may present special problems in the design of traffic railings. Small vehicles have snagged support posts and rolled over in traffic railing tests. Existing impact attenuators have been designed to accommodate vehicles weighing 1.02 to 2.04 Mg (2,250 to 4,500 lb). The performance criteria for these devices should be reexamined to consider the special problems of mini-sized vehicles.

This article reviews the evolution of vehicle size considerations in roadside safety hardware research. Data are discussed on the selection of initial mini-sized vehicles for Federal Highway Administration (FHWA) research. Results from ongoing research to identify safety problems in collisions of smaller vehicles and appropriate countermeasures are reviewed.

Ran-Off-the-Road Accidents

Ran-off-the-road accidents are a major highway safety problem and the development and use of roadside safety hardware has been an important strategy in reducing the severity of such accidents. The importance of this problem is illustrated in figure 1, which shows the distribution of fatal accidents in 1977 classified by first harmful

[1] This article is a condensation of an article in the *Proceedings of the Twenty-Third Conference, American Association for Automotive Medicine*, Louisville, Ky., Oct. 3–6, 1979.

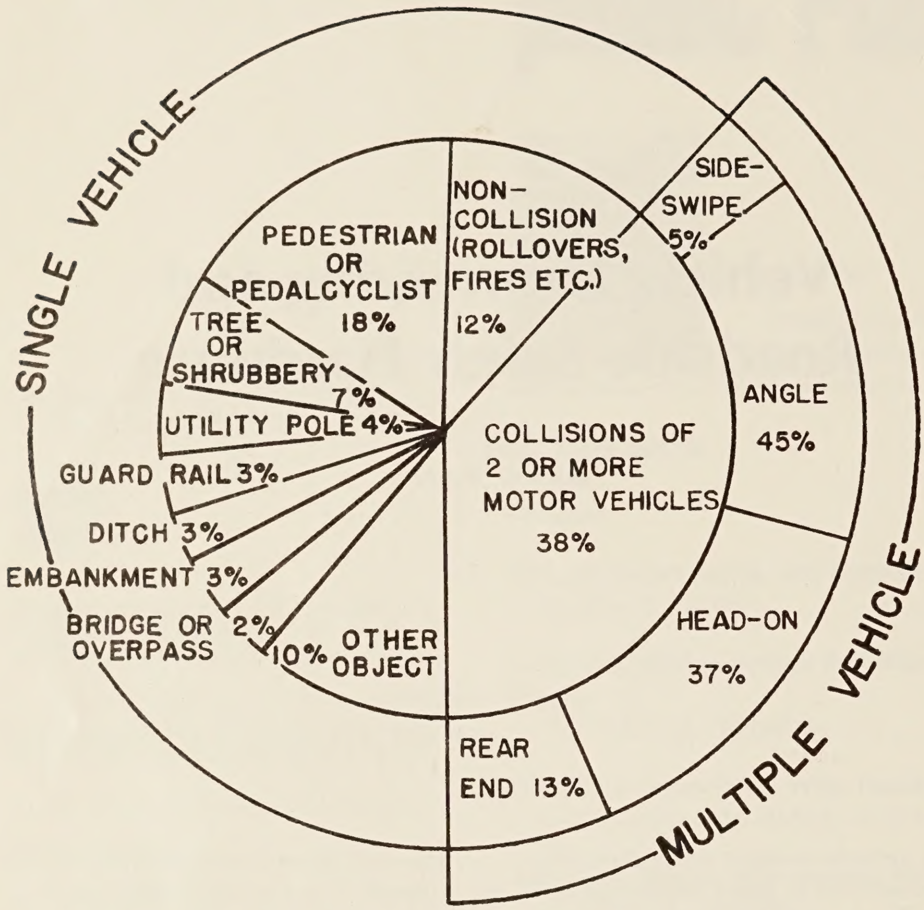


Figure 1.—Distribution of fatal accidents by first harmful event.

event. (*1*)[2] Subtracting the pedestrian or pedalcyclist accidents from the single vehicle figures, it can be seen that in 44 percent of the 1977 fatal accidents a roadside feature was listed as the first harmful event. Because of the high speed involved in fatal accidents, one or more roadside features are involved in a significant fraction of the fatal accidents classified as multiple vehicle.

[2] Italic numbers in parentheses identify references on page 8.

Impact Tests

Developmental work in the 1960's was essentially concerned with only passenger vehicles 1.59 to 1.81 Mg (3,500 to 4,000 lb). Often, only one impact condition was examined in full scale proof testing. Guardrail and median barrier crash tests were conducted with 1.81 Mg (4,000 lb) vehicles traveling 97 km/h (60 mph) with an impact angle of 25°, in accordance with a 1962 recommendation of the Highway Research Board Committee on Guardrails and Guideposts. (*2*) Only one vehicle—a mid-1950's standard-sized sedan weighing about 1.59 Mg (3,500 lb)—was used in the

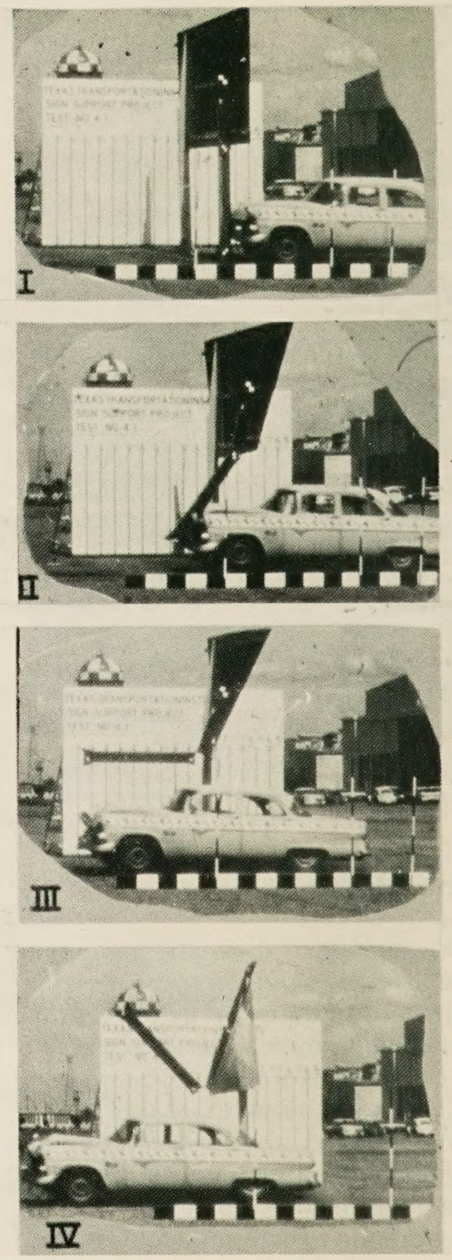


Figure 2.—Breakaway sign test—a 1.63 Mg (3,600 lb) vehicle traveling 64 km/h (40 mph).

mid-1960's to research the development of the breakaway sign. (*3-5*) Figure 2 shows a 64 km/h (40 mph) test with a breakaway sign.

Actual vehicle crash tests on breakaway luminaire supports conducted in the late 1960's used vehicles weighing between 1.59 and 1.81 Mg (3,500 and 4,000 lb). Like the breakaway sign tests, all of these tests were frontal at the vehicle's centerline. In addition, the breakaway luminaire support crash tests were conducted at a single speed—64 km/h (40 mph). (*6*) Figures 3 and 4 show the results of such a test with a 12 m (40 ft) high luminaire support. In this test the change in vehicle velocity was 9.7 km/h (6.0 mph). The change in vehicle momentum was 4.40 kN-sec (980 lb-sec) and the maximum vehicle deformation was 430 mm (17 in). An analysis of this research predicted that severity of impacts with luminaire supports increased as vehicle weight and impact speed decreased.

Figure 4.—Vehicle after impact test with luminaire support shown in figure 3.

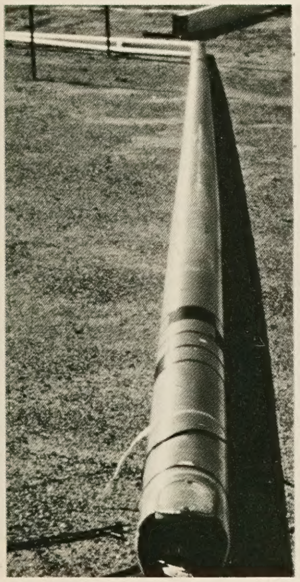


Figure 3.—A luminaire support used to test the impact with a 1.63 Mg (3,600 lb) vehicle traveling 64 km/h (40 mph).

In 1968 acceptance criteria were written for breakaway luminaire supports used on Federal-aid highways. The change in vehicle momentum was limited to 4.90 kN-sec (1,100 lb-sec) in such crash tests. (*7*) This limit corresponds to a 9.7 km/h (6.0 mph) change in velocity for a 1.81 Mg (4,000 lb) vehicle. Information available at the time suggested that serious head and chest injuries could occur to unbelted vehicle occupants when the vehicle underwent a sudden change in velocity—18 to 21 km/h (11 to 13 mph). (*8–10*) Thus, for the unbelted occupants this criterion was thought to provide a margin between expected vehicle behavior and serious head injury.

At the beginning of the research program on the development of impact attenuators, it was recognized that the effectiveness of impact attenuators greatly depended on the weight of the impacting vehicle. Impact attenuators were evaluated for vehicles weighing 0.91 to 2.04 Mg (2,000 to 4,500 lb) and traveling 97 km/h (60 mph) when the impacts occurred. (*11*) Figure 5 shows a vehicle weight distribution observed from traffic on Michigan freeways in 1968. (*12, 13*) More than 80 percent of the vehicles used in this initial impact attenuator developmental program weighed between 0.91 and 2.04 Mg (2,000 to 4,500 lb).

Figure 6 shows a 0.88 Mg (1,940 lb) vehicle traveling at 95 km/h (59 mph) in an impact attenuator test. (*14*) Figure 7 shows the vehicle after the test. The stopping distance in this test was 5.8 m (19 ft), which corresponds to an average deceleration of 59.8 m/s^2 (6.1 g). The maximum permitted deceleration in such a test is 117.6 m/s^2 (12 g). However, several attenuators produce decelerations of 58.8 to 78.4 m/s^2 (6.0 to 8.0 g) or even less under conditions specified in the test criteria, and the design and use of such attenuators have been encouraged. (*15*)

In 1974, procedures recommended for vehicle crash testing of traffic railings, impact attenuators, and breakaway or yielding supports were published to promote uniform testing practices for comparing the merits of two or more promising roadside safety devices. (*16*) Tests with 1.02 Mg (2,250 lb) and 2.04 Mg (4,500 lb) vehicles were specified for each device type, and a tolerance of ± 0.09 Mg (± 200 lb) was permitted

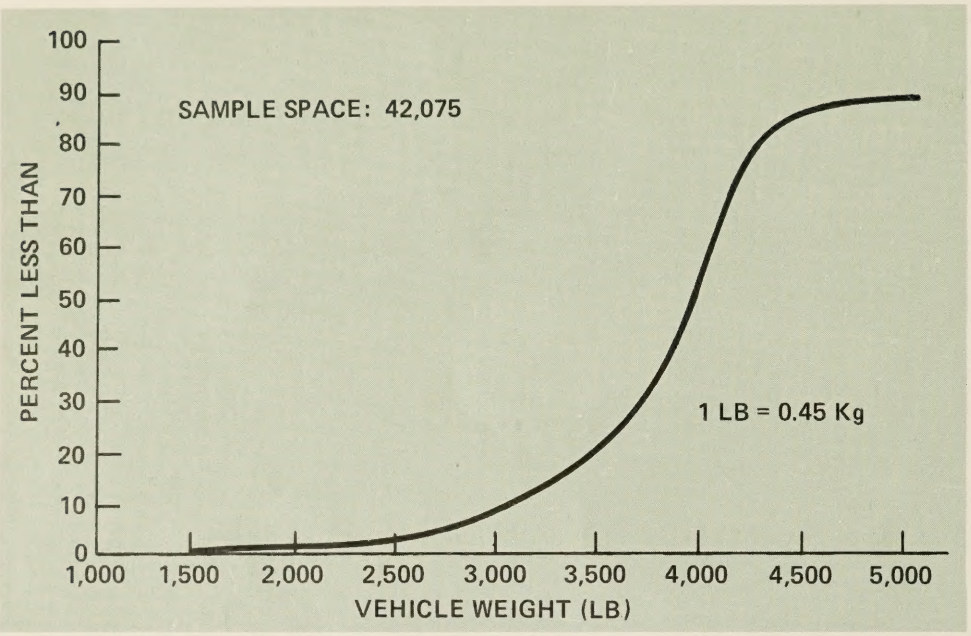
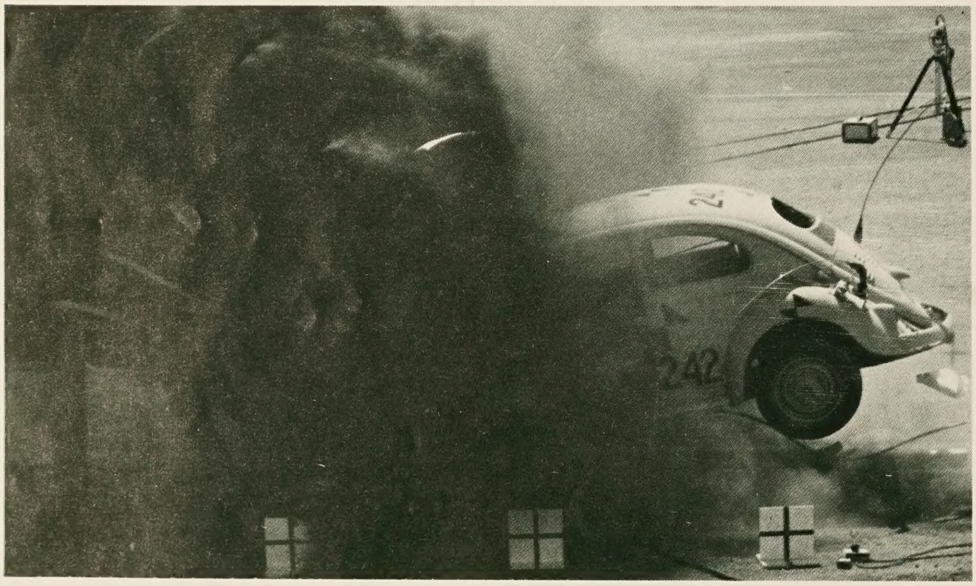


Figure 5.—Vehicle weight distribution from 1968 Michigan freeway data.

Figure 7.—Vehicle after test shown in figure 6.

Figure 6.—An impact attenuator test—an 0.88 Mg (1,940 lb) vehicle traveling 95 km/h (59 mph).



in the weight of the test vehicle. Figure 8 shows that approximately 85 percent of the passenger vehicles registered in the United States in 1974 weighed between 1.02 and 2.04 Mg (2,250 and 4,500 lb). (*17*) Domestic automobiles weighing about 1.02 Mg (2,250 lb) were a significant portion of new car sales in 1974.

Figure 9 shows a 1.02 Mg (2,250 lb) vehicle next to a shaped concrete median barrier. Figure 10 shows the difference between two barrier shapes—New Jersey (NJ) and General Motors (GM) profiles—widely used in the mid-1970's. (*17*) The results of tests with vehicles weighing 1.02 Mg (2,250 lb), traveling 97 km/h (60 mph) with an impact angle of 15° on the NJ and GM barrier shapes showed rollover with the GM shape but no rollover with the NJ shape. (*16*) These tests provide insight on collision severity under conditions more common than the previously used single test using a vehicle weighing 1.81 Mg (4,000 lb), traveling 97 km/h (60 mph), with an impact angle of 25°.

The frequency of rollovers of compact vehicles with the GM and NJ barrier shapes was found by real world accident analyses, vehicle crash testing, and computer simulations. Unlike compact vehicles, full-sized 2.04 Mg (4,500 lb) vehicles did not have a higher frequency of rollovers upon impact with the NJ barrier shape. Minor changes in barrier design produced important changes in the severity of impact of compact vehicles.

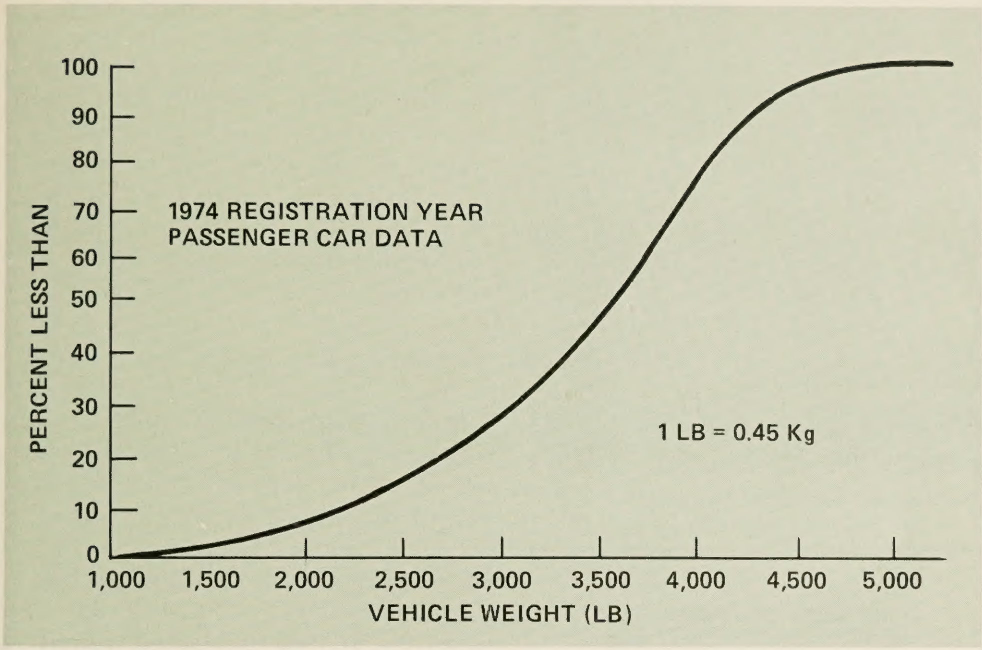


Figure 8.—1974 passenger vehicle weights.

Figure 11.—Intercity bus and guardrail accident.

Figure 12.—Straight truck and bridge rail accident.

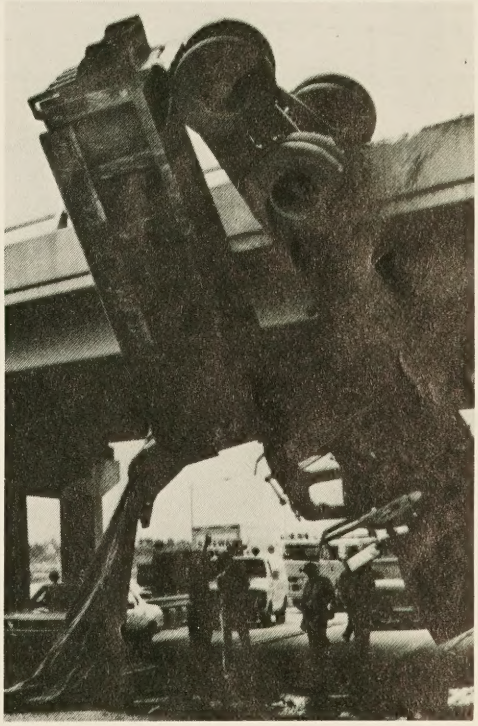
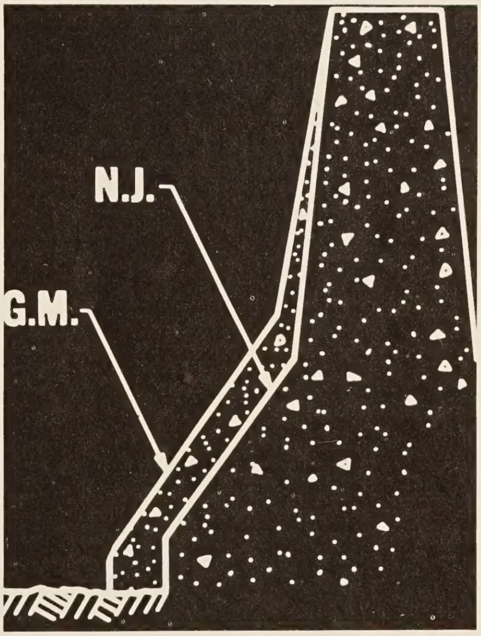


Figure 9.—A 1.02 Mg (2,250 lb) vehicle and a shaped concrete median barrier.

Figure 10.—NJ and GM shaped barriers.



The British reported impact tests of a mini vehicle weighing 0.76 Mg (1,670 lb) with NJ shaped concrete barriers. (*18*) Violent multiple rollover resulted in a 111 km/h (69 mph), 20° test with this vehicle.

Figures 11 and 12 demonstrate the need to develop traffic railings to retain heavy vehicles. FHWA research shows that it is feasible to develop traffic railings that can retain heavy vehicles without increasing collision severity for passenger car impacts. (*19*) Because these railings may be significantly more expensive than conventional railings, appropriate placement criteria are necessary to make the best use of available funds.

Developmental work on railings for heavy vehicles has focused on high occupancy vehicles—school buses, intercity buses, and tractor-trailers—with impact conditions as severe as 89 to 97 km/h (55 to 60 mph) at 15°. Figure 13 shows such a bridge rail and figure 14 shows the test results of an 18.14 Mg (40,000 lb) intercity bus traveling 89 km/h (55 mph), with an impact angle of 15°. (*20*) The bus was driven from the site after this test.

