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