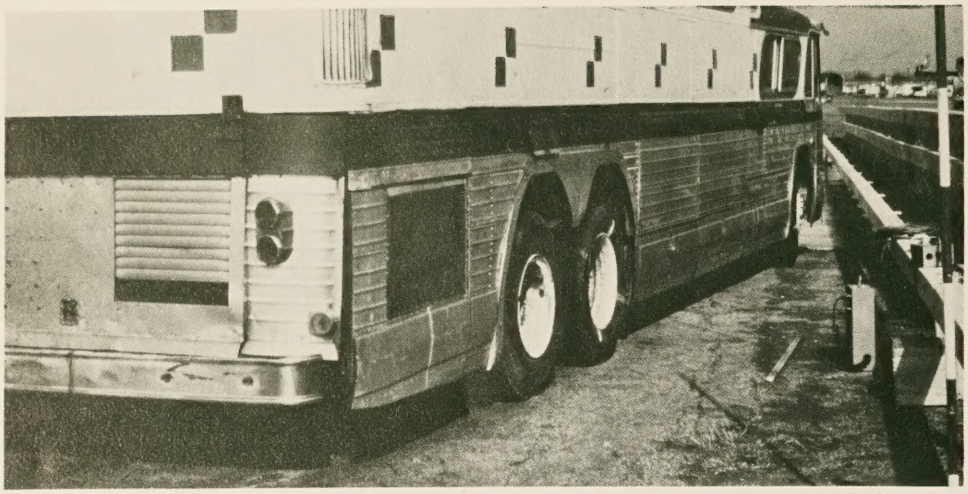


Figure 13.—18.14 Mg (40,000 lb) intercity bus impacting collapsing ring bridge rail.

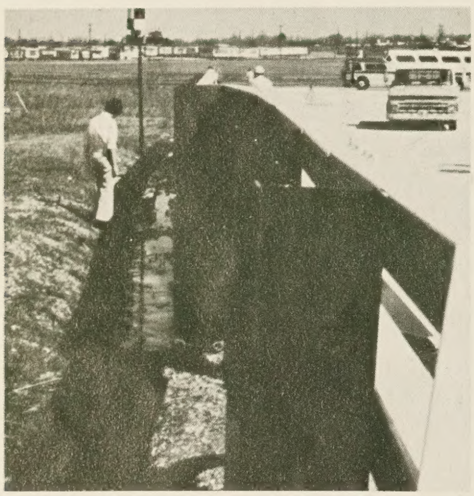


Figure 14.—Test results of 18.14 Mg (40,000 lb) bus traveling 89 km/h (55 mph), with an impact angle of 15°.

Figure 15.—Small sign support.

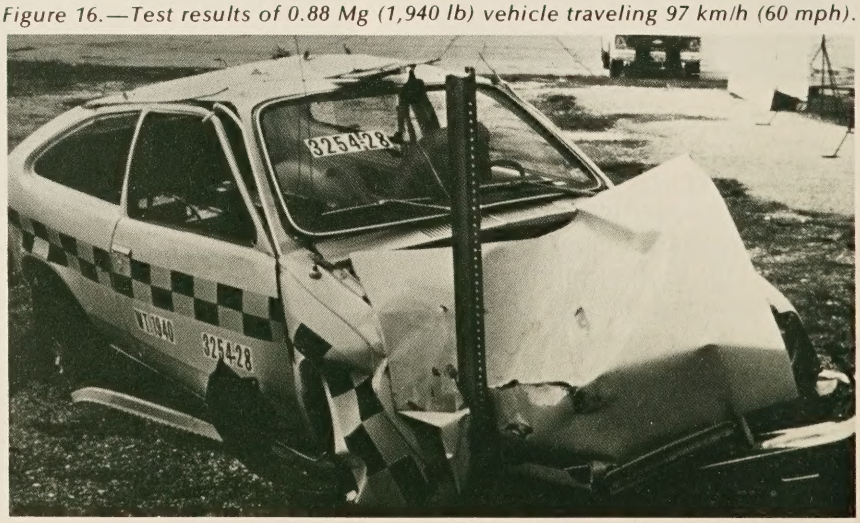
The updated recommended crash test procedures call for offcenter impacts, as stated above, with base bending signs to check for problems; however, the recommended vehicle weight of 1.02 to 2.04 Mg (2,250 to 4,500 lb) has not been changed. (*23*) An FHWA research study is planned to examine problems with smaller vehicles.

Meeting CAFE Requirements

The role of downsizing domestic passenger vehicles to meet the CAFE requirements has been predicted in a recent study. (*24*) Figure 17 shows the curb weight of domestic passenger vehicles manufactured in the United States in 1978 (actual weight) and 1986 (projected weight). A significant number of the domestic models expected to be produced in 1986 will weigh only 0.77 to 0.81 Mg (1,700 to 1,800 lb). A sales projection made by the Transportation Systems Center, Research and Special Programs Administration, U.S. Department of Transportation, predicts that 29 percent of 1985 passenger vehicle sales will be two

In a recent research program 1.02 Mg (2,250 lb) vehicles impacted small highway sign supports (fig. 15) head on at 97 km/h (60 mph) with the point of impact 381 mm (15 in) from the centerline. (*21*) In two of these tests vehicles overturned. This raised a concern about the increasing number of lighter vehicles with shorter wheel bases and narrower track widths that are on U.S. highways. In a limited followup effort, three tests were conducted with 0.88 Mg (1,940 lb) vehicles. A rollover occurred in one of these tests (fig. 16). (*22*)

Figure 16.—Test results of 0.88 Mg (1,940 lb) vehicle traveling 97 km/h (60 mph).



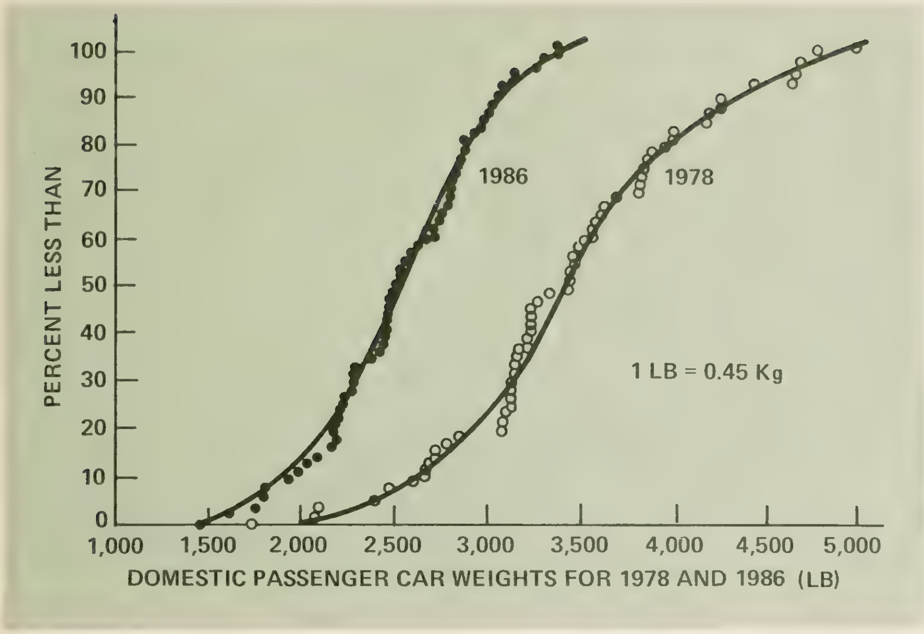


Figure 17.—Domestic passenger car weights for model years 1978 (actual) and 1986 (projected).

Figure 18.—Honda Civic and bridge rail impact test —0.81 Mg (1,800 lb) vehicle traveling 97 km/h (60 mph), with an impact angle of 15°.

seater (1 percent) or subcompact (28 percent) sizes.[3] Therefore, vehicles lighter than 1.02 Mg (2,250 lb) must be considered in the development and use of roadside safety hardware; FHWA research in this area will consider mini-sized vehicles. (*25*)

Selecting Mini-Sized Vehicles

Table 1 lists passenger vehicles weighing less than 1.02 Mg (2,250 lb) that were sold new in the United States in 1978. Values for a 1971 Vega Hatchback, a frequently used test vehicle in the 1.02 Mg (2,250 lb) class, are shown for comparison.

The Honda Civic has been selected as the initial mini-sized vehicle for FHWA research. This vehicle has front-wheel drive and a traverse mounted engine—a configuration expected to be used widely in future mini-sized vehicles.

Figures 18 and 19 show the test results of a Honda Civic traveling

Table 1.—1978 passenger vehicles in order of increasing base weight

Model and make	Wheel base	Tread		Individual model weight
		Front	Rear	
	(in)	*(in)*		*(lb)*
Honda Civic	86.6	51.2	50.4	1,664
Honda Civic CVCC	86.6	51.2	50.6	1,761
Renault LeCar	95.2	50.2	49.3	1,819
VW Rabbit	94.4	54.7	53.5	1,837
VW Scirocco	94.5	52.7	52.5	1,948
Fiat 128	96.5	51.3	51.8	1,950
Mazda GLC	91.1	51.0	51.6	1,965
Subaru DL	96.9	50.2	49.6	1,965
Chevette	94.3	51.2	51.2	1,991
Datsun F-10 Coupe	94.3	50.4	49.0	1,995
Subaru GF	96.9	50.2	49.6	2,015
Datsun B-210 Sedan	92.1	50.2	49.0	2,020
Honda Accord	93.7	55.1	54.7	2,030
Toyota Corolla	93.3	51.2	50.6	2,050
Audi Fox	96.7	52.7	52.5	2,070
VW Beetle	95.3	53.1	53.1	2,110
Omni	99.2	55.5	55.1	2,137
Horizon	99.2	55.5	55.1	2,137
Opel Coupe	94.3	51.4	51.4	2,150
VW Dasher	96.7	52.7	52.5	2,162
Arrow 160 Coupe	92.1	51.1	50.0	2,194
Opel Sedan	94.3	51.4	51.4	2,220
Datsun 510 Sedan	94.5	52.5	52.4	2,240
Vega Hatchback (1971)	97.0	55.0	54.2	2,250

1 in=25.4 mm 1 lb=0.45 kg

[3] K. H. Schaffer and J. S. Yarmus, "Projection of U.S. Motor Vehicle Sales," Transportation Systems Center, Energy Demand Branch, Sept. 11, 1978. Unpublished memorandum.

89 km/h (55 mph) impacting a current bridge rail design at a 20° angle.[4] Much deceleration occurred when the vehicle snagged a bridge rail post in this test, and the right door window was broken by a dummy occupant. In a comparable test with a 1.02 Mg (2,250 lb) vehicle the vehicle redirection was much smoother. Wheel size and rail geometry are important factors in this problem. This study is still underway; recommendations to reduce such problems are planned.

Figure 19.—Vehicle after test shown in figure 18.

Summary

Existing roadside safety devices have played an important role in reducing the severity of ran-off-the-road accidents. The expected increase of mini-sized vehicles makes it necessary to reexamine highway safety hardware development and acceptance criteria. Important safety questions have been identified and work has begun to provide the needed answers.

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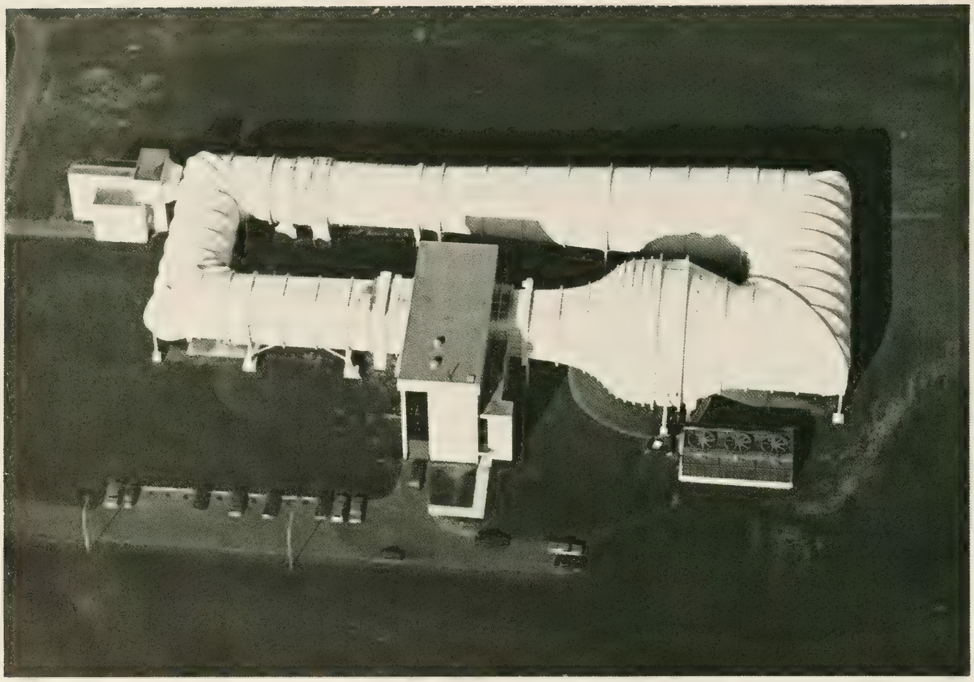
John G. Viner has been Chief of the Protective Systems Group, Structures and Applied Mechanics Division, Federal Highway Administration, since 1970. For the past 13 years he has been engaged in the physical aspects of highway safety research, having been involved in the development of roadside safety hardware. Prior to that he conducted structural vibration research for the Naval Ship Research and Development Center.



The National Aeronautical Establishment, National Research Council, 9 m by 9 m (30 ft by 30 ft) low speed wind tunnel in Ottawa, Canada.

Sectional Model Versus Full Model Wind Tunnel Testing of Bridge Road Decks

by
Robert L. Wardlaw

This article reviews fundamental engineering aspects of the wind tunnel modeling of bridge road decks. The advantages and disadvantages of current wind tunnel techniques for predicting the dynamic response of bridge road decks to wind are examined. The sectional model approach is shown to be reliable in predicting vortex shedding excitation and flutter. However, it may be excessively conservative because of the omission of wind turbulence in the simulation. Complete simulation of the Earth's surface wind is possible with the full model so that the effects of turbulence are fully accounted for. It is concluded that for major bridges, extensive aerodynamic investigations, including the more expensive full model, are justified.

Introduction

In a 1973 state-of-the-art review of bridge road deck aerodynamics, it was observed that the deck behavior could not be predicted satisfactorily by analytical means alone. (*1*)[1] Gaps in the knowledge of the effects of turbulence and terrain were evident. More extensive investigations of the motion of completed structures were necessary to obtain more information on structural damping levels and, more importantly, to provide a basis for the comparison of model and full-scale measurements.

Although these problems have not been entirely resolved, significant progress has been made in all areas through field and laboratory investigations in several countries including Australia, Canada, Japan, and the United States. Therefore, it is timely to discuss some of the implications of these advances on techniques and approaches to wind tunnel investigations. The advantages and disadvantages of sectional model versus full model testing should be reexamined; however, more research is needed before all questions can be answered.

The effect of the gustiness or turbulence in the Earth's surface wind on the bridge dynamic behavior is the major issue needing discussion. In addition, the

[1] Italic numbers in parentheses identify references on page 17.

adequacy of the traditional gust factor approach for estimating peak wind loads in turbulent wind must be challenged. The following questions arise:

- Does testing in smooth flow give overly conservative estimates of aerodynamic stability in natural wind?
- Can dynamic behavior in natural wind be predicted analytically from results obtained with sectional models in smooth flow?
- Can the Earth's surface wind be modeled adequately in the wind tunnel?
- Can the bridge be modeled precisely at reduced scale?
- Do the advantages of the sectional model approach—low cost, large scale, configuration flexibility, and short model preparation time—have to be sacrificed for the advantages of the full model—simulation of the natural wind, inclusion of three-dimensional aeroelastic effects, and inherent modeling of the bridge modal dynamics?

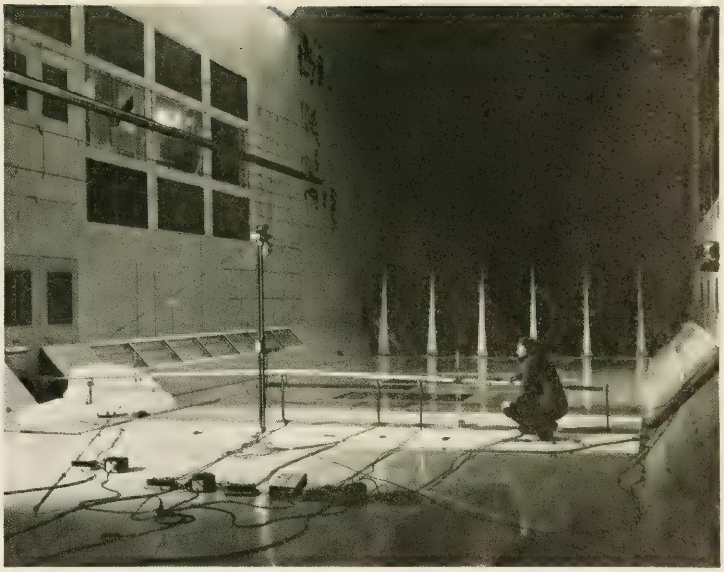


Figure 1.—The 1:110 scale full model of Lions' Gate Bridge in a 9 m by 9 m (30 ft by 30 ft) wind tunnel.

These and other questions on wind tunnel investigations of bridge behavior will be examined in this article. A compromise between the full model and sectional model—the taut-strip model—will also be discussed. (2)

Figure 2.—Closeup view of the model shown in figure 1.

Wind Tunnel Testing Techniques

The full model

The full model, a reduced scale, geometric facsimile of the entire prototype bridge, includes all structural elements—the towers, the suspension cables, the road deck, and the road deck hangers. For dynamic studies, the mass, the mass distribution, and the elastic characteristics of the prototype must also be modeled according to established scaling principles. The scale ratio for a long-span bridge may be very small; for example, the model of a bridge 1.0 km (3,280 ft) long would have to be constructed at a scale ratio of about 1:400 to be tested in a wind tunnel test section 2.5 m (8.2 ft) wide. Clearly, large reductions in size present difficulties in model design and construction. Figures 1 and 2 show a 1:110 scale full model in a 9 m by 9 m (30 ft by 30 ft) wind tunnel. The model is a widened version of the Lions' Gate Bridge that crosses the Burrard Inlet in Vancouver, British Columbia, Canada. Aerodynamic "spires" at the entrance to the test section (fig. 1) generate the shear and turbulence properties of the Earth's surface wind. The depth of the wind layer, which depends on the roughness of the terrain at the bridge site, will be about 300 to 600 m (985 to 1,970 ft); consequently, at a scale of 1:110, the wind layer will be between 2.7 and 5.5 m (9.0 and 18.0 ft) deep in the wind tunnel.

The sectional model

The term sectional model is derived from the aeronautical engineering practice of measuring the two-dimensional or sectional properties of aerofoils in wind tunnels by using constant section models that span the test section.

Rather than model the entire bridge, a model representing a short, midspan section of the deck can be constructed to study the aerodynamics of the bridge road deck. The model spans the test section and is supported rigidly at the walls for force measurements or mounted on pairs of springs for dynamic measurements. For dynamic measurements, the mass, the mass distribution, and the elastic properties must be modeled according to scaling criteria, as is required with the full model. The bending mode natural frequency is controlled by the spring stiffness, and the ratio of the bending to torsional mode frequencies is controlled by the spacing between the two component springs of the pair located at each end of the model.

Figure 3 shows a 1:30 scale sectional model of Long's Creek Bridge, a cable-stayed orthotropic girder bridge on the Trans-Canada Highway in New Brunswick, Canada. The model was tested in a vertical wind tunnel 4.6 m (15.0 ft) in diameter. The junction between the ends of the model and the wall introduces three-dimensionality into the flow. To minimize this "end effect," the model should be as slender as is practical; a span-to-chord ratio of 4:1 would be acceptable. For testing in a test section 2.5 m (8.2 ft) wide, a model chord of 0.6 m (2.0 ft) would be suitable; for a bridge road deck 30 m (98 ft) wide, this would give a scale ratio of 1:50.

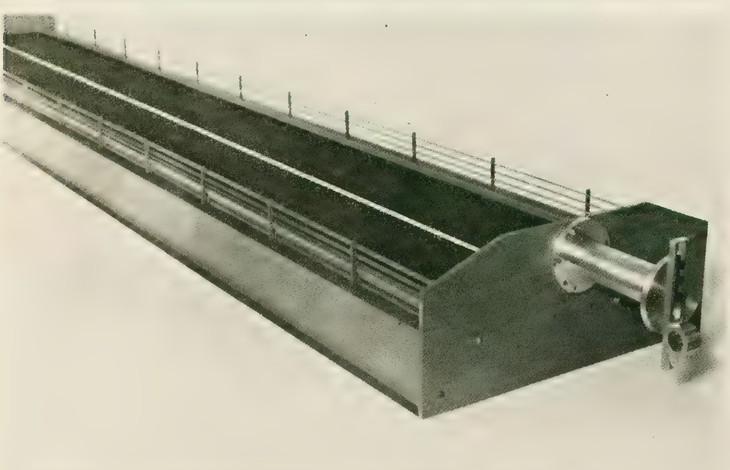


Figure 3.—The 1:30 scale sectional model of Long's Creek Bridge (original road deck configuration).

Because the sectional model is simpler and has a larger scale than the full model, it is less expensive to construct. Also, the sectional model can be built more quickly and can be modified quickly for examining corrective configuration changes. Although the air flow can be made turbulent (for example, by using coarse upstream turbulence grids), it is not fundamentally possible to simulate all of the natural wind properties with the sectional model, particularly the large size of the gusts. Research to develop large longitudinal scale, two-dimensional turbulence will be particularly useful in sectional model studies. (*3*)

Although road deck response to natural wind turbulence cannot be determined directly, it may be possible to analytically predict the response from sectional aerodynamic measurements made in smooth flow. (*4*) If successful, this approach would produce an ideal method for predicting bridge behavior in natural wind. However, problems still remain. One question that must be resolved is whether the aircraft aerodynamicist's classical strip theory applies to bridge road decks. This theory divides an airfoil into thin chordwise strips that are assumed to behave independently so that the overall airfoil performance can be calculated by spanwise integration.

It is common for slender elastic structures to be aerodynamically unstable because of their cross-sectional geometry—energy extracted from the airstream causes oscillatory motion of the structure. As will be discussed later, different aeroelastic mechanisms may account for the instability. In some cases the phenomenon is amplitude limited, but even though excessive stress levels may not result, the user will consider the bridge road deck unacceptable. (*5*) Other mechanisms can cause catastrophic motion levels in a few motion cycles.

It has been conventional practice to use the spring-mounted sectional model in smooth flow to assess road deck aerodynamic stability. For amplitude-limited excitations, amplitudes must not exceed a prescribed criterion of acceptability. (*5*) For catastrophic instabilities, motion must occur only at windspeeds greater than those expected at the site. Field and wind tunnel experience indicates that the sectional model conservatively predicts instabilities on prototype bridges. No unacceptable aerodynamic instability of prototype bridges has been observed when the road decks were wind tunnel tested in smooth flow before construction. The Golden Gate Bridge, the original Tacoma Narrows Bridge in Washington State, and the Long's Creek Bridge were not tested before construction and were aerodynamically unstable. (*6-8*) Subsequent wind tunnel studies in smooth flow confirmed the instabilities and demonstrated that the problems could have been avoided if wind tunnel testing had been done in advance.

The taut-strip model

With the taut-strip model, the road deck is attached to two parallel taut wires suspended across the wind tunnel test section. (*2*) The vertical and horizontal bending mode natural frequencies are controlled by the wire tension, and the ratio of the bending mode to torsional mode natural frequencies is controlled by the distance between the wires.

Advantages of the taut-strip model over the sectional model are that natural wind turbulence can be appropriately simulated and the three-dimensionality of the model deformations are inherent. In addition, the taut-strip model is simpler in concept than the full model.

The taut-strip model scale is constrained not so much by the size of the prototype bridge as by how deep a wind layer, and thus how large a turbulence scale, can be accommodated in the wind tunnel. Wind layer depths may be about one-half the height of the wind tunnel test section. Therefore, in a test section 2.4 m (7.9 ft) high, a wind layer 1.2 m (3.9 ft) deep could be developed, representing a full-scale layer 300 m (985 ft) deep at a scale ratio of 1:250. If the model were installed in a test section 2.4 m (7.9 ft) wide, 600 m (1,970 ft) of bridge road deck could be represented. Although the taut-strip model may offer some advantage in scale over the full model, the scale will still be much smaller than that of the sectional model.

Model Scaling Considerations

Model scale observations can only be extrapolated with confidence to prototype scale if sound scaling principles are used in the design of the model and the experiment. This insures that the relative magnitudes of the forces involved in bridge dynamics—gravitational, inertial, aerodynamic, elastic, and structural damping—are the same for the model and the prototype and that the motion amplitudes are in the same proportion as the geometric scale ratio.

Dynamic scaling

The model scale value of the wind velocity must be lower than the full scale value by the square root of the geometric scale ratio for a full model of a long-span bridge to insure that gravitational forces are correctly proportional to aerodynamic and other bridge dynamic forces. Thus, for a scale ratio of 1:100, a full-scale windspeed of 100 km/h (62 mph) would be represented by a model scale windspeed of 10 km/h (6.2 mph) in the wind tunnel.

It follows that the elastic properties have to be adjusted so that the stiffnesses are lower than they would be for a replica model that is made from the same materials as the actual bridge. For some structures, this can be done by using materials that have lower values of the elastic modulus, E, and by reducing material thickness. The resulting deficiency in mass can be corrected by adding distributed nonstructural mass. For open truss designs, however, the elastically scaled values of component thickness are usually impractically small and some other way of constructing the model has to be devised. A structural spine, on which are mounted nonstructural modules that give the correct aerodynamic shape, can be designed for the road deck.

Aerodynamic scaling

Aerodynamic scale effects can cause problems when extrapolating model results to full scale. The wind flow at model scale may not be "similar" to flow at full scale. For example, at high windspeeds, the flow changes over a large diameter circular cylinder. The Reynolds Number, the aerodynamic similarity parameter, is a measure of aerodynamic scale. The Reynolds Number is as follows:

$$Re = \rho U b / \mu$$

Where,

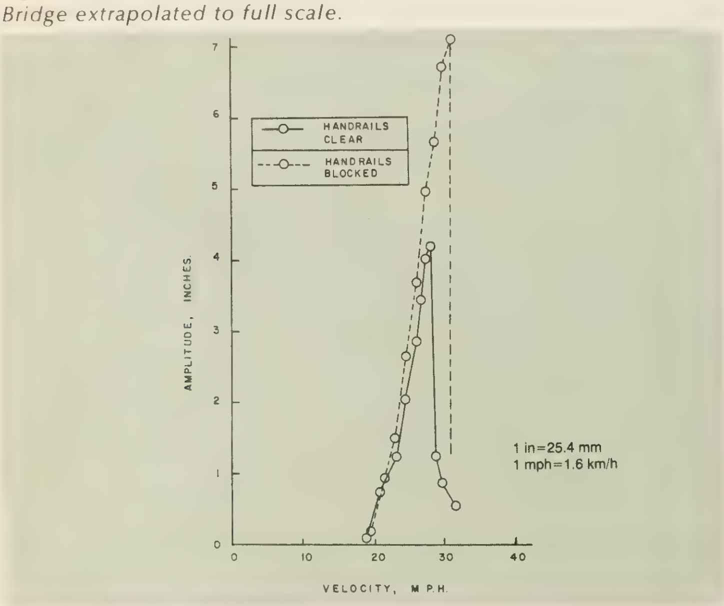
ρ = Air density.
U = Windspeed.
b = Cylinder diameter or bridge deck width.
μ = Air viscosity.

At large values of the Reynolds Number, the drag coefficient ($C_D = D/\frac{1}{2}\rho U^2 b$, where D = aerodynamic drag/unit length) for a circular cylinder abruptly drops in value. This is because the cylinder wake becomes narrower—a consequence of a rearward shift on the cylinder surface of the "separation points" where the smooth flow on the forward part of the cylinder detaches from the surface of the cylinder. The separation point is fixed at the sharp upstream corner of rectangular cylinders; as a consequence, the Reynolds Number dependency disappears. This is not the case for rectangles with rounded corners. At a very small scale, it is difficult to reproduce precisely the prototype geometric shape, particularly the small corner radii of structural components.

It is difficult to answer the question of how large the model scale should be to avoid Reynolds Number problems. Model-prototype comparisons and wind tunnel investigations over a range of scales indicate that Reynolds Number scaling is not necessary. Wind tunnel sectional model measurements of the Long's Creek Bridge (fig. 4) at a 1:30 scale are extrapolated to prototype scale in figure 5. The peak amplitude of 110 mm (4.3 in) near 44 km/h (27 mph) agrees with

Figure 4.—The modified Long's Creek Bridge.

Figure 5.—Sectional model response of the unmodified Long's Creek Bridge extrapolated to full scale.



several observations at the bridge site. The narrow velocity range of excitation is the same for the model and the prototype. Winds normal to the bridge at velocities above the critical range—56 km/h (35 mph)—were measured at the site and no bridge motion was observed. Amplitudes as high as 200 mm (7.9 in) were also reported. These larger responses were probably because of snow blocking the handrails. In the wind tunnel a peak amplitude of 180 mm (7.1 in) was recorded with blocked handrails.

Effects of Turbulence

The Earth's surface wind layer

The planetary winds are retarded near the Earth's surface by flow resistance from elements on the ground and by fluid friction associated with the air viscosity. As a result of this shearing action, the velocity varies from zero at the surface—the fluid dynamicist's no-slip condition—to gradient wind at about 300 to 600 m (985 to 1,970 ft) above the surface as shown in figure 6.

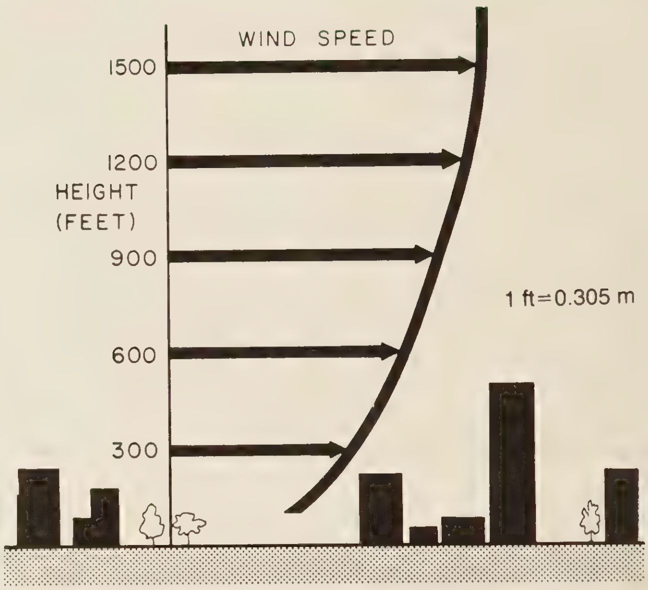


Figure 6.—The variation of mean wind velocity with height.

The shearing action also causes mechanical agitation of the flow, or turbulence. To an observer fixed on the ground the turbulence manifests itself as gustiness with continual and sometimes abrupt changes of direction and magnitude. A measure of the magnitude of the wind's fluctuating component is its root-mean-square (RMS) value, which is known as the turbulence intensity. At the height of the gradient wind, the turbulence intensity is nearly 0. However, it increases toward the surface, and the longitudinal or streamwise component can be as high as 30 to 40 percent of the local mean windspeed near the ground. Peak excursions can be several times the RMS value.

The wind turbulence is characterized by a nearly random distribution of the physical size of the disturbances. In the longitudinal direction the length can vary from near zero to hundreds of metres (feet). The lateral dimensions of the disturbances are somewhat smaller. The random fluctuations in wind velocity, seen by a fixed observer, represent wind energy distributed over a range of frequencies.

Modeling the wind layer

There are several methods for simulating wind layer properties in the wind tunnel. A well-established approach used in special-purpose wind tunnels with long working sections is to develop the layer "naturally" by having the wind blow over a long fetch of surface elements. A second method, suitable for aeronautical wind tunnels with shorter working sections is to install spires at the entrance to the test section as shown in the background of figure 1. Both of these techniques make it possible to develop wind layer depths that approach one-half the height of the test section.

Testing in turbulent wind

The effects of turbulence on bridge behavior depend on the scale of the turbulence relative to the size of the bridge and the intensity and frequency spectrum of the turbulence. For example, if the lateral scale of the turbulence were large enough that the velocity at any one instant was constant along most of the span, turbulence would play a different role than if the scale were much smaller and the velocity varied considerably along the span. Similarly, if the natural frequencies of the bridge were in a high energy part of the spectrum, the response to turbulence would be greater than if the frequencies were in a low energy part. It is important to simulate all of the properties of turbulence. This cannot be done with sectional models because large scale turbulence components are absent. Therefore, the full model or taut-strip model must be used to fully study the effects of turbulence.

Buffeting response

Because there is significant turbulent energy near bridge natural frequencies, the dynamic response becomes important at high windspeeds for lightweight, low damping, modern suspended bridges, and the gust factor approach to estimating loads may not be satisfactory. An alternative is to use full models in simulated wind to estimate the stress levels caused by the dynamic response from motion. (*9*) As experience with full models increases, the goal of predicting dynamic response from sectional model data should be pursued vigorously.

Road deck flutter

Flutter is an oscillatory instability of the road deck that occurs above a critical windspeed. It is characterized by rapid buildup of amplitude; catastrophic levels may be reached in a few cycles of motion. At higher windspeeds the buildup rates normally increase. Torsion is usually the dominant vibration mode, but flutter is a consequence of aerodynamic coupling between vertical bending and torsion, although single-degree-of-freedom torsional flutter may occur. Unless the shape of the bridge is aerodynamically stable, it is essential that critical flutter speeds be well above windspeeds expected at the site.

Flutter stability can be readily assessed using sectional models. These assessments seem to be a conservative approach to design against flutter because the stability in turbulent flow will be greater than in the uniform flow of the sectional model test. There is evidence that the sectional model method can be too conservative. The Halifax Harbour Bridge (*2*) and the Lions' Gate Bridge (*9, 10*) are open truss bridges, and for both bridges, the flutter instability observed with sectional models was not observed for the full models in the turbulent flow, even for velocities well above the sectional model critical speeds. Torsional response of the Lions' Gate Bridge is shown in figure 7.

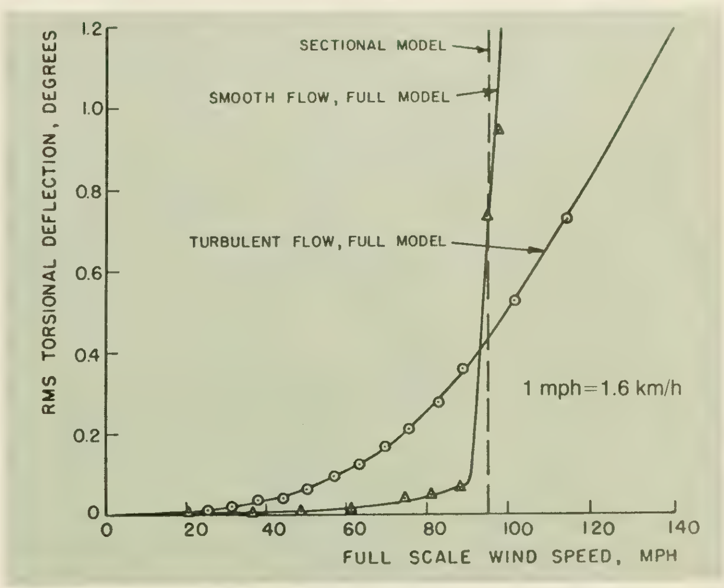


Figure 7.—The Lions' Gate Bridge full model torsional response at the 1/10th span position in smooth and turbulent flow. (10, 11)

Evidence that the sectional model is conservative is less compelling for the plate girder and box section road deck structures than for the open truss design. With the taut-strip model of the H-section road deck of the original Tacoma Narrows Bridge, turbulence inhibited

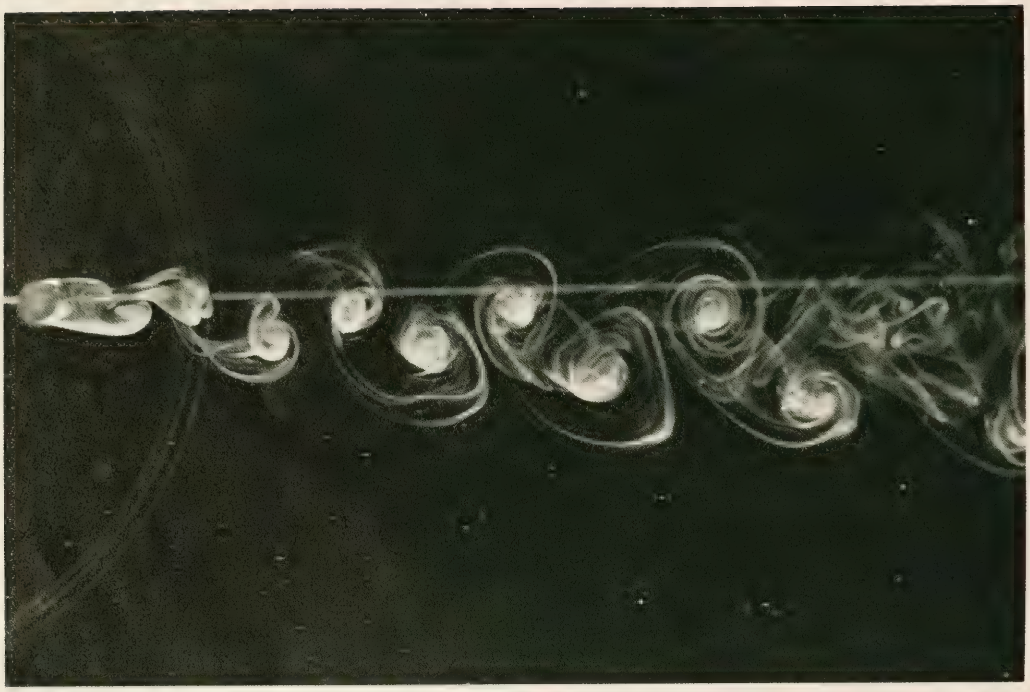


Figure 8.—The vortex wake behind a circular cylinder.

noncatastrophic, possibly vortex-induced, motion at subcritical windspeeds but only marginally postponed catastrophic torsional motion. (*2*) The effects became greater as the turbulence intensity was increased to represent rougher terrain.

Vortex shedding response

The wake structure behind slender, bluff structures is an orderly sequence of vortexes, as illustrated in figure 8 by visualization of the flow behind a circular cylinder. The vortexes form alternately on either side of the wake with marked periodicity.

As the windspeed is increased over a slender, elastic structure, the frequency of vortex formation increases. When this frequency matches a structural natural frequency, large crosswind vibration amplitudes develop. The critical windspeed range often will be at common windspeeds. The vertical bending oscillations of the Long's Creek Bridge, shown in figure 5, are caused by the periodic wake vortexes. (*8*) The finite, nondestructive amplitudes and the narrow velocity range over which the motion occurs are characteristic of vortex shedding response. Torsional modes are equally susceptible to this form of excitation. Although in the short term the amplitudes do not damage the bridge, they can be large enough to make the bridge unacceptable to the user and may result in long term damage.

Sectional models are suitable for studying vortex shedding response because easily made configuration changes are a helpful feature for developmental or remedial investigations. Experience with the Long's Creek Bridge demonstrates that the sectional model can reliably predict full-scale vortex shedding behavior for plate girder decks. Whether the sectional model is overly conservative for more streamlined box sections, for truss bridges, or for higher turbulence levels has yet to be established.

Conclusions

Although advantages and disadvantages of the full, sectional, and taut-strip model approaches have been discussed, it is not possible with the present state of the art to favor one model approach. The investigative procedure or combination of procedures to be used will depend on a variety of considerations—the size of the bridge, the road deck configuration, and the leadtime available.

As previously stated, advantages of the sectional model are low cost, large scale, configuration flexibility, and short model preparation time. The model also provides data for prediction procedures now under development. (*4*)

The advantages of the full model are that the Earth's surface wind can be completely simulated so that the turbulence response is obtained directly and the effects of turbulence on flutter and vortex shedding are intrinsic. Also, all three-dimensional aeroelastic effects are modeled implicitly.

The taut-strip model can also be tested in simulated natural wind but it is simpler and less costly than the full model. Within the limits of this simplified model, some of the three-dimensional aeroelastic effects are included. To a certain extent it shares the low cost and short model preparation time of the sectional model. Analytical procedures for predicting prototype behavior from taut-strip measurements have been described in the literature. (*2*)

For a large bridge, extensive aerodynamic investigations are justified and can include a full model wind tunnel investigation. It can be advantageous to precede the full model investigation with sectional investigations so that stable aerodynamic deck shapes, free from vortex shedding excitation, are developed quickly and cheaply. This will also permit the assessment of Reynolds Number effects for the small scale full model. If the full model is difficult to justify for smaller bridges, the sectional model or the sectional model plus taut-strip model can be used.

In choosing the testing procedure it should be remembered that the cost of the aerodynamic investigation is a fraction of 1 percent of the cost of the full-scale structure. However, because of the importance of the aerodynamic behavior to the final design, the selection of the investigative technique should not be unduly restricted by cost considerations.

Although our knowledge of the dynamic behavior of bridge road decks and of investigative procedures has improved, there are still gaps to fill, particularly concerning turbulence, three-dimensional effects, and analytical prediction procedures. Future research should emphasize model to full-scale comparisons to further validate investigative procedures.

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[2]Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

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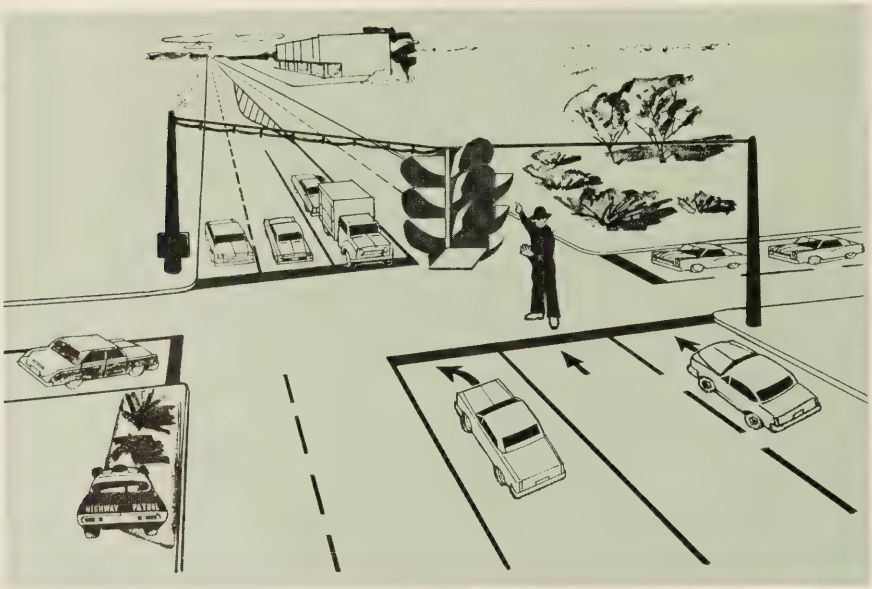

Problems Associated With Power Failures at Signalized Intersections

by
Woodrow W. Rankin
and Merton J. Rosenbaum, Jr.

Some traffic engineers have expressed concern about the potential adverse safety effects of power failures at signalized intersections. This article discusses the results of a workshop and survey that indicate that power failures at signalized intersections, particularly areawide failures, are not a significant safety problem.

Introduction

Periodically, the Federal Highway Administration (FHWA) Office of Highway Safety asks State and local jurisdictions to help identify critical subjects for highway safety research. In 1978 traffic engineers from Erie County, N.Y., San Juan, Puerto Rico, and the Florida Department of Transportation submitted problem statements requesting that power failures at signalized intersections be studied. They specifically requested the following:

- Develop methods for interim control of traffic signals when power fails during peak traffic periods.
- Evaluate potential safety hazards associated with power failure at signalized intersections.
- Develop criteria and warrants for installing power "failsafe" devices at signalized intersections.

Because the type and size of power failures at signalized intersections were not well defined, the FHWA Office of Research sponsored a small study to determine the potential seriousness of power failures at signalized intersections. The results of this study, which was performed by the Highway Users Federation for Safety and Mobility (HUFSAM), are summarized in this article as are complementing data from an independent national survey conducted by Herman Kimmel and Associates (HKA). The data collected by HKA and discussed in this article were analyzed by FHWA personnel.

The study techniques used by HUFSAM and the survey techniques used by HKA are briefly discussed. Also discussed are areawide and localized power disruption, the relationship of power failures to traffic accidents, and procedures used by operating agencies when signal outages occur.

Study and Survey Methodology

HUFSAM reviewed published and unpublished literature and conducted a workshop with traffic engineers from Dade County, Fla., Indianapolis, Ind., New York, N.Y., Tulsa, Okla., and the Illinois Department of Transportation to study power failures at signalized intersections. Additional information was obtained from traffic engineers

Related to the problem of signalization, we'd like to obtain information on the extent and frequency of *Power Failures at Signalized Intersections.*

1. Do you consider power failures at signalized intersections a major problem in your jurisdiction? (Check one):
MAJOR ☐ MINOR ☐ NO PROBLEM ☐

2. How would you rank the major causes of power failure in your area?
1. ________ 2. ________ 3. ________ 4. ________

3. What measures are taken to solve the problem?
NONE ☐ BACKUP SYSTEM ☐ OTHER ________

4. Could you estimate the number of power failures at signalized intersections per year in your jurisdiction?
Less than 50 ☐ 50-100 ☐ 100-200 ☐ 200+ ☐

5. What legal issues exist related to this problem? ________

6. Please estimate the number of signalized intersections in your state by the following categories:
RURAL ______ SUBURBAN ______ URBAN ______ TOTAL ______

7. What are some of the options available to solve the problem of power failure at signalized intersections?

8. Is there evidence of an increase in traffic accidents when signals are out due to power failure? Rate severity of accidents. LOW MEDIUM HIGH YES ☐ NO ☐

9. What issues do you believe should be investigated related to power failures at signalized intersections?

10. Would you be willing to serve on a panel to answer further questions on power failures at signalized intersections?
YES ☐ NO ☐

Figure 1.—HKA questionnaire on power failures at signalized intersections.

responsible for the operation and maintenance of signal systems in Baltimore, Md., Milwaukee, Wis., San Diego, Calif., Erie County, N.Y., King County, Wash., and Montgomery County, Md. The Maryland Department of Transportation and some manufacturers and suppliers of signal equipment were also contacted by HUFSAM during the study. Each traffic engineer who furnished information is responsible for the operation and maintenance of more than 300 signalized intersections.

In HKA's survey, a questionnaire (fig. 1) was sent to over 2,500 traffic engineers located in or associated with local jurisdictions and State agencies. A range of jurisdictions responded to the questionnaire including Coronado, Calif., with 10 traffic signals and the California Department of Transportation with 130,000 traffic signals. Figure 2 shows the geographical distribution of the responses.

Power Failures

Causes of signal outages can be divided into three categories: Areawide irregularities in power supply, localized irregularities in power supply, and signal equipment failure not related to power supply. Because this article is concerned with power failure related outages, only the first two categories will be addressed.

Areawide power disruptions

Areawide power disruptions include bulk power interruptions (blackouts), load reductions (brownouts), and other emergency situations. In the first 6 months of 1978, 20 blackouts were reported in the United States compared to 49 blackouts reported in 1977. (*1–4*)[1] These power interruptions lasted from 5 minutes to 6 days for various large groups of customers. Such large outages were caused by natural events—tornadoes, hurricanes, heavy flooding, severe ice storms, earthquakes, and other disasters—as

[1] Italic numbers in parentheses identify references on page 25.

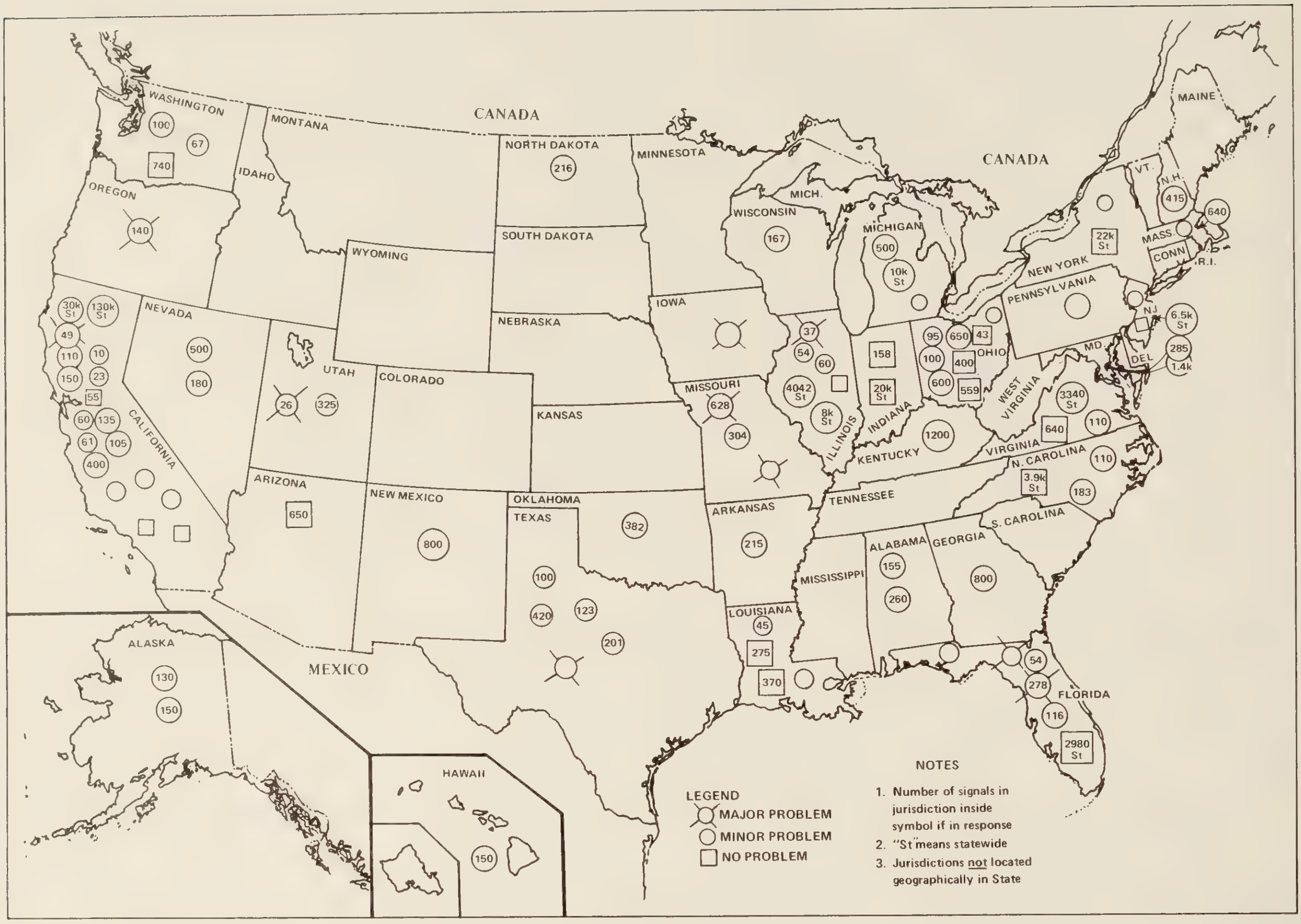

Figure 2.—Response distribution from HKA survey.

well as by human switching errors and equipment failures that result in outages cascading throughout a system.

HUFSAM concluded that areawide blackouts do not cause traffic safety hazards. Because the public readily perceives a problem, most drivers exercise caution.

Localized power disruptions

Localized power irregularities include the effects of lightning and high winds, unexplained surges (or "spikes") in normal power transmission, and small interruptions caused by switching lines or sources in power grids or networks.

Lightning may damage traffic signal equipment by a direct hit, by inducing voltage and current surges into wires and cables that feed into signal controls, or by causing the power supply to a signalized intersection to fail.

Some areas of the United States have more electrical storm activity than others. A National Aeronautics and Space Administration report contains an isoceraunic map of the United States, that indicates that the most active area is in central Florida, where thunder can be heard more than 100 days per year. The least active area is along the west coast, with about 5 days of thunder per year. A "flash density," which includes flashes between clouds and flashes to the ground, between 0.3 and 1.0 flashes per square kilometre (0.8 and 2.6 flashes per square mile) on each thunderstorm day affects traffic control equipment because changes in the density may induce surges in power and signal lines. (5)

Lightning often causes failure of the power supply to a controller and damage to detector amplifiers. To reduce traffic control equipment susceptibility to damage from induced surges, various lightning protection devices may be adapted to existing systems or installed on new systems. These devices are increasingly important with new, sophisticated systems involving solid-state electronics. The new systems are more efficient than earlier systems, but more vulnerable to power surges.

Localized outages also occur when power transmission lines feeding nearby signalized intersections are damaged by wind or downed poles from vehicular accidents. The duration of such outages depends on the time it takes for a maintenance crew to correct the problem or for power to be rerouted in established grids. In some areas up to 50 percent of the outages are caused by irregularities in the normal power supply, resulting in surges that blow fuses or cause other equipment failure.

Frequency and duration

Frequency and duration of power outages, and the resulting signal outages, vary with the geographic area, the cause of an outage, and the type of signal equipment. Computerized systems can detect and report many outages that normally would not be noticed or recorded. Average frequency and duration figures therefore have little significance.

HUFSAM reported the following experiences:

- A Southwestern city traffic agency with 385 signalized locations reported an average of two localized power failures per month, lasting between 15 minutes and 2 hours. Most of these were caused by wind, electrical storms, or accidents.

- A Midwestern State agency estimated an average of one signal outage per signal per year from various causes, although the agency indicated that small outages were not a serious problem. The agency also indicated there may be power outages that are not reported. Many of these outages result from problems in power transmission lines that often are automatically corrected. In areas with a well designed power network these outages may be detected and corrected in 2 to 5 minutes by switching the power supply.

- A Midatlantic State agency reported an annual signal outage rate of approximately 20 percent of the number of traffic signals. These outages lasted 15 minutes to more than 3 days, were primarily caused by electrical storms, and usually affected only one signal per incident.

- A recent week's data from a Midatlantic city agency (where controllers are on a central computer system) indicated an annual outage rate of about 36 percent of the number of controllers. Duration averaged slightly over 35 minutes.

- A Florida county traffic agency with 1,760 signalized locations reported that in fiscal year 1978 traffic maintenance crews responded to 193 signal outages caused by power failures not reported by the power company. This accounts for about 1 percent of all trouble calls responded to. To measure the reliability of the area's power service in conjunction with the phasing in of central computer control for the traffic signals (about 45 percent complete), the agency investigated power supply outages (hits) reported by area computer facilities. One facility, discerning hits as short as one power cycle, reported an average of 18 hits per week with one-half lasting 0.5 to 30 seconds. Another computer facility that identified hits of 1 second and longer reported 73 hits over 1 year, 4 of which were major areawide power failures.

- A north-central city agency reported that in a recent 1-year period there were 231 power failures in a network of 720 signalized intersections: 103 were caused by local power company troubles (15 percent were weather related), 34 by vehicular accidents, and 94 by vandalism or contractor damage.

- A west coast city agency reported approximately six power failures per year lasting from 10 minutes to 4 hours and averaging 45 minutes to 1 hour. These failures were caused by electrical storms, equipment failures, and accident damage. Ten to twenty of the city's 650 signalized intersections may have been affected because of common power feeds.

- A large east coast city agency reported that individual signal outages occur annually on approximately 60 percent of all signalized intersections. For the same period the power company reported power failures on about 4 percent of the number of signalized intersections. Signal outages were caused by blown fuses, coordination failure, vandalism, mechanical failures, and defective traffic cables and power company cables.

Some rural areas experience an above average number of power failures that are not always caused by electrical storms. However, as power suppliers become more reliable, this is a decreasing phenomenon.

The number of contiguous signalized intersections affected by a power failure varies with the signal system and where the power fails in the system. Localized power outages usually affect 3 to 10 closely grouped signalized intersections, with 3 being the more common. More intersections may be affected when groups are tied to a common power feed (not usual practice) that fails or when there is an areawide outage.

Responses to HKA's questions support these HUFSAM findings. Only 10 percent of HKA's respondents considered power failure to be a major problem, 72 percent a minor problem, and 18 percent considered power failure to be no problem. The estimated number of annual power failures at signalized intersections in the respondents' jurisdictions (question 4 in figure 1) is reported in table 1.

Table 1.—Annual power failures at signalized intersections

Estimated annual power failures	Respondents
	Percent
Less than 50	72
50 to 100	21
101 to 200	3
Greater than 200	4

The information collected by HUFSAM and HKA indicates that the annual number of signal outages from power failures that agencies can expect is from 10 to 100 percent of the number of signalized intersections. These outages last an average of 30 minutes to 1 hour.

Power Failures and Accidents

Statistical data for relating accidents to signal outages are either unavailable or unsuitable. This may be part of the reason few traffic engineers responsible for signal operation and maintenance regard signal outages as a severe accident problem.

A major obstacle in effectively linking signal outages and accidents is that most accident reports record (if at all) only that a traffic control device failed and that this contributed to the accident. For a signal outage accident analysis to be useful, the accident report narratives must be reviewed to establish whether a traffic control device failed because of a signal outage and if so, to what extent the outage contributed to an accident. Also, signal outage reports would have to be reviewed to determine if the outage was caused by a power failure.

Because the public is usually aware of areawide power outages, traffic engineers attending HUFSAM's workshop felt that areawide outages are not as hazardous as localized outages. Also, large outages usually accompany other conditions that cause the public to be more cautious.

The following signal outage accident experiences were reported by HUFSAM:

- A large east coast city agency determined that from January to June 1978 approximately 0.25 percent of accidents occurred where a traffic control device failed. Traffic control device failure could be anything from a downed stop sign to a malfunctioning traffic signal. Accident reports did not always indicate whether the traffic control device failed.
- Data from a west coast city agency indicated that over 2½ years approximately 25 percent of all accidents during signal outages occurred when the signal malfunction was caused by a power failure. Approximately 0.04 percent of all accidents were attributed to power failure signal outages.
- A Southeastern county agency's reports of accidents at signalized intersections during a power outage revealed that failure of a traffic control device directly contributed to only one-third of such accidents.
- A Midwestern State agency's computer codes all accidents to intersections whether they occur at the intersection or midblock. Accident reports would have to be analyzed manually to determine an accident-to-signal outage relationship.
- A Southwestern city agency reported no accidents during a 24-hour areawide power outage caused by tornadoes and flooding. This was attributed to public awareness and the public exercising caution during a crisis.

In HKA's survey, 70 percent of the respondents indicated that power failures did not increase traffic accidents. Two-thirds of the respondents who indicated there was an increase felt the accidents were not severe.

It may be extrapolated from the information above that less than 0.1 percent of accidents occur when a signal outage was a contributing factor. Less than 0.05 percent of accidents occur because of signal outages from power failure, and these accidents are not severe. Available data are insufficient to yield nationally representative statistics. The extrapolations merely indicate an order of magnitude.

Signal Outage Operating Procedures

Traffic agencies learn about signal outages from their staffs, police, citizens, or central computers if the signal system is computerized. Generally, signal outages are self-evident in blackouts. Response procedures include mitigating the hazard, correcting the signal outage, and notifying the public. Agencies that maintain signals themselves respond to signal outages with their own maintenance crews. Agencies that contract for maintenance work have the contractors respond to signal outages.

Mitigating the hazard

Most maintenance and repair vehicles carry portable temporary stop signs that are usually erected if this control is adequate for traffic flow.

At signalized intersections where traffic flow is critical, police usually direct traffic while the signal system is being repaired. One jurisdiction stated that it has a civilian force to direct traffic during emergency situations. Police coverage in some rural areas may be difficult to obtain because of limited staff. It also was noted that police sometimes must stop directing traffic to respond to a more important call.

A Florida county uses "failsafe" devices (fig. 3) on many of its signalized intersections and intends to phase in full application. These battery-operated flashing units are located in the signal head but operate independently of the standard signal bulb and power source. They are actuated when power to the signal controller is cut off. Some of the signalized intersections have an emergency generator as a backup for power outages.

A Southwestern city agency installs four-way stop signs if three or fewer signalized intersections are affected by power failure. Stop signs are not used if more than three intersections are affected because the situation is similar to an areawide outage where people readily perceive the problem. In addition, an outage involving more than three intersections is usually caused by a transformer problem that often can be corrected faster than the traffic agency can respond with alternate traffic controls at all affected locations.

An Illinois State law stipulates that intersections with signal outages automatically be considered four-way stops. At urban arterial intersections that do not function well as four-way stops because of traffic flow conditions, police must direct traffic. As a result of the legislation, using temporary stop signs at intersections with signal outages is rare.

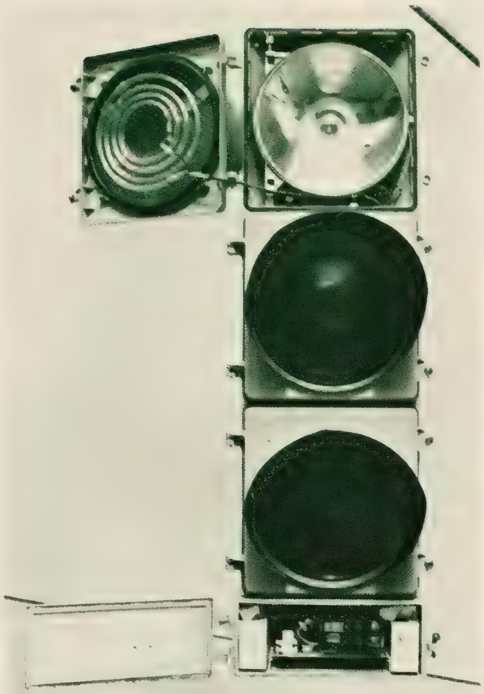

Figure 3.—"Failsafe" battery-operated traffic signal flashing unit. —courtesy of Metro Dade Department of Traffic and Transportation

Correcting the signal outage

A 1966 survey estimated that about 90 percent of State and city traffic agencies maintain and repair their own signals, while 10 percent contract for signal maintenance.[2] Standard operating procedures usually cover staff signal maintenance. Most contracted maintenance specifies a maximum allowable time to respond to an outage. In urban areas the time is typically 2 hours or less; longer times usually are specified in suburban and rural areas. Some agencies have service agreements with several contractors, providing for more flexibility. Agencies usually have a goal of less than 1 hour for response to an outage. Some urban areas indicate a normal response time of 30 minutes. An Indiana court action determined that 2½ hours was too long for response to outages so a closer repair unit was ordered. The court did not determine what the maximum allowable response time should be.

Agencies maintaining and repairing their own signals usually have crews on call for emergencies during offhours and weekends. In large cities crews may be on duty at all times. The goal of all traffic agencies is to be able to respond 24 hours a day to signal outages.

Planned outages at signalized intersections may require police control, stop sign erection, signal head bagging, or signal head removal, depending upon the expected duration of the outage.

Respondents to the HKA survey listed a range of options to use during power failures (question 7 in figure 1) including police control and several alternate power sources such as batteries, multiple power feed, solar, and generators. However, over 70 percent indicated that *no* measures were taken to solve the problem (question 2 in figure 1).

Notifying the public

Actions to notify the public often depend on how long the signal outage is expected to last, how critical the signalized intersection is, how widespread the power outage is, and the standard operating procedure of the traffic agency. Because areawide power outages are self-evident, no special notification efforts are needed.

[2] "Intersection Controller Survey," Battelle Memorial Institute, 1966. Unpublished.

In large urban areas, local radio stations often have helicopter surveillance of traffic during peak hours and broadcast traffic reports including information on signal outages. This information on outages is obtained by observation, direct communication with police, or monitoring police or citizen band (CB) radio.

Usually the public is notified of planned outages at critical intersections by radio or newspaper. In some cases residents have been notified by mail.

Traffic agencies with a central computer control system usually have a backup emergency power supply for computer operation. New York City's emergency power supply also powers radio and teletype equipment, providing direct teletype to radio stations. The traffic agency maintains special telephone numbers the public can call for emergency information.

It was noted in HUFSAM's workshop that CB radio, with its rapidly growing popularity, is providing more traffic information to the public. The Illinois Department of Transportation plans to study the potential of CB radios to determine the cause of a lane blockage on the Eisenhower Expressway in Chicago, Ill. A lane blockage detected by lane occupancy detectors will switch on a CB monitor at the traffic agency.

Participants at the HUFSAM workshop felt that informing the public reduces the potential hazards of outages at signalized intersections. They also felt that when approaching a signal outage a motorist who is unaware that an intersection is normally signalized is more of a hazard than a motorist who is familiar with the route. This is especially true if the driver is assuming the right-of-way on a major route and if the traffic volume is not heavy enough to control the driver's movement.

Power Outages and Signal Equipment

Although power restoration at signal outages may eliminate the potential for increased accident hazards and congestion, additional work is needed to restore some types of signal control equipment to normal operation. Restoration of any coordinated timing relationship of noninterconnected signal systems or the resetting of time clocks for switching signal cycles or offsets for multidialed isolated controls and interconnected coordinated systems often is required to regain traffic flow efficiencies at affected locations.

Followup maintenance to adjust timing is normally not required for isolated, single dial, pretimed controllers and most actuated controllers. It may be necessary, however, to reset the time clock mechanisms for isolated multidial, pretimed signal controllers and pretimed and background cycle actuated system-master controllers unless there is a backup spring drive for the time control. The adjustment is minor and can be made in a short time.

After a power failure outage, the adjustment of one or more of the controllers of a group of pretimed controllers that operate as a coordinated noninterconnected system is relatively simple but may require more than a nominal amount of time. Each affected controller must be checked and manually corrected to reestablish the proper time relationship between the zero or baseline time of the coordinated system and the start of the main street green.

The need for controller retiming after power failure signal outages is expected to decrease in the future. Intersection control and system control equipment that require these adjustments after a power failure are outmoded. As signal equipment is modernized and more sophisticated controls are installed, power failure signal outages are expected to have little, if any, adverse impact on traffic flow efficiencies after the power is restored and the signal resumes operation.

Conclusions

The consensus of the HUFSAM study and the HKA survey is that power failure at signalized intersections is considered by most traffic engineers to be a minor problem. The limited data indicate that annual power failure signal outages vary from 1 per signalized intersection to 1 per 10 signalized intersections, with the outage lasting an average of 30 minutes to 1 hour. The number and severity of accidents do not appear to increase significantly. Less than 1 accident in 1,000 occurs because of a signal outage and less than 1 accident in 2,000 occurs when power failure caused the signal outage.

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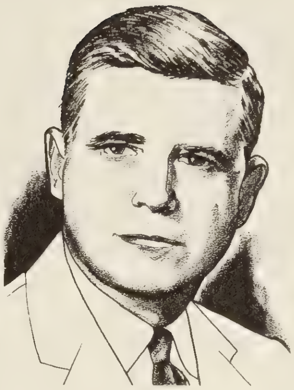

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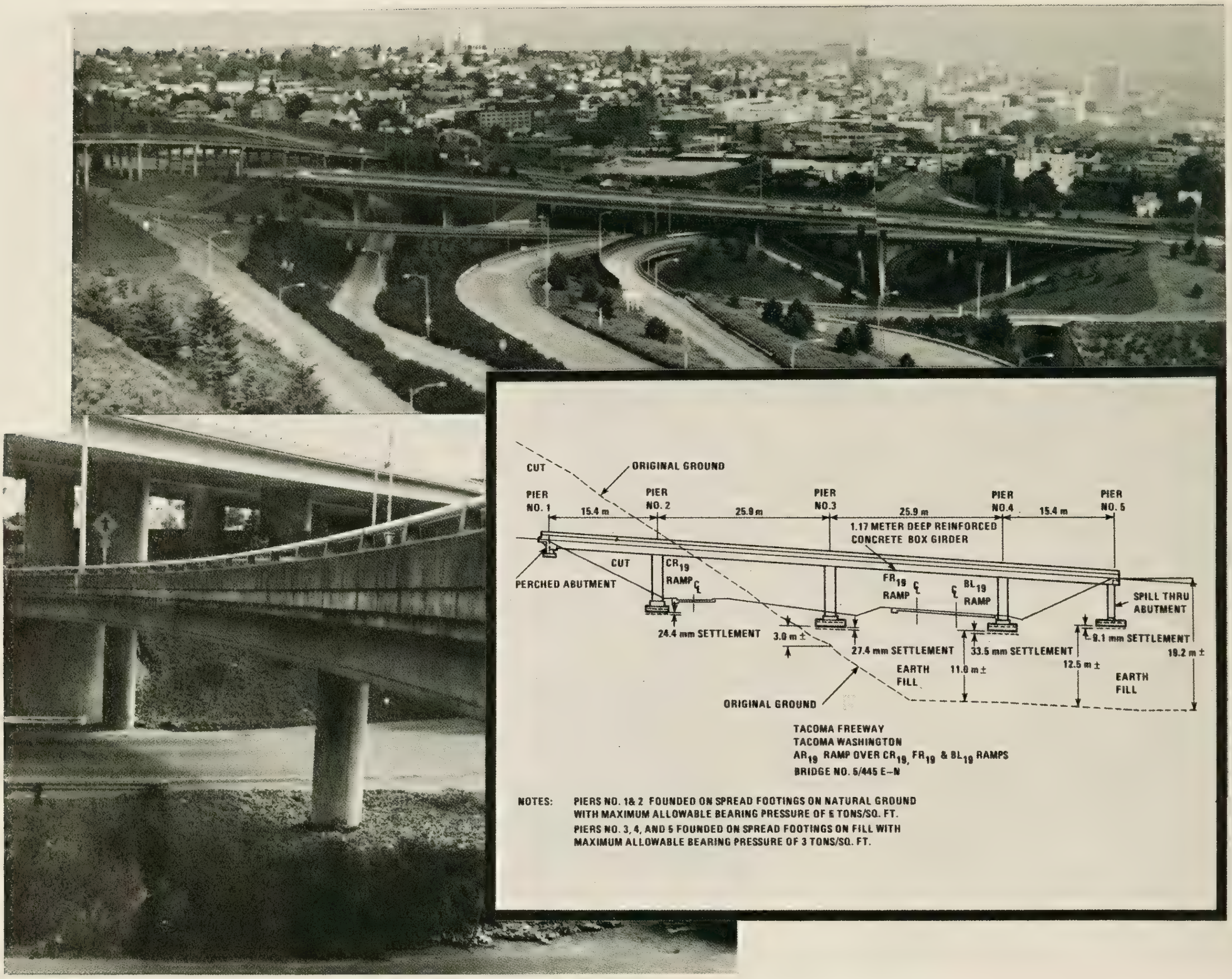

Tolerable Bridge Movements Introduction

by

Albert F. DiMillio

This article introduces a series of engineering research articles on current efforts to develop improved tolerable movement criteria for highway bridges.

This series will be published in the next three issues of *Public Roads* and will consist of the following parts:

- Part 1—Bridge Movements and Their Effects.
- Part 2—Analytical Studies of the Effects of Movements on Steel and Concrete Bridges.
- Part 3—Tolerable Movement Criteria for Highway Bridges.

Foundation movements under highway bridges have been predicted and measured; however, there have been few investigations of the effect these movements have on the safety and serviceability of the bridge structure. As a result, tolerable foundation movement has not been established beyond the conceptual stage.

The majority of highway bridge foundations are designed to prevent settlement. Many highway engineers believe such movement will cause significant superstructure distress and poor vertical alinement. Consequently, most bridges are supported by piles because they provide greater protection against settlement than do spread footings.

In many cases, a spread footing can adequately provide bearing capacity support and reasonable protection against excessive settlements at a much lower cost. When deciding whether to use piles or spread footings, the highway engineer must have an agreed upon definition of "excessive settlement" or more appropriately "tolerable movement" of various bridge types.

In addition to the disagreement on the definition of tolerable settlement, there is also disagreement on the accuracy of settlement predictions. As a result, many engineers still prescribe strict settlement constraints to guard against erroneous predictions. In many cases, the precautions are unrealistic and underestimate the accuracy of the prediction techniques, which are accurate within 10 to 20 percent on the average and are rarely more than 50 percent off the measured value. This series of articles will discuss only the tolerable movement aspects of highway bridge engineering.

Using piles does not guarantee that there will be little or no settlement, unless the piles are founded on hard rock. The nature of load transfer from the pile to the soil surrounding (side friction) and beneath the pile (tip bearing) requires the movement of the pile to mobilize the load bearing capacity of the soil material. The relative movement required to mobilize maximum frictional strength is approximately 6.35 mm (0.25 in), while the displacement required to mobilize the soil's shear strength under the pile tip is usually much larger than that value. If the allowable settlement is restricted to a smaller value than what is required to mobilize tip resistance, the load carrying capacity of the pile is artificially reduced to the level of side friction support. The support capacity available at the pile tip then becomes an additional safety factor.

Overly restrictive settlement constraints can reduce the supporting capacity of friction piles significantly. To comply with the prevalent requirements of zero or near zero settlement, the foundation engineer must reinforce his or her design by adding more piles, using longer piles, or increasing the diameter of the piles. The conservative effect of this design philosophy is evident by examining a load-settlement curve from a pile load test (fig. 1). A slight increase in the allowable settlement

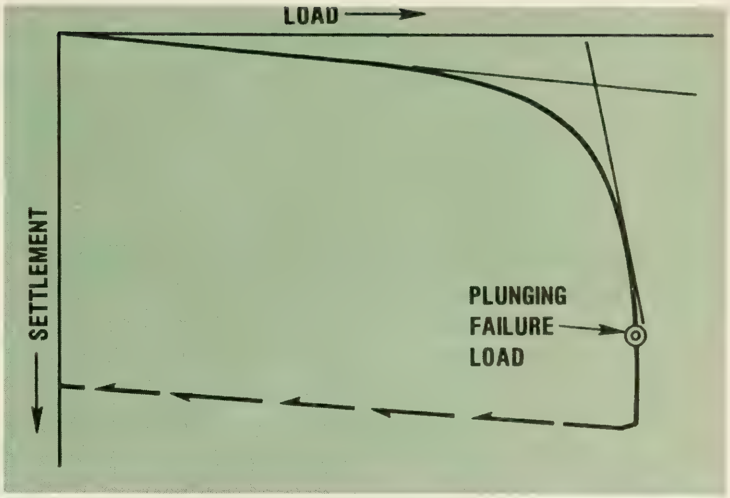

Figure 1.—Typical load-settlement curve from a pile-load test.

value can impact significantly the allowable design load for a pile foundation, especially within the straight line portion of the curve.

A relaxation of the stringent movement criteria also will impact significantly the use of spread footings whose major drawback is the risk of some settlement. Because the economic breakpoint in the normal design procedure for spread footings centers around 25.4 mm (1 in) of settlement, the majority of bridge engineers are reluctant to use this less expensive method of foundation support. As a result, the highway industry has been accused of having numerous "buried treasures of money" beneath bridge approach fills and intermediate piers because in some cases piles were used instead of spread footings or because more piles were used than were necessary. (*1*)[1]

Structural Consequences of Foundation Movement

Research indicates that bridge structures can withstand a reasonable amount of settlement and the amount of tolerable settlement varies according to the span arrangement (simple versus continuous) and length of span as well as other design variables. This indicates that the blanket criterion approach is inappropriate and the zero settlement design approach is too conservative. (*1–3*)

[1] Italic numbers in parentheses identify references on page 29.

An informal survey of tolerable movements was conducted to gather case histories that provide measured movement data and corresponding damage assessments. The information was enlightening but insufficient to establish rational tolerable movement guidelines. (*1*, *2*)

Federal Highway Administration (FHWA) researchers studied the survey data and concluded that rational guidelines for tolerable movement could be developed through structural analysis if additional supporting field data could be obtained. A study was initiated in June 1978 with West Virginia University (WVU) to perform structural analysis studies and evaluate and expand the survey data. Functional distress and time rate effects also were assessed in the study.[2]

Foundation movements that induce dangerously high stress levels in structural elements are uncommon because the movements usually become intolerable for other reasons (such as poor rideability or esthetically displeasing) before the structural integrity of the bridge is threatened. If the movements occur slowly, the functional distress patterns usually will signal the need for corrective maintenance long before the danger point (structural collapse) is reached. To determine the various danger points WVU researchers analyzed various bridge types subjected to specific differential settlements at certain supports. Various span arrangements, span lengths, and girder depths were studied in both steel and concrete bridges.

During the same period, FHWA researchers studied the performance of approximately 150 highway bridges in Washington State that were supported by spread footings on compacted fills to determine if any damage had occurred as a result of settlement or other factors associated with the use of spread footings. In cooperation with the Washington State Highway Commission (WSHC), a circuit level survey was conducted on 28 of these bridges to obtain current elevations that could be compared with the as-built elevations to determine more precise movement data. The comprehensive case histories for the 28 bridges supported the survey data.

The FHWA researchers concluded that spread footings are a safe, reliable, and cost-effective alternative to pile foundations for the support of bridge piers and abutments on natural ground and engineered fills. Under certain conditions, such as potential scour at water crossings or when very difficult subsoil conditions are present, the cost differential between spread footings and piles decreases and the failure risk increases. Piles will always be in demand by the highway industry; however, resource economy requires the prudent use of spread footings where conditions permit.

All of the bridges inspected in the WSHC study performed well and in some instances caused fewer problems than nearby bridges on piles that had undergone differential settlement between the approach fill and the pile-supported abutment. The potential for a noticeable "bump" at the bridge end is much higher if the independently supported abutment does not settle with the approach fill, as it does with the spread footing that rests on top of the fill. Many of the bridges surveyed settled significantly without showing any signs of structural distress or deterioration of ride quality.

Although none of the bridges supported on spread footings on compacted fills experienced any distress or failures because of scour, one abutment was seriously damaged because of scour. The spread footing was supported on natural ground that gradually eroded because the soil was not adequately protected by riprap or other suitable measures.

[2] "Tolerable Movement Criteria for Highway Bridges," by West Virginia University. Not yet published.

Functional distress is more difficult to assess than structural distress because of its subjectivity. Functional distress is defined under this study as damage to the architectural elements or a reduction in ride quality. Architectural damage is less severe than structural damage that affects the integrity of a main supporting element of the bridge; however, architectural damage is usually more visible and causes an annoying or insecure feeling on the part of the motorist. It is also referred to as "cosmetic damage" and includes cracking or misalinement of bridge railings, curbs, decks, abutment wing walls, and damage to light poles and utility lines. The deterioration of ride quality involves the "bump" at the end of the bridge and other roadway unevenness associated with bridge foundation settlement.

The WVU researchers conducted structural analysis studies of the time-rate effects of foundation settlement to determine if certain bridge structures adjust more readily to a different shape if the settlement occurs slowly. For example, mitigating factors such as plastic flow in steel and long term creep of concrete were investigated.

Building codes in many major U.S. cities permit larger settlements than the American Association of State Highway and Transportation Officials (AASHTO) bridge specifications, even though building elements, such as glass doors and windows, elevator shafts, utility lines, and brittle wall panels, are more sensitive to foundation settlement than bridge elements. WVU researchers reviewed the research that resulted in improved building codes.

The proper definition of tolerable foundation movement will improve bridge design so that proper trade offs between structural design and foundation design can be made to insure that structural design is not optimized at the expense of the foundation system. The necessary guidelines for tolerable foundation movement were developed by WVU researchers in terms of settlement-induced stresses resulting from a predetermined amount of differential settlement acting on a particular superstructure configuration of a given stiffness. The design procedures package will be submitted to AASHTO for consideration and adoption.

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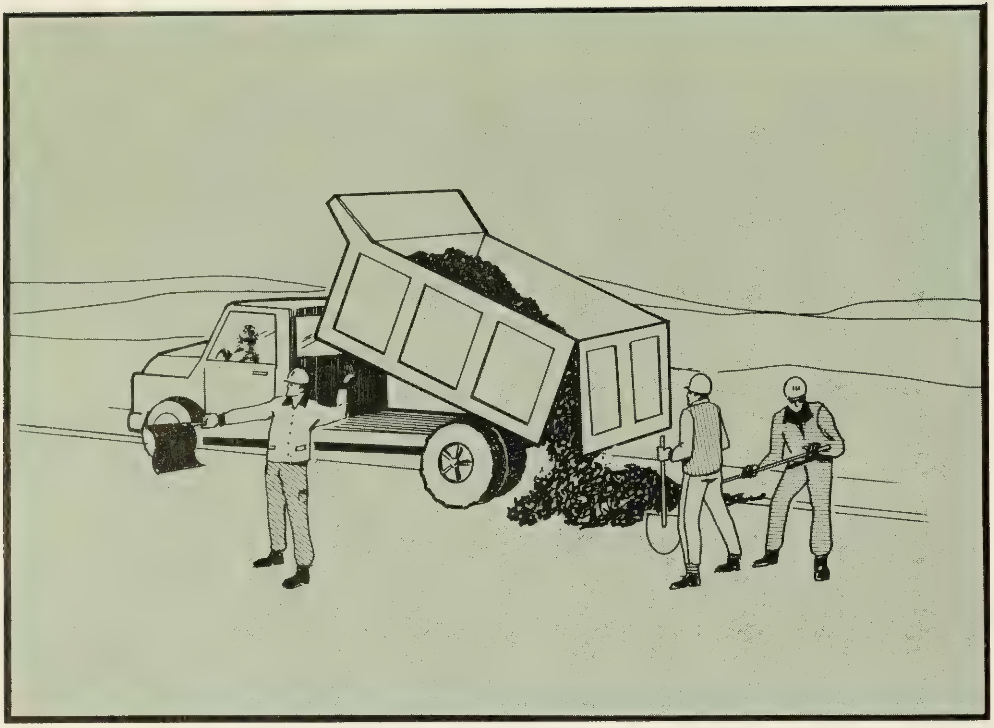

Optimizing the Expenditure of Maintenance Resources

by
Charles W. Niessner

Introduction

The cost of maintaining the Nation's highways is increasing by $300 million a year. The Federal Highway Administration (FHWA) has estimated that in 1979 State highway agencies spent $4.3 billion for highway maintenance.

In 1975 the Transportation Research Board under contract with FHWA recommended a program of high priority research and development maintenance needs. Twenty-eight needs were identified. The need having the highest priority was "Optimizing the Expenditure of Maintenance Resources." *(1)*[1]

The Implementation Division of the Office of Development, FHWA, initiated a series of studies to develop a methodology for meeting this need. Selected maintenance activities were analyzed using the techniques of value engineering (VE). Before 1975, VE was not widely applied in the highway field; only one State highway agency had a full-time VE staff. In 1975 FHWA awarded a contract to a private firm to conduct a national VE training program. During the training program, which is sponsored by the Office of Engineering and the National Highway Institute, FHWA, students work through the VE process by applying it to a real problem.

This article summarizes the Implementation Division's program to use VE techniques for maintenance activities. The program consists of 11 separate studies. Individual reports documenting the findings and recommendations of each study are or will be available.

Value Engineering Process

Value engineering is the systematic application of recognized techniques that identify the function of a product or service, establish a value for that function, and reliably provide the necessary function at the lowest overall cost. *(2)*

The value engineering job plan was established to perform the VE studies and to implement the recommendations of the studies. The key factors distinguishing the job plan from other methods used to solve routine engineering problems and reduce costs are analysis of function, use of specific, creative effort to develop alternatives, and maintenance of the performance needed by the user. Table 1 lists the nine phases of the VE job plan and the techniques used for each phase. *(2)*

[1] Italic numbers in parentheses identify references on page 34.

Table 1.—Value engineering job plan (2)

Phases	Techniques
Selection	Solicit project ideas Identify high-cost/low-value areas Plan the project Allocate resources
Investigation	Get facts from best sources Work with specifics Get all the available costs Challenge everything Identify the function
Analysis	Evaluate by comparison Put dollar value on specifications and requirements Put dollar value on key tolerances and finishes Put dollar value on key standards
Speculation	List everything Use creative techniques Deter judgment Do not criticize
Evaluation	Weigh alternatives Choose evaluation criteria Refine ideas Put dollar value on each main idea Compare
Development	Use search techniques Get information from best sources, specialists, and suppliers Consider specialty materials products and processes Use new information Consider standards Compile all costs Work with specifics Gather convincing facts
Presentation	Make recommendations Use selling techniques Be factual Be direct Give credit Provide implementation plan
Implementation	Translate plan into action Overcome problems Expedite action Monitor project
Audit	Verify accomplishments Report to management

In general, the job plan breaks down into the following elements: (1) Dividing an activity into its functions, (2) establishing costs for each function, (3) selecting the high cost functions and speculating on alternatives, (4) evaluating the most promising alternatives, and (5) developing and presenting the recommendations.

Efficient execution of the VE job plan requires a team effort—a maximum of five persons supported part-time by other staff in the organization. *(2)* Depending on the phase of the VE project, the study team size can vary from one person (presentation phase) to seven or eight persons (speculation phase).

Pilot Study

Snow and ice control was the topic selected for the pilot study to determine if the VE techniques could be successfully applied to maintenance activities. Because this topic proved too large for one study, the scope was reduced to materials handling and storage for snow and ice control. The pilot study was conducted by Colorado, Montana, Wyoming, and Utah in cooperation with FHWA.

Each State provided a two- or three-person team consisting of maintenance, research, and traffic engineers. The study lasted approximately 6 months, however, the actual time spent by the individual team members was considerably less. The study did not severely hamper the performance of the individuals' normal duties.

Three meetings were held in Denver, Colo., to organize and coordinate the study. The first session, an orientation meeting, included a discussion of VE techniques, limitations of the study, and the development of checklists for data collection. Because California was the only State highway agency with a VE staff when the pilot study was initiated, representatives of the California Department of Transportation were present at the first meeting to share their in-house VE experience. A second meeting was held about 1 month later to review the checklists and coordinate the data collection. The final meeting, a "brainstorming" session, developed recommendations. At the conclusion of the study, a summary report was prepared by the FHWA program manager. *(3)*

The study's major recommendations concerned specifications, procurement practices, processing, and optimum storage. Many maintenance stations pay excessive amounts for unnecessary, high-grade materials because small purchases of low-grade materials are not economical. The study showed that large purchases from a centralized source can economically supply stations in a 40 to 48 km (25 to 30 mile) radius.

Ineffectively and inefficiently mixing salt and abrasives in the field was found to be a high-cost/poor production practice (figs. 1 and 2). The study indicated that mixing at the plant would reduce costs by 15 percent and greatly increase the uniformity of the mix.

It was also found that more expensive bagged (rather than bulk) salt was being used for no apparent reason. Using bulk salt could save money and probably reduce back injuries.

The estimated total annual savings in the four States from the implementation of only one of the recommendations (automatic mixing) is $220,000. The study costs of $60,000 compare favorably to the estimated minimum annual savings of $5 million if automatic mixing is implemented nationwide. VE techniques were proven effective in the analysis of maintenance activities, even in States with maintenance management systems in operation. *(3)*

Figure 1.—Spreading salt to be mixed.

Figure 2.—Sand and salt being mixed with a motor grader.

Continuing Studies

Because the pilot study results show substantial savings and the recommendations are applicable to other States, the program was continued. Ten additional maintenance activities were selected. Each maintenance activity is analyzed by three to five State highway agencies, with each State providing a two- or three-person team to conduct the study. A coordination meeting is held in each of the participating States to allow each State to see firsthand the other States' operations. A summary report based on the State reports is prepared and distributed by FHWA. Table 2 lists the studies that have been undertaken and the States involved.

Each summary report contains recommendations for revising the particular maintenance activity. These recommendations include changes in material purchasing procedures, use of new equipment or modifications to existing equipment, use of alternate materials, revisions in operating procedures, and training.

The following are a few of the more significant findings and recommendations from the various summary reports:

• In writing chemical compound specifications for traffic paints, the chemical components should be reviewed annually to determine the most cost-effective composition. The prices for chemical compounds are

Table 2.—Maintenance activities

Maintenance activity	States
Snow and ice control—materials handling and storage (*3*)	Colorado Montana Wyoming Utah
Snow and ice control—operations (*4*)	Colorado Utah Pennsylvania California
Shoulder maintenance (*5*)	Idaho Arizona West Virginia Iowa
Repair of continuously reinforced concrete pavement (*6*)	Texas Arkansas Louisiana Mississippi
Bituminous patching (*7*)	Oregon Utah Arkansas Pennsylvania
Sign maintenance (*8*)	Florida Arkansas Kentucky
Bridge painting (*9*)	New Jersey New York Wisconsin
Traffic striping (*10*)	Florida North Carolina Illinois New Mexico
Pavement joint repair (*11*)	Iowa Kansas Nebraska South Dakota
Rest area maintenance (*12*)	Alabama Michigan Illinois Virginia Wisconsin
Mowing operations (report to be published)	Alabama Florida Georgia

constantly fluctuating and in some cases it is possible to make a substitution without sacrificing performance or color. During the course of the study on traffic striping, one State revised its specifications and saved $211,000 when buying 1 155 m^3 (305,000 gal) of paint.

- In rest area maintenance, labor accounts for 60 to 80 percent of the maintenance cost. Therefore, any reduction in employee-hours will reduce the overall cost. A review of current staffing plans resulted in the development of several alternatives that would accomplish the same level of service. Statewide annual savings from implementing one of these alternatives ranged from $247,000 to $395,000.

- The study on bridge painting found that the establishment of a flexible painting program that includes a general repainting cycle and an inspection program to identify precise repainting schedules for each structure could result in substantial savings. The establishment of this type of program can reduce or eliminate the need for extensive preparation of the steel prior to painting. If complete sandblasting of the steel is required, the cleaning costs can be as high as 60 percent of the total cleaning-painting costs.

- The use of ground control spreaders can save time, material, and fuel at a relatively low cost for the initial investment. Many existing hydraulic spreaders can be adapted to ground control systems for less than $1,500. Controlled tests have shown that these units can save up to 40 percent of the material that would normally be spread by nonground-oriented spreaders.

Although some of the findings apply only to those States that participated in the study, many of the changes can be applied in several States.

Summary

The total estimated cost for the VE studies was $500,000. The total estimated annual savings for only those States that participated in the various studies exceeds $8 million. Reducing maintenance costs will improve service because more of the activity or other activities can be undertaken.

Thirty-six training workshops have been conducted for both FHWA and State personnel. These workshops and the maintenance studies have induced several other States to initiate their own VE studies. Several State highway agencies have also established a full-time VE staff.

This program has demonstrated that VE techniques can be successfully applied to maintenance activities. Also, the program gave State maintenance engineers the opportunity to meet and discuss mutual problems and share information with personnel from other States. These meetings resulted in potential solutions for several problems not directly related to the study such as snow removal equipment and policies, mowing policies, and signing and traffic control. Other special purpose maintenance items were observed such as a small trailer for carrying warning signs and traffic cones to a worksite and an air deflector mounted horizontally above the tailgate of a sand spreader truck to prevent snow from accumulating on the taillights.

It is essential that every effort be made to optimize the expenditure of maintenance dollars. It is imperative that, where applicable, each State highway agency implement recommendations from other studies and analyze in depth its high-cost maintenance activities.

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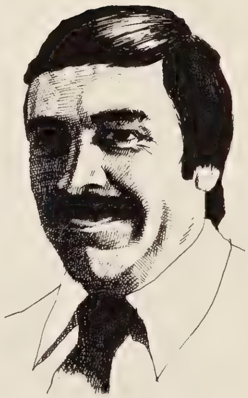

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Capacity and Quality of Flow on Urban Arterials

by
Guido Radelat and Gary Euler

This article describes a proposed process for determining the capacity and quality of flow on urban arterials. Many of the ideas presented here originated in the first phase of a study on the quality of flow in urban arterials, sponsored by the Federal Highway Administration (FHWA).

This article discusses arterial capacity and the influence of capacity controlling features that are used to define a section of arterial capacity. This sectional concept is proposed as an extension of the cross sectional or point measure currently used in the Highway Capacity Manual. (*1*)[1] A procedure for computing arterial capacity is also described.

The designation of levels of service for an urban arterial as a function of newly selected measures of effectiveness (MOE) is proposed. The correlations among these MOE's are discussed as well as their implications on the definition of levels of service. Estimating MOE's from independent arterial characteristics is described and the effect of one of these characteristics, signal progression, on the arterial MOE's is analyzed.

The article concludes with a discussion on updating the Highway Capacity Manual.

Introduction

An arterial street's ability to function as a traffic facility can be described by conventional measures of effectiveness (MOE). MOE's such as traffic volume, spot speed, and intersection delay apply to a *point* on the arterial; others, such as travel in vehicle-kilometres (vehicle-miles), travel time, and total delay, are related to a *section* of the arterial. The section MOE's determine the performance of a section of arterial whereas the point MOE's localize possible deficiencies.

[1] Italic numbers in parentheses identify references on page 43.

On arterials and other traffic facilities, both the quality and the quantity of traffic flow must be determined. The quality of traffic flow is determined by MOE's such as spot speed, intersection delay, travel time, total delay, travel time variance, number of stops, number of accidents, vehicle emissions, and fuel consumption per vehicle. The quantity of flow, on the other hand, is determined by MOE's such as traffic volume and vehicle-kilometres (vehicle-miles) of travel.

The following example illustrates the importance of determining the quantity as well as the quality of arterial flow. Suppose that a new traffic management strategy improves the quality of flow of an arterial section but does not change the quantity of flow. Obviously, the traffic performance of the arterial has been improved. If, however, the improved traffic conditions attract new traffic to the arterial, the quality of flow may be degraded to its previous level while the quantity of flow increases significantly. However, traffic performance on the arterial still has been improved, because more traffic is now being served with the same quality of flow as before.

Some of the quality of flow MOE's are related to the speed of the traffic flow: Spot speed, intersection delay, travel time, and total delay. Other quality of flow MOE's, such as travel time variance and the number of stops, indicate the flow's smoothness or jerkiness. The number of accidents MOE demonstrates the safety of traffic flow. Other MOE's—vehicle emissions and fuel consumption—relate traffic flow to social goals such as reducing air pollution and conserving resources.

One problem inherent in using arterial MOE's is accounting for trade offs among the MOE's. A potential solution is to combine the MOE's into a composite MOE. This is difficult, however, because individual MOE's are expressed in different units that cannot be converted easily into a single unit. Also, individuals may perceive the importance of a given MOE differently, which makes it difficult to rate individual MOE's when calculating the composite. For example, a driver is not directly interested in quantity of flow MOE's, but is primarily concerned with the speed MOE's and the jerkiness of the flow, which increase or decrease the driving effort. On the other hand, quantity of flow MOE's are important to traffic engineers and other public officials. For people living or working near a traffic facility, vehicle emissions and noise may be the most important MOE's. Finally, fuel consumption has become an important MOE for everyone because of the energy crisis.

An important feature of a traffic facility is its capacity—the maximum number of vehicles the facility can handle. Capacity is related to the quantity of flow MOE's and is often the only measure used to indicate a traffic facility's effectiveness. Another important feature of a traffic facility is the level of service (LOS)—the quality of service a facility can provide under specific conditions for a certain quantity of flow.

Capacity

The 1965 Highway Capacity Manual (HCM) defines capacity as follows:

> Capacity is the maximum number of vehicles which has a reasonable expectation of passing a given section of a lane or roadway in one direction (or in both directions for a two-lane or three-lane highway) during a given period of time under prevailing roadway and traffic conditions. (*1*)

In this definition, capacity is a unique volume—the maximum number of vehicles under prevailing roadway and traffic conditions at a given section of a lane or a roadway; thus it is a cross sectional measure, not a longitudinal one. Capacity corresponds to a point on the traffic facility under observation, and therefore, is defined in the HCM as a point MOE (fig. 1).

Figure 1.—1965 HCM concept of capacity focuses on the capacity of a point or single intersection.

When analyzing an arterial, however, the primary question is the flow along the arterial (fig. 2). The number of vehicles that can pass through an arterial section is more important than the number that can pass by a given point. In the first phase of a recently completed study on the quality of flow on urban arterials, the point concept of capacity was extended to apply to a section of an arterial. (2) The proposed definition of capacity follows:

Arterial capacity is the maximum number of vehicle-miles of travel which can reasonably be expected to be accommodated by a given arterial segment (or section) per unit time and length under prevailing roadway and traffic conditions. This capacity can be expressed in either one-way or two-way flows. In the absence of time and distance modifiers it would be expressed in vehicle-miles of travel per hour per mile. (2)

In this definition, capacity is expressed in vehicles per unit of time (travel units divided by the length of the section under observation). Capacity is expressed in the same units in the HCM definition; however, the units of the HCM definition refer to a point, whereas the units of the proposed definition apply to an arterial section and constitute an average volume over the section.

To compute arterial capacity, a segment is defined as the zone of influence of a bottleneck or capacity controlling feature, and a section is a series of continuous segments. A capacity controlling feature is an element of the arterial whose point capacity is considerably lower than the point capacities along the portion of arterial upstream of it. It is generally an intersection, but occasionally may be a midblock point with a capacity lower than that of an intersection downstream.

In the study on quality of flow on urban arterials, segment capacity is defined as the maximum segment volume multiplied by the length of a segment, that is, the point capacity of the downstream capacity controlling feature times the length of the segment. (2) Segment capacity is expressed in vehicle-kilometres (vehicle-miles).

The segment capacities of a longitudinal section of arterial with six segments are shown in figure 3. The area under the segment capacity profile line—determined by the point capacities of the capacity controlling features—represents the segment capacity.

The capacity of a section is also determined by the arterial volume profile—the profile of current traffic volumes at points along the arterial section (fig. 3). For each segment, the shaded area under this line is the travel in the segment under prevailing conditions. For the conditions shown, the arterial section is not operating at capacity.

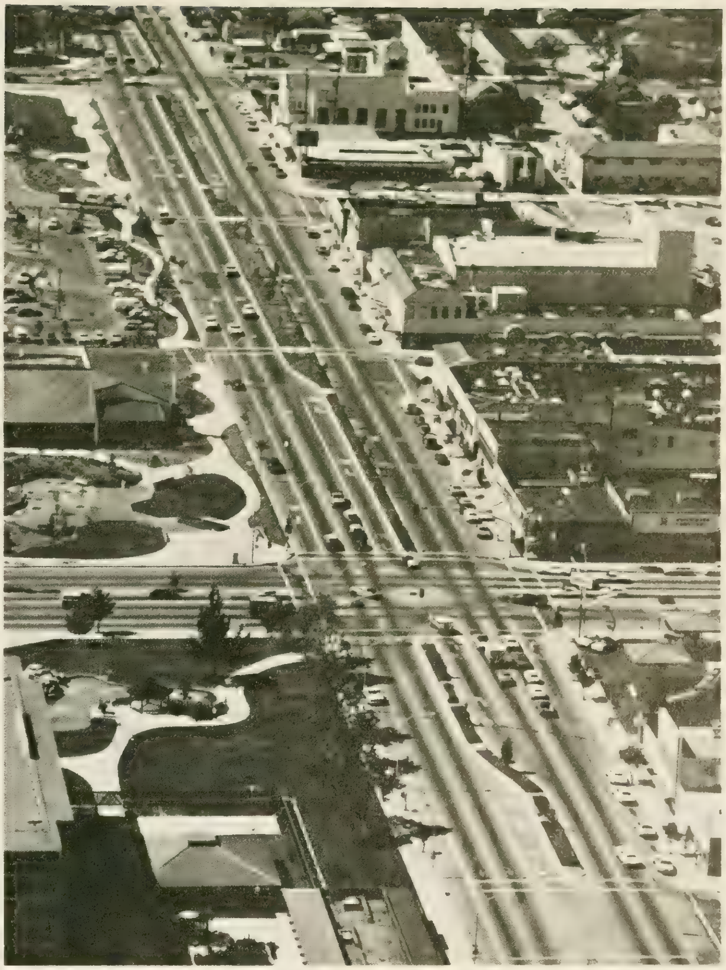
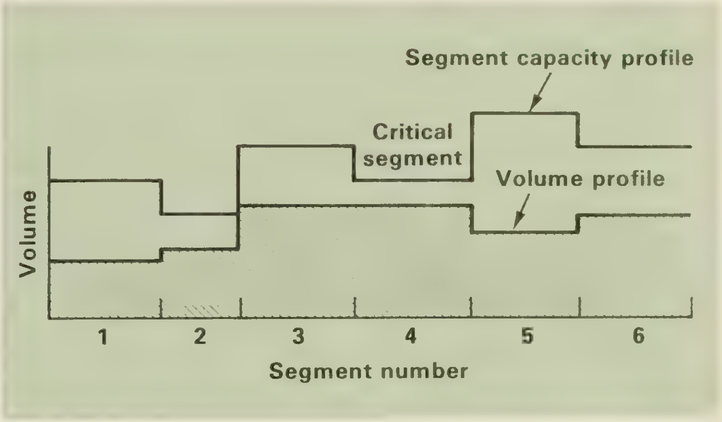


Figure 2.—Proposed concept of arterial capacity focuses on the flow along an entire length of an arterial.

Figure 3.—Segment capacity profile.



To compute arterial capacity, the travel/capacity ratio must be known for each segment in the section being observed. This ratio is found by dividing the segment travel by the segment capacity (both quantities expressed in either vehicle-kilometres or vehicle-miles). The segment with the highest travel/capacity ratio is then identified as the "critical segment" (segment 4 in figure 3). The arterial capacity of the section is found by determining the total travel that will result from expanding travel in the critical segment to its capacity with the same proportional expansion of travel in the other segments; that is, travel is expanded, keeping the same travel ratios among the segments. Thus prevailing road and traffic conditions are maintained. This expansion is shown in figure 4. The volume profile resulting from this expansion—the arterial capacity profile—is also shown in figure 4.

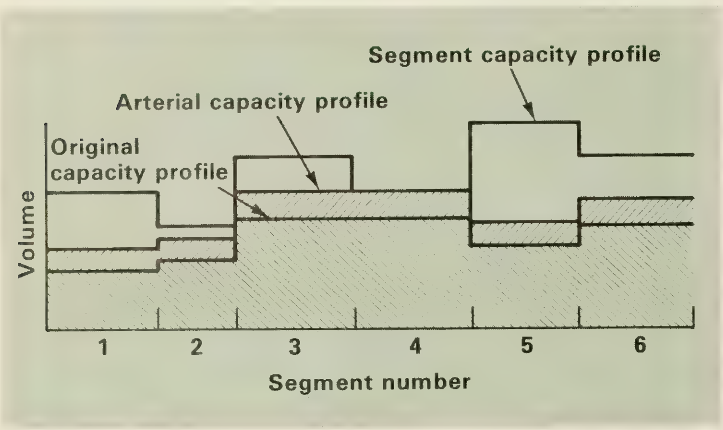


Figure 4.—Arterial capacity profile.

The procedure for computing arterial capacity is as follows:

1. Determine the capacity controlling features along the arterial section being observed.

2. Measure traffic volumes and compute point capacities at these locations in vehicles per hour.

3. Compute travel and segment capacity for each segment, multiplying the volume and point capacity at the downstream capacity controlling feature by the segment length (results in vehicle-kilometres [vehicle-miles] per hour).

4. Compute travel/capacity ratios for each segment and identify the segment with the highest ratio—the critical segment.

5. Expand travel in the critical segment to its capacity and expand travel in other segments by dividing their travel by the travel/capacity ratio of the critical segment (results in vehicle-kilometres [vehicle-miles] per hour).

6. Find the arterial capacity of the section by adding the expanded segment travel figures and dividing the sum by the length of the section (results in vehicles per hour).

This definition of arterial capacity assumes that the turning movement percentages remain constant. This is not always true, but it is a reasonable assumption. There still may be unused segment capacity when an arterial section is operating at capacity. The above procedure will identify the wasted capacity and may suggest changes in vehicle routing to better utilize segment capacity.

Levels of Service

Capacity is essentially a quantity of flow MOE indicating the maximum quantity of flow that a traffic facility can handle. It can also express indirectly the quality of flow because for the same traffic demand, the higher the capacity of the facility, the better the service that can be expected.

However, it is important to know the quality of traffic flow, which can be determined by several MOE's, when a facility is operating either at or below capacity.

The 1965 HCM uses levels of service (LOS) from A through F to designate the quality of flow and defines LOS as follows:

> Level of service is a term which, broadly interpreted, denotes any one of the infinite numbers of differing combinations of operating conditions that may occur on a given lane or roadway when it is accommodating various traffic volumes. Level of service is a qualitative measure of the effect of a number of factors, which include speed and travel time, traffic interruptions, freedom to maneuver, safety, driving comfort and convenience, and operating costs. In practice, selected specific levels are defined in terms of particular limiting values of certain of these factors. (*1*)

LOS must be defined in terms of MOE's. The HCM uses three MOE's to determine the LOS of urban and suburban arterials: Average overall travel speed, load factor, and service volume/capacity (V/C) ratio.

The average overall travel speed is a good measure of arterial performance because it reflects the influence of local bottlenecks and the effectiveness of signal coordination. If measured for each segment of an arterial section, it can show where deficiencies exist.

The load factor determines the LOS of individual intersection approaches. Load factor is the ratio of the number of loaded green phases on an approach during a certain period of time to the total number of green phases available on that approach during that time. A phase is loaded when vehicles are ready to enter the intersection in all lanes during the entire phase without "exceedingly long spaces between the vehicles at any time due to lack of traffic." (*1*)

The maximum possible value of the load factor is 1. However, an intersection approach can offer various levels of service while the load factor is at its maximum value. Also, the load factor loses its meaning as a measure of flow quality when an intersection is controlled by traffic-actuated signals or by no signal at all. Furthermore, the load factor cannot assess the quality of flow at midblock locations.

The V/C ratio is also used in the HCM to define the LOS at individual intersection approaches. It is actually a quantity of flow MOE that indicates how loaded a facility is. It can only be used to determine the LOS if a relationship is developed deductively between the V/C ratio and LOS or between the V/C ratio and a quality of flow MOE such as speed. However, it is not feasible to show any typical speed-volume curve for urban arterials. (*1*) An attempt to determine such relationships was unsuccessful because of the effects of other variables such as signal timing, turning percentages, and interference from parking maneuvers. Therefore, the V/C ratio does not seem to be an adequate MOE for LOS.

The limitations of the MOE's used in the HCM became evident during the study on quality of arterial flow. (*2*) This study included a literature review to identify all MOE's suitable for defining LOS. These MOE's were systematically analyzed and the following list of quality of flow MOE's was selected:

- *Travel time per kilometre (mile).* This speed MOE is equivalent to the average overall travel used in the HCM and reflects the overall performance of the entire arterial section under observation or each of its segments.
- *Intersection delay.* This point MOE measures the effect of individual intersections on the quality of flow and is a hybrid between a speed and a smoothness of flow MOE.
- *Intersection stops.* This point MOE indicates the influence intersections have on the smoothness of flow.
- *Travel time variance.* This segment MOE measures the overall smoothness of flow along an arterial section.

In addition to the quality of flow MOE's chosen for estimating the LOS, service volume, a quantity of flow MOE directly related to capacity, was selected. This MOE can be either a point or segment MOE and is expressed in vehicles per hour, vehicles per lane per hour, or vehicle-kilometres (vehicle-miles) per hour. As applied to a section, service volume is not really a measure of volume but of travel.

In the opinion of the authors, four MOE's are still too many for defining LOS. As discussed earlier, the difficulties of combining them are considerable. Also manipulating four LOS MOE's would be unduly complex and cumbersome for practical application. Furthermore, there is a significant correlation among the quality of flow MOE's; therefore, once the key MOE's have been identified, little meaningful information can be obtained from the others.

Several years ago, over 4,000 floating car runs were made on forty 1.6 km (1 mile) arterial sections in Washington, D.C.[2] Travel time, stopped delay at intersections, and number of stops over each arterial section were among the variables observed. The authors recently performed a regression analysis on the data collected for each MOE, and the following coefficients of linear correlation were obtained:

Number of stops versus intersection stopped delay	0.86
Travel time versus intersection stopped delay	0.84
Travel time versus number of stops	0.80

Similar conclusions were drawn in a study of the influence of various urban driving conditions on automobile fuel consumption. (*3*) A test vehicle was driven 383 km (238 miles) through nine routes in the Detroit, Mich., metropolitan area. Fuel consumption, speed, acceleration, and other travel-related variables were recorded. The statistical analysis of the data collected produced, along with many other results, the following coefficients of linear correlation:

Number of stops versus intersection stopped delay	0.79
Travel time versus intersection stopped delay	0.86
Travel time versus number of stops	0.85

In a study to develop a technique for measuring delay at intersections, field data were collected photographically at 10 intersections in four metropolitan areas. (*4*) An analysis of the correlation between the percent of vehicles stopping at an intersection and the stopped time per vehicle yielded a coefficient of linear correlation of 0.72. However, in a regression analysis of the percent of vehicles stopping and the logarithm of the approach delay per vehicle[3], a coefficient of linear correlation of 0.94 was found.

[2] "A Method for Evaluating the Efficiency of Traffic Operations in a Signalized System," by G. Radelat, J. Raus, and F. A. Wagner. Paper presented to Committee TO-7 of the Transportation Research Board, January 1966.

[3] Time spent by the vehicle in traversing the intersection approach minus free flow time.

In summary, the LOS of an arterial might possibly be defined as a function of a section MOE that provides a measurement of speed, such as travel time, and a point MOE indicating the smoothness of flow, such as intersection stops or intersection stopped delay. Furthermore, the possibility that only one MOE may be used to define LOS is not discarded.

Estimating MOE's

Direct measurement of arterial MOE's is generally costly and time consuming. Also, in planning and design work MOE's are not measurable because the desired conditions do not exist in the field. Nevertheless, relationships exist between the MOE's and road, traffic, and environmental characteristics that may be measured or predicted. Knowledge of these relationships would make it possible to estimate the MOE's and then compute point capacity and quality of flow as a function of the MOE's. Also, the effect of variations in selected characteristics on the MOE's could be estimated. The HCM is used to perform this type of estimation based on relationships between two or more variables developed from empirical methods.

Independent characteristics

In the study on quality of arterial flow, independent road, traffic, and environmental characteristics related to either capacity or quality of flow were analyzed. (2) The study research team then identified which of these characteristics influence the point MOE's associated with an intersection and which influence arterial segment MOE's (table 1). A subset of these characteristics will be used in the proposed HCM to estimate the selected MOE's, which in turn will be used to predict point capacity and quality of flow along an arterial.

Procedures to relate characteristics to MOE's

The conventional procedure to develop relationships between independent road, traffic, and environmental characteristics and the MOE's for a traffic facility is to observe simultaneously these variables in the field and then perform a statistical analysis on the observations. It is possible, however, to augment the field data from this empirical procedure with information generated from "logical" relationships between the variables. For example, the position, speed, and acceleration of a vehicle following another one is related to the position, speed, and acceleration of the vehicle being followed; also, a left-turning vehicle is not delayed if no opposing vehicle approaches during a time equal to or greater than the acceptable gap. These logical relationships are inherent in many analytical and simulation models; therefore, these models can supplement empirical information.

Table 1.— Independent road, traffic, and environmental characteristics related to capacity or quality of flow (2)

Related to Intersection Point MOE's	
1. Approach width	11. Progression
2. Number of lanes	12. Total green per approach
3. Lane control	13. Number of times green
4. Median type	14. Volume per movement
5. Type of bus stop	15. Truck volume
6. Right turn on red	16. Local buses
7. Parking condition	17. Bus dwell time
8. One/two way	18. Lane blockage
9. Parking turnover	19. Pedestrian levels
10. Distance to first parked vehicle	
Related to Arterial Segment MOE's	
1. Width	8. Parking turnover
2. Number of lanes	9. Segment demand
3. Median type	10. Truck volume
4. Length	11. Local buses
5. Bus stop type	12. Bus dwell time
6. Parking condition	13. Lane blockage
7. One/two way	

Traffic simulation models, such as NETSIM (5), are useful in analyzing intricate urban arterial situations because they combine the empirical and logical elementary relationships needed to define more complex relationships. Nonetheless, several statistical problems arise when combining field data with simulation data. These problems must be resolved to fully use simulation to establish the necessary relationships.

Progression

One of the independent characteristics that is difficult to measure is traffic progression along an arterial. Progression is related to traffic signal coordination and has been measured by the width of the progression band that goes through the green intervals in a time-space diagram. This measure is not very accurate because it depends on the chosen progression speed and the lengths of the straight portions of the band. Also, it does not take into account vehicle arrival distribution and the secondary flow that turns onto the arterial.

A simpler measure is the number or proportion of vehicles that arrives during the green interval. This measure has a unique value, it can be measured at each intersection, and it accounts for secondary flow from side streets. The measure loses accuracy when vehicles join a queue during the green interval because even though these vehicles are counted as arriving on green, they must actually stop before proceeding through the intersection. This would be reflected in MOE's such as stops, delay, or travel time. Previous research, however, concluded that ". . . delay is a linear function of the decimal fraction of vehicles arriving on red." (6) This conclusion, which was reached analytically, was tested by the authors using the NETSIM computer simulation model.

A hypothetical arterial having three intersections was constructed to study the effects of signal progression on two links. All approaches on the main arterial were one lane, with left turn pockets at each intersection. The selected free flow speed was 55 km/h (34 mph) and the intersections were 450 m (1,477 ft) apart. All approaches on the cross streets were also one lane. Speeds on the cross streets varied from 40 to 55 km/h (25 to 34 mph) for the three alternatives studied.

The alternatives were constructed by varying volumes on all approaches (both on the main arterial and on the cross streets) and by varying the turning percentages from the cross streets onto the main arterial. Cycle length and proportion of green times were different for each alternative, but were constant within each alternative being studied. The signal offset between adjacent intersections was then varied in 10-second increments. The timings used were not necessarily optimum, although judgment was used in their formulation insofar as splits were proportional to approach demands and larger cycle lengths were used for higher demand alternatives.

The proportion of vehicles arriving on green (PROGREEN) was calculated using NETSIM. This was then compared with the MOE's—average speed on the link, average delay per vehicle, and average number of stops per vehicle—derived for the corresponding alternatives by NETSIM. The resulting regression lines and multiple correlation coefficients are shown in figures 5, 6, and 7.

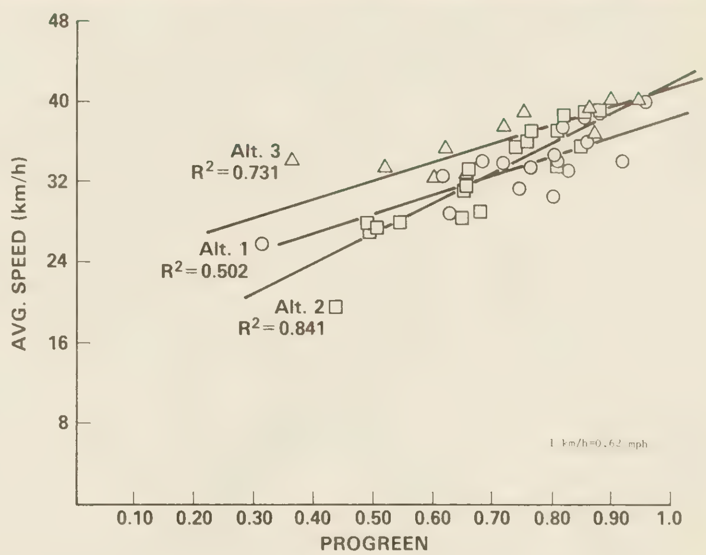


Figure 5.—Plot of link average speed versus PROGREEN.

These figures show that although a linear trend exists in all of the comparisons, a single relationship does not exist and the PROGREEN variable is not consistently correlated with any of the other MOE's. This is because of the following influences that confound the relationship between the PROGREEN variable and the traffic MOE's:

- Demand—Higher demand can produce higher delays for the same value of PROGREEN. For example, longer cycle lengths are normally used in higher demand situations. For the same values of PROGREEN, the average time that vehicles will be stopped will be longer (assuming the same split) than in a lower demand situation when a shorter cycle length is used.

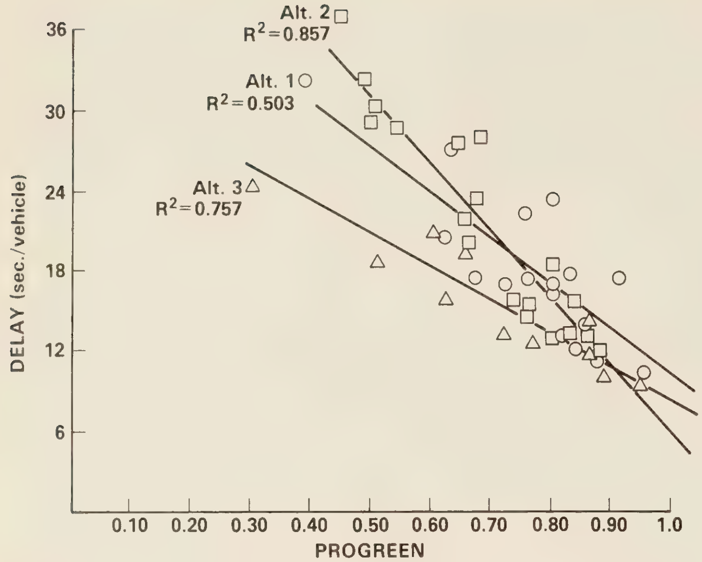
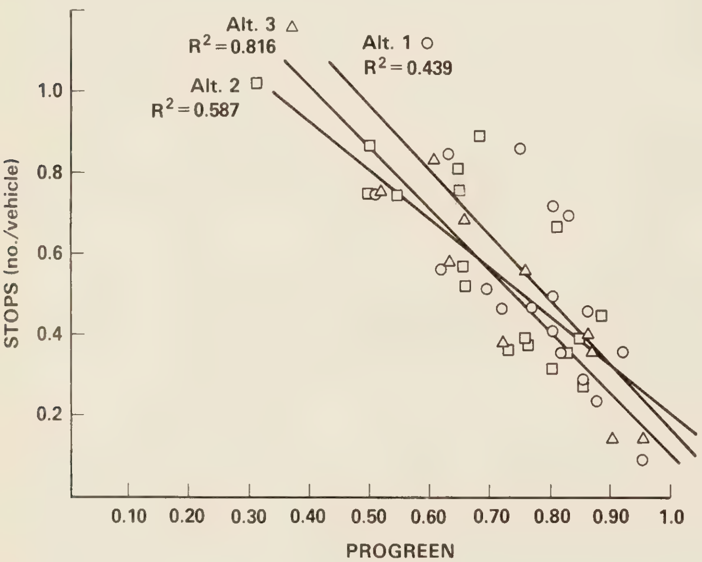


Figure 6.—Plot of average delay per vehicle versus PROGREEN.

Figure 7.—Plot of average number of stops per vehicle versus PROGREEN.



• Residual queues—A platoon of vehicles that overlaps the end of the previous red interval may produce the same value of PROGREEN as a platoon that overlaps the beginning of the next red interval. However, the average number of stops per vehicle will be higher in the former case because the vehicles that arrive on red as well as the vehicles that arrive at the beginning of the green will stop until any residual queue has dissipated. The residual queue could also be caused by cross street traffic from an upstream intersection. This may not make much difference in average delay or travel time, but it can make a big difference in the average number of stops per vehicle.

The conclusion is that the PROGREEN variable indicates the quality of signal progression, but other influences, particularly demand, must be taken into account before relationships between PROGREEN and the traffic MOE's can be established. Perhaps the best way to accomplish this is by computer simulation where an extensive, controlled study can quantify the effect of demand or other influences on the relationships between PROGREEN and the MOE's.

Finally, it should be noted that although PROGREEN indicates the quality of progression, it will not pinpoint the problem—a faulty cycle length, split, or offset—if the progression is poor.

Estimation of Capacity and Level of Service

According to the approach outlined in the study on the quality of arterial flow, an arterial section's capacity may be calculated by first measuring or estimating the section's road, traffic, and environmental characteristics that significantly affect arterial capacity. (*2*) Using these relationships, the maximum service volume or point capacity of the capacity controlling features can be computed. Segment capacity and arterial volume profiles can then be sketched and arterial capacity estimated.

A similar method can be used to calculate the quality of flow for a given service volume. If the road, traffic, and environmental characteristics and the relationships between them and the quality of flow MOE's are known, these MOE's can be computed for each segment. Then, based on the value of the MOE's, the LOS of each segment as well as that for the entire arterial section can be determined.

Many relationships between independent characteristics and MOE's have been formulated in the development of the HCM. However, more work is needed in this area to reflect changes in vehicle and driver behavior since 1964 when data for the HCM were collected, to define more precisely relationships that were developed using insufficient data, to reflect new developments in the traffic engineering field, and to respond to new concepts originated in the first phase of the study on quality of flow in urban arterials. (*2*)

Research to define the characteristic-MOE relationships will be done in the second phase of the FHWA study. A process will also be developed to express LOS in terms of the chosen MOE's. It has been recommended that a panel of experts on arterial capacity determine what that process should be.

The National Cooperative Highway Research Program (NCHRP) of the Transportation Research Board is conducting a study on urban intersection capacity that will contribute to the understanding of arterial capacity and quality of flow. By 1982 or 1983, other research planned and conducted by the FHWA and NCHRP on different aspects of highway capacity will produce the material necessary to prepare the improved, proposed Highway Capacity Manual. The new manual is expected to be a valuable guide for planners, designers, and traffic engineers for the rest of this century.

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(4) W. R. Reilly, C. C. Gardner, and J. H. Kell, "A Technique for Measurement of Delay at Intersections, Vol. I—Technical Report," Report No. FHWA-RD-76-135, *Federal Highway Administration,* Washington, D.C., September 1976. PB No. 265701.

(5) "Traffic Network Analysis With NETSIM: A User Guide," Implementation Package FHWA-IP-80-3, *Federal Highway Administration,* Washington, D.C., January 1980.

(6) P. P. Papapanou, "Analysis of Operating Parameters for Arterial Traffic Control Systems," Ph.D. dissertation, *University of Florida, Civil Engineering Department,* 1976.

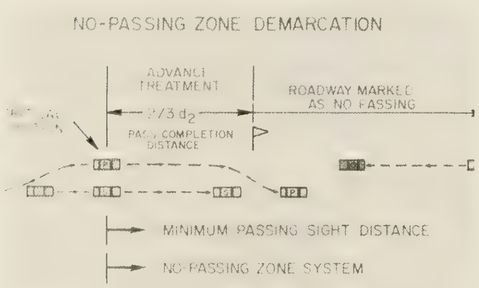
Guido Radelat is a highway research engineer in the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been with FHWA since 1964 and is currently manager for Task 2K2 of FCP Project "Metropolitan Multimodal Traffic Management," Task 2M1 of FCP Project "Arterial Flow and Control," and Task 9B2 of FCP Project "New Concepts Development and Systems Characterization." Dr. Radelat has been active in the field of traffic simulation and lately has been involved in research on highway capacity and quality of flow.

Gary Euler is a highway research engineer with the Traffic Systems Division, Office of Research, Federal Highway Administration. He has been in his current position for 3 years and has worked principally in the areas of signal control, model development, and strategy evaluation. Mr. Euler previously was employed as a planner with the Metropolitan Dade County Planning Department in Miami, Fla.

[4]Reports with PB numbers are available from the National Technical Information Service, 5285 Port Royal Rd., Springfield, Va. 22161.

Recent Research Reports You Should Know About

The following are brief descriptions of selected reports recently published by the Office of Research, Federal Highway Administration, which includes the Structures and Applied Mechanics Division, Materials Division, Traffic Systems Division, and Environmental Division. The reports are available from the address noted at the end of each description.



Passing and No-Passing Zones: Signs, Markings, and Warrants, Report No. FHWA-RD-79-5

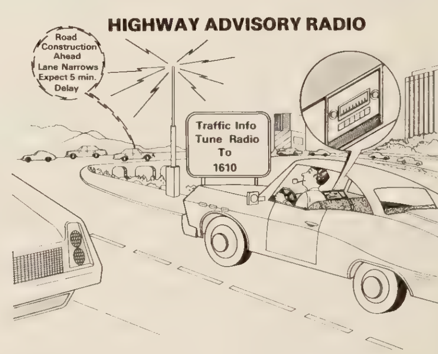
by FHWA Traffic Systems Division

This report gives suggested criteria, warrants, and traffic control devices for designing and designating passing and no-passing zones on rural two-lane highways. The criteria are based on vehicle performance during a passing maneuver, the premise that sight distance should be provided at the critical position where the passing vehicle and passed vehicle are abreast, and evaluation of traffic control devices in providing the required visual information at this critical position. Minimum passing sight distance is the sight distance required at the critical position to permit a passing driver to perceive an opposing vehicle at a sufficient distance to safely complete the pass.

An advance pavement marking system is proposed consisting of a dotted yellow centerline throughout the pass completion distance and a no-passing zone pennant sign located at the beginning of the no-passing zone. This will advise a driver that a pass should not begin beyond the start of this marking system because sufficient distance will not be available to complete the pass before reaching the no-passing zone. An economic analysis indicates a benefit/cost ratio of 6.1 if the system were to be implemented nationwide.

The use of the single solid yellow centerline on rural two-lane highways to designate "restrictive" passing zones is evaluated based on driver understanding tests, accident rate comparison, and opinions of engineering, enforcement, and legal personnel on expected safety, enforcement, and compliance problems.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 80 114564).

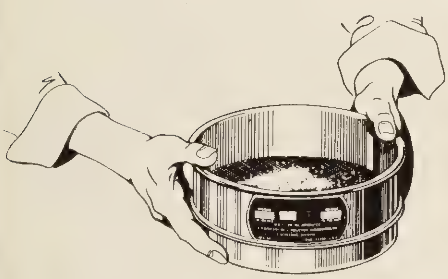


Highway Advisory Radio Operational Site Survey and Broadcast Equipment Guide, Report No. FHWA-RD-79-87

by FHWA Traffic Systems Division

This report summarizes highway advisory radio (HAR) operations activities, commercially available hardware, and licensing requirements in the United States. The report reviews HAR systems used in eight States by discussing HAR system surveillance, advance signing, and broadcasting equipment. The report also catalogs commercially available HAR transmitters, modulating sources (tape recording equipment), and antenna types. The final section of the report focuses on the current Federal Communications Commission (FCC) rulemaking concerning HAR. Included in this discussion are HAR application and authorization requirements as well as HAR frequencies, conditions, limitations, and technical standards. All current FCC rulemaking for HAR is included in the appendixes.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 301379).



Present Aggregate Gradation Control Practices and Alternative Test Methods, Report No. FHWA-RD-79-100

by FHWA Materials Division

Aggregate gradation must be controlled in current highway construction practices to insure the production of good pavements. A survey of current control practices in the United States was conducted to discover how alternative testing methods would affect the adequacy of the control and the amount of time devoted to control practices. The results of the survey are summarized in this report.

Alternatives to the standard test methods of gradation determination were compared with the standard tests in terms of total testing time required and the accuracy of the gradation results. The sources of variation in the test results and the relative importance of each were also investigated. Recommendations on the frequencies of sampling and gradation testing are given based on the study findings.

The report is available from the Materials Division, HRS-22, Federal Highway Administration, Washington, D.C. 20590.

Investigation of the Use of Coal Refuse-Fly Ash Compositions as Highway Base Course Material: State of the Art and Optimum Use Area Determinations, Report No. FHWA-RD-78-208

by FHWA Materials Division

The need to recycle waste products is becoming more crucial as the cost of their disposal escalates and as conventional materials become scarce. Two such byproducts of the coal industry, coal mine refuse and fly ash, show promise as construction materials.

This report outlines the phase I findings of a study on the use of coal refuse-fly ash compositions as highway base course material. This portion of the investigation involved a comprehensive literature search into the engineering properties and field use of the two waste materials. An annotated bibliography was assembled and the 10 areas were identified where the use of such compositions would be optimal based on economic, environmental, and material availability considerations.

The succeeding two phases of the investigation will study the development of an optimum coal refuse-fly ash blend for each of the 10 optimum use areas based on standard laboratory tests and comparisons with conventional materials.

The report is available from the Materials Division, HRS-21, Federal Highway Administration, Washington, D.C. 20590.

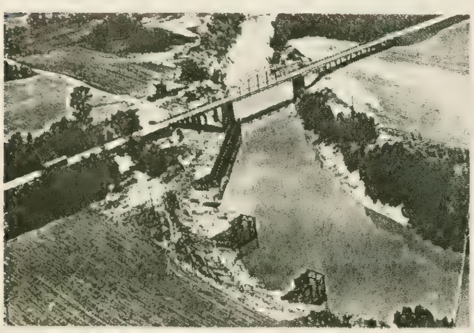
Provisions for Elderly and Handicapped Pedestrians: Volume 1 (Report No. FHWA-RD-79-1), Volume 2 (Report No. FHWA-RD-79-2), and Volume 3 (Report No. FHWA-RD-79-3)

by FHWA Environmental Division

The problems and hazards experienced by elderly and handicapped pedestrians were investigated and the results, expressed in typological form, were prioritized. These reports present research on the pedestrian environment and discuss some of the high priority environmental problems and proposals for improving accessibility. Curb ramp criteria and tactile surface materials were evaluated and the results incorporated into countermeasures that were field tested. The priority accessible network as a conceptual approach to barrier-free urban areas has been developed, and a methodology for establishing the network is described.

Volume 1, **Executive Summary,** summarizes the research results presented in Volume 2, **Hazards, Barriers, Problems, and the Law,** and Volume 3, **The Development and Evaluation of Countermeasures.**

Limited copies of the reports are available from the Environmental Division, HRS-41, Federal Highway Administration, Washington, D.C. 20590.



Countermeasures for Hydraulic Problems at Bridges: Volume I (Report No. FHWA-RD-78-162) and Volume II (Report No. FHWA-RD-78-163)

by FHWA Environmental Division

When hydraulic conditions endanger the structural integrity of a bridge, either the structure must be reconstructed or the hydraulic conditions must be improved through countermeasures. It is usually less costly to use an appropriate countermeasure such as bank revetment (riprap or slope paving), bed armoring, or flow control measures (spurs, retards, dikes, spur dikes, check dams, and jetty fields). These reports examine countermeasures applied in the United States and Canada and provide guidelines to assist design, construction, and maintenance engineers in selecting measures to reduce scour and bank erosion.

Volume I, **Analysis and Assessment,** discusses hydraulic problems, countermeasures, geomorphic factors, and bridge factors. It documents a survey of stream hazards to bridges, provides a statistical analysis of problems, and rates the performance of countermeasures. Volume I also classifies streams to account for different characteristics of stream lateral stability and behavior.

Volume II, **Case Histories for Sites 1-283,** is a collection of 224 case histories of stream sites. Each case history reports on the original bridge design, flood hazards, countermeasures used, and results.

Limited copies of the reports are available from the Environmental Division, HRS-42, Federal Highway Administration, Washington, D.C. 20590.

Investigation of Tire-Pavement Interaction During Maneuvering, Volume I (Report No. FHWA-RD-78-72), Volume II (Report No. FHWA-RD-78-102), and Volume III (Report No. FHWA-RD-78-103)

by FHWA Structures and Applied Mechanics Division

These reports present the results of analytical and experimental investigations of dynamic tire-pavement interaction phenomena. Volume I, **Theory and Results,** includes a new computerized method for analyzing the fundamental effects of tire material properties, idealized pavement surface texture parameters, sliding speeds, and contact pressures on highway vehicle tire-pavement friction. A finite element tire structure model incorporates the effects of tire geometry and material in the calculation of the shear force generated between tire and pavement—an important element in the precise evaluation of vehicle handling performance. The tire-pavement friction analysis includes 81 individual contact points. Several physical quantities including displacement, sliding velocity, shear force, and the friction coefficient are calculated at each contact point. The actual continuous distribution of each quantity is believed to be adequately represented. Every effort has been made to realistically model tire geometry and material, although the analysis does not yet account for the effect of tread profile.

The pressure and speed dependence of the friction coefficient for low-speed sliding on the lightly wetted asphalt surfaces used in the experimental investigation were also predicted accurately.

The report recommends that further experimental friction data be gathered using actual tire tread and pavement surface specimens to investigate the range of validity of the tire-pavement friction analysis and to guide further generalization of theory.

Volume II, **Computer Program Manual,** and Volume III, **Users Manual,** further document the project.

The reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock Nos. PB 291257, PB 291258, and PB 291259).

Effect of Cargo Shifting on Vehicle Handling, Report No. FHWA-RD-78-76

by FHWA Structures and Applied Mechanics Division

Cargo shifting is a potential problem in practically all types of highway truck handling operations. This report describes a test program that determines to what extent shifting cargoes affect driver control of heavy tractor-semitrailer vehicle combinations. The shifting cargoes studied included water carried in low density and high density semitrailer tankers and free-hanging beef halves carried in a semitrailer refrigerator van. Vehicle handling performance was evaluated relative to baseline tests with a tractor-semitrailer carrying nonshifting cargo.

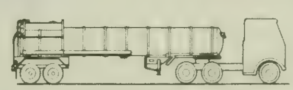
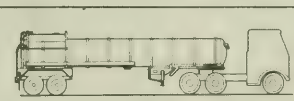
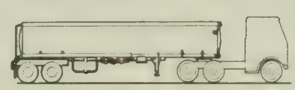
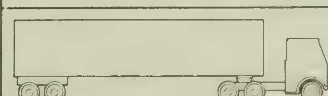
The test program included realistic vehicle braking, cornering, and lane-changing maneuvers by experienced drivers at various speeds and under various roadway geometric and pavement surface conditions. The tests were conducted with vehicle gross load levels up to 338 kN (76,000 lbf). Onboard vehicle instrumentation recorded driver inputs to steering and braking controls and corresponding vehicle dynamic responses during test maneuvers.

Films taken with a motion picture camera during braking tests verified that hanging beef halves in the semitrailer move forward against the restraint when the brakes are first applied and remain there until the stop is completed. A resultant longitudinal surge sensed by the driver under these conditions was also experienced in braking tests with the tankers.

The performance of the test vehicles during cornering maneuvers indicated that the lateral stability was principally affected by the height of the vehicle center of gravity (C.G.) The C.G., which initially is relatively high, may increase while the vehicle is in motion because of dynamic load shifts. The refrigerator van carrying hanging beef halves had a static C.G. height approximately 25 percent higher than the other test vehicles and therefore exhibited more severe lateral instability characteristics.

During lane-changing tests, large roll angles were measured in all vehicles with shifting cargoes. High density tankers had roll angles about twice as large as those of the baseline vehicle; the low density tanker and refrigerator van roll angles were about three times as large as those of the baseline vehicle.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 298110).

TEST VEHICLE		LOADING CONDITION: EMPTY	1/2 FULL	3/4 FULL	7/8 FULL	FULL
BASELINE VAN TRAILER WITH NON-SHIFTING CARGO		X	X			X
HIGH DENSITY TANK TRAILER WITHOUT BAFFLES			X	X	X	
HIGH DENSITY TANK TRAILER WITH BAFFLES			X	X	X	
LOW DENSITY TANK TRAILER WITH COMPARTMENTS				X		
MEAT TRAILER WITH MEAT						X

Implementation/User Items "how-to-do-it"

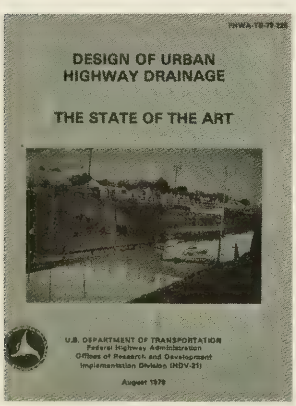
The following are brief descriptions of selected items that have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Office of Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies.

U.S. Department of Transportation
Federal Highway Administration
Office of Development
Implementation Division (HDV-20)
Washington, D.C. 20590

Items available from the Implementation Division can be obtained by including a self-addressed mailing label with the request.

Design of Urban Highway Drainage, The State of the Art, Report No. FHWA-TS-79-225

by FHWA Implementation Division

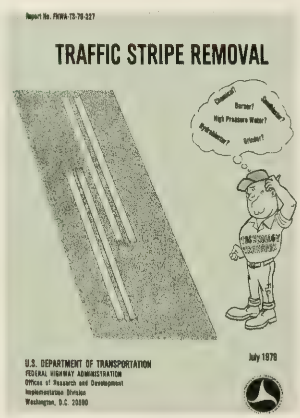


This report presents methods and procedures commonly used in the United States for urban highway stormwater drainage design. The principles, objectives, and design considerations in the current approaches to stormwater drainage include collection, storage treatment, and disposal. Each of these concepts is an integral and interrelated part of any stormwater management system, and each concept is thoroughly discussed in this report.

Limited copies of the report are available from the Implementation Division.

Traffic Stripe Removal, Report No. FHWA-TS-79-227

by FHWA Implementation Division

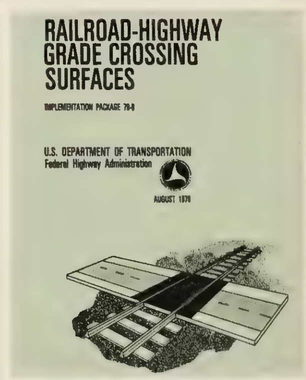


In the past several years much attention has been directed toward the acceptable removal of obsolete pavement markings. There are many methods available for marking removal including chemical paint removers, sandblasting, high pressure water jet, grinding, hydroblasting, and high temperature burning. Although most of these methods are successful to some degree, no one method appears to be superior for use under all conditions. This report summarizes the available methods and includes a brief description and a discussion of the effectiveness, problems, and limitations of each method. Average removal costs are also included.

Limited copies of the report are available from the Implementation Division.

Railroad-Highway Grade Crossing Surfaces, Implementation Package 79-8

by FHWA Implementation Division and the Office of Engineering



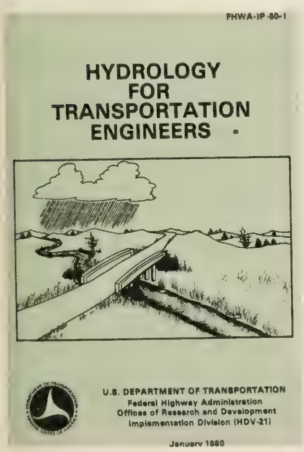
Railroad-highway grade crossings and the physical characteristics of the crossing surface are a concern to both railroad and highway authorities. A crossing requires the normal railroad track structure to be modified, which increases both construction and maintenance costs. In addition, railroad crossings disrupt normal highway surfaces and reduce riding quality. This discontinuity in surface may result in increased vehicle operating costs and significant hazard and inconvenience to vehicular traffic.

This report presents information on available types of railroad-highway grade crossing surfaces. The information should aid in the selection and installation of suitable types of surfacing material. This report supersedes FHPM 6-6-2-3, Railroad-Highway Grade Crossing Surfaces.

The report is available from the Office of Engineering, HNG-14, Federal Highway Administration, Washington, D.C. 20590.

Hydrology for Transportation Engineers, Implementation Package 80-1

by FHWA Implementation Division



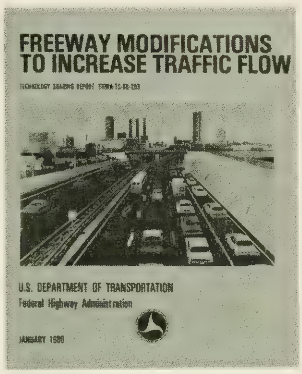
The selection and sizing of drainage structures are essential elements in the development of safe and efficient transportation systems. In the event of flooding, extensive damage may result from inadequate drainage design. Hydrologic analysis is the critical first step in drainage design. The output of this analysis is the hydraulic loading used by drainage engineers to select and size drainage structures.

This manual provides current information to assist design engineers in hydrologic analysis. It covers the essential principles, procedures, and analytical methods needed in making hydrologic studies.

The manual may be purchased for $12 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-002-00108-6).

Freeway Modifications to Increase Traffic Flow, Report No. FHWA-TS-80-203

by FHWA Implementation Division



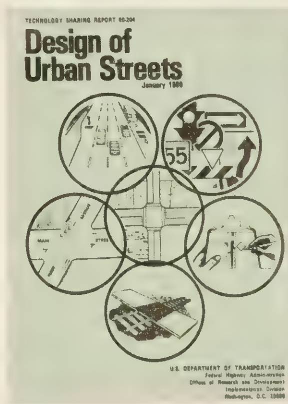
Transportation agencies develop, operate, and maintain urban transportation systems needed to sustain economic development of an area. Because of rising travel demands and costs, buses and other high occupancy vehicles must effectively operate on the streets and freeways of an urban area. To accomplish this, techniques and procedures that provide priority treatment are being designed and implemented. The objectives of these priority treatments are to increase the people-carrying capacity of the roadway and to reduce vehicle-kilometres (vehicle-miles) of travel by encouraging a shift in mode of travel. This report addresses different roadway cross section modifications that can be made to improve the efficiency of freeways.

The report also discusses the experiences of ongoing projects. Low cost, easily implemented modifications to freeway traffic patterns, operational and safety experience resulting from lane adjustments, and suggestions for a public acceptance survey are described.

The report may be purchased for $4 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-003-00371-9).

Design of Urban Streets, Report No. FHWA-TS-80-204

by FHWA Implementation Division

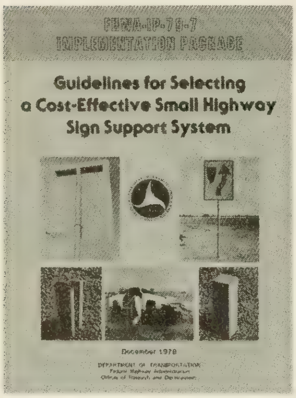


Increases in the urban population in the United States have created serious deficiencies in many urban transportation services. Because of this, increased emphasis must be placed on effective operation of the existing urban street network through specialized training in design. This report provides practical, state-of-the-art information to aid in the design and operation of urban facilities and streets. Major emphasis is placed on the functional or operational safety aspects of design that apply to minor design revisions of existing facilities as well as to major reconstruction or new construction.

The report may be purchased for $10 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-003-00373-5).

Guidelines for Selecting a Cost-Effective Small Highway Sign Support System, Implementation Package 79-7

by FHWA Implementation Division



The probability of severe accidents from impacts with small sign supports is increasing because of greater numbers of small automobiles on the highways. Support systems are being reexamined to evaluate the hazards the supports present and the replacement costs as a result of accidents, vandalism, and normal wear.

This report provides criteria for selecting the most cost-effective systems from among those that can respond favorably under impact with a 1.02 Mg (2,250 lb) vehicle.

A 30-minute film, "Small Sign Supports," is also available. It describes the scope of small sign use within highway jurisdictions, the crash tests that identified the support types that responded satisfactorily to small automobile impacts, and the procedure to consider in making a cost effectiveness analysis.

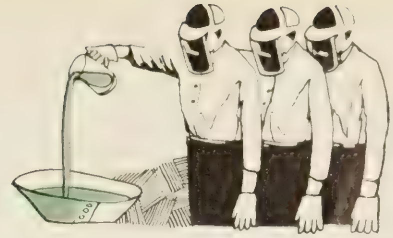
Limited copies of the report are available from the Implementation Division. The report also may be purchased for $3.50 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050-001-00158-6). The film is available from the Implementation Division.

New Publication

1979 Standard Highway Signs contains detailed drawings of the standard highway signs prescribed in the 1978 edition of the Manual on Uniform Traffic Control Devices. The drawings are provided to promote nationally uniform highway sign design in accordance with Title 23, U.S. Code, Sections 109(b), 109(d), and 402(a) and Highway Safety Program Standard 13, Traffic Engineering Services. The standards apply to signs used on all streets and highways, regardless of type or class or the government agency having jurisdiction. This booklet also contains general design guidelines for developing signs that conform with the basic standards. Most guide signs must be designed separately because of the variability of message or legend. An example design of an overhead advance guide sign is included. Specific design criteria are included for designing diagrammatic signs.

The booklet may be purchased for $10 from the Superintendent of Documents, U.S. Government Printing Office (GPO), Washington, D.C. 20402. There is a 25 percent discount for orders of 100 or more shipped to one address. The cost includes the design details, a looseleaf binder, and a subscription service maintained by GPO for subsequent page changes.

New Research in Progress



The following items identify new research studies that have been reported by FHWA's Offices of Research and Development. Space limitation precludes publishing a complete list. These studies are sponsored in whole or in part with Federal highway funds. For further details, please contact the following: Staff and Contract Research—Editor; Highway Planning and Research (HP&R)—Performing State Highway or Transportation Department; National Cooperative Highway Research Program (NCHRP)—Program Director, National Cooperative Highway Research Program, Transportation Research Board, 2101 Constitution Avenue, NW., Washington, D.C. 20418.

FCP Category 1—Improved Highway Design and Operation for Safety

FCP Project 1M: Rural Highway Operational Safety Improvements

Title: Quantification of Two-Lane Rural Highway Problems. (FCP No. 31M1012)
Objective: Consider the design characteristics, traffic demands, traffic mix, peak volumes, traffic control, traffic conflicts, and accident characteristics as a function of the purpose, use, and class of highways.
Performing Organization: JHK and Associates, Alexandria, Va. 22304
Expected Completion Date: July 1981
Estimated Cost: $427,000 (FHWA Administrative Contract)

Title: Safety and Operational Impact of 3R-Type Projects. (FCP No. 31M1022)
Objective: Evaluate contiguous geometric and safety improvements and the relative cost effectiveness of 3R-type applications for obtaining two-lane rural highway safety improvements.
Performing Organization: Systems and Applied Sciences Corporation, Riverdale, Md. 20840
Expected Completion Date: November 1981
Estimated Cost: $291,000 (FHWA Administrative Contract)

Title: Development of Warrants for Special Turning Lanes at Rural Nonsignalized Intersections. (FCP No. 41M2542)
Objective: Evaluate the effectiveness of special turning lanes in reducing accidents and congestion, assess the cost effectiveness of these lanes, and develop warrants for their implementation.
Performing Organization: University of Nebraska, Lincoln, Nebr. 68588
Funding Agency: Nebraska Department of Roads
Expected Completion Date: June 1981
Estimated Cost: $408,000 (HP&R)

FCP Project 1T: Advanced Vehicle Protection Systems

Title: Development of a Lightweight Truck-Mounted Crash Attenuator (TMA). (FCP No. 41T6103)
Objective: Develop a TMA that is lighter, less expensive, and more effective than current designs.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95805
Expected Completion Date: June 1981
Estimated Cost: $149,000 (HP&R)

FCP Project 1Z: Implementation of Previously Completed Safety Projects

Title: Process for Safety Improvement. (FCP No. 31Z1333)
Objective: Develop a users manual and training course depicting the various processes of a highway safety improvement program (FHPM 8-2-3).
Performing Organization: Goodell-Grivas, Inc., Southfield, Mich. 48075
Expected Completion Date: January 1981
Estimated Cost: $89,000 (FHWA Administrative Contract)

FCP Category 2—Reduction of Traffic Congestion, and Improved Operational Efficiency

FCP Project 2M: Arterial Flow and Control

Title: Urban Signalized Intersection Capacity (3-28(2)). (FCP No. 52M1057)
Objective: Define terms, concepts, and computational procedures for the analysis of traffic signal controlled (pretimed and actuated) intersections for determining their level of service and capacity.
Performing Organization: JHK and Associates, Tucson, Ariz. 85701
Expected Completion Date: September 1981
Estimated Cost: $290,000 (NCHRP)

FCP Category 3—Environmental Considerations in Highway Design, Location, Construction, and Operation

FCP Project 3E: Reduction of Environmental Hazards to Water Resources Due to the Highway System

Title: Effects of Highway Runoff on Receiving Waters. (FCP No. 33E3222)
Objective: Determine the impact of highway runoff on receiving water quality and aquatic biota. Formulate guidelines for assessing impacts. Provide guidelines for conducting

field studies to determine effects of highway runoff on receiving waters.
Performing Organization: Rexnord, Environmental Research Center, Milwaukee, Wis. 53201
Expected Completion Date: October 1982
Estimated Cost: $529,000 (FHWA Administrative Contract)

FCP Category 5—Improved Design to Reduce Costs, Extend Life Expectancy, and Insure Structural Safety

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Performance, Reliability, and Durability of State-of-the-Art Weigh-in-Motion Systems for Highway Vehicles. (FCP No. 45D3214)
Objective: Evaluate system performance, reliability, and durability of present state-of-the-art WIM weigh bridges and systems for acquisition of highway statistical traffic data (Phase A) and for truck overweight screening (Phase 3).
Performing Organization: California Department of Transportation, Sacramento, Calif. 95805
Expected Completion Date: June 1981
Estimated Cost: $208,000 (HP&R)

FCP Project 5E: Premium Pavements for "Zero Maintenance"

Title: Environmental Effects for Zero Maintenance Pavements. (FCP No. 35E3012)
Objective: Identify and define interdependency relationships and interaction criteria between aging, environmental, and load effects upon life cycle of pavements and their support systems. Develop techniques to evaluate interdependency and provide documentation to advance present technology leading to a complete set of procedures to account for aging, environmental, and load effects in pavements.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Expected Completion Date: December 1981
Estimated Cost: $230,000 (FHWA Administrative Contract)

FCP Project 5K: New Bridge Design Concepts

Title: Signal Enhancement Interpretation for Detection of Flaws in Reinforcing Steel and Prestressed Concrete Bridge Members. (FCP No. 35K1072)
Objective: Upgrade existing prototype FHWA magnetic field disturbance (MFD) system to incorporate improved signature acquisition and processing capability and to provide immediate readout of physical condition of reinforcing steel.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: January 1981
Estimated Cost: $126,000 (FHWA Administrative Contract)

FCP Project 5L: Safe Life Design for Bridges

Title: Acoustic Emission Monitoring of Steel Structures. (FCP No. 45L2001)
Objective: Perform acoustic emission monitoring of electroslag butt weldments in Dunbar Bridge, Charleston, W. Va.
Performing Organization: Dunegan/Endevco Corporation, Charleston, W. Va. 25305
Expected Completion Date: October 1981
Estimated Cost: $225,000 (HP&R)

FCP Project 5M: Rehabilitation and Maintenance of Low-Volume Roads

Title: Design and Construction of Low-Water Stream Crossings. (FCP No. 35M1052)
Objective: Produce guidelines with documentation for selection, design, and construction of low-water stream crossings on low-volume roads. Base the guide on a review of the state of the art and case studies to develop design criteria and methodology, system drawings, and construction procedures for use by county and local engineers.
Performing Organization: Williams and Sheladia, Mt. Rainer, Md. 20822
Expected Completion Date: October 1981
Estimated Cost: $251,000 (FHWA Administrative Contract)

Title: Interim Guidelines for Using Marginal Soil and Aggregate Materials and Waste Materials for Constructing and Maintaining Low-Volume Roads. (FCP No. 35M3013)
Objective: Identify available sources of low-cost materials suitable for use in constructing and maintaining low-volume roads. Provide guidelines for the proper use and treatment of these materials.
Performing Organization: Globetrotters Engineering Corporation, Chicago, Ill. 60605
Expected Completion Date: May 1981
Estimated Cost: $117,000 (FHWA Administrative Contract)

FCP Category 0—Other New Studies

Title: Impact of Variations in Material Properties on Asphalt Pavement Life. (FCP No. 40M1712)
Objective: Identify causes of performance problems in asphalt pavements, estimate effects on pavement life of noncompliance with existing pavements, and develop improved guidelines for controlling materials and construction.
Performing Organization: Oregon State University, Corvallis, Oreg. 97330
Funding Agency: Oregon Department of Transportation
Expected Completion Date: June 1981
Estimated Cost: $65,000 (HP&R)

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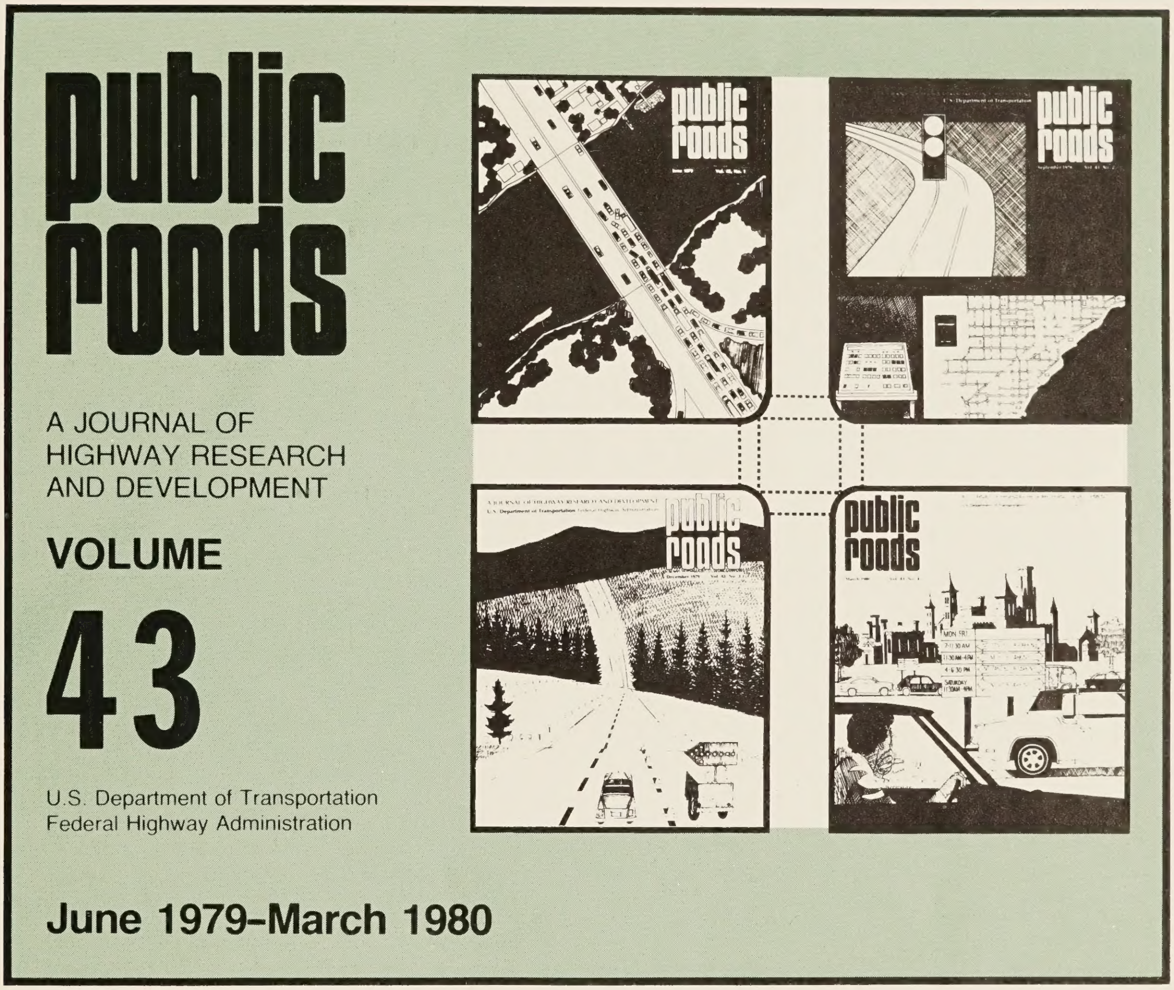
TITLE SHEET, VOLUME 43



The title sheet for volume 43, June 1979–March 1980, of Public Roads, A Journal of Highway Research and Development, is now available. This sheet contains a chronological list of article titles and an alphabetical list of authors' names. Copies of this title sheet can be obtained by sending a request to the editor of the magazine, U.S. Department of Transportation, Federal Highway Administration, HDV-14, Washington, D.C. 20590.

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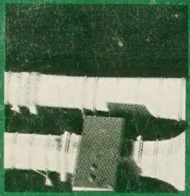

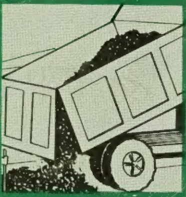

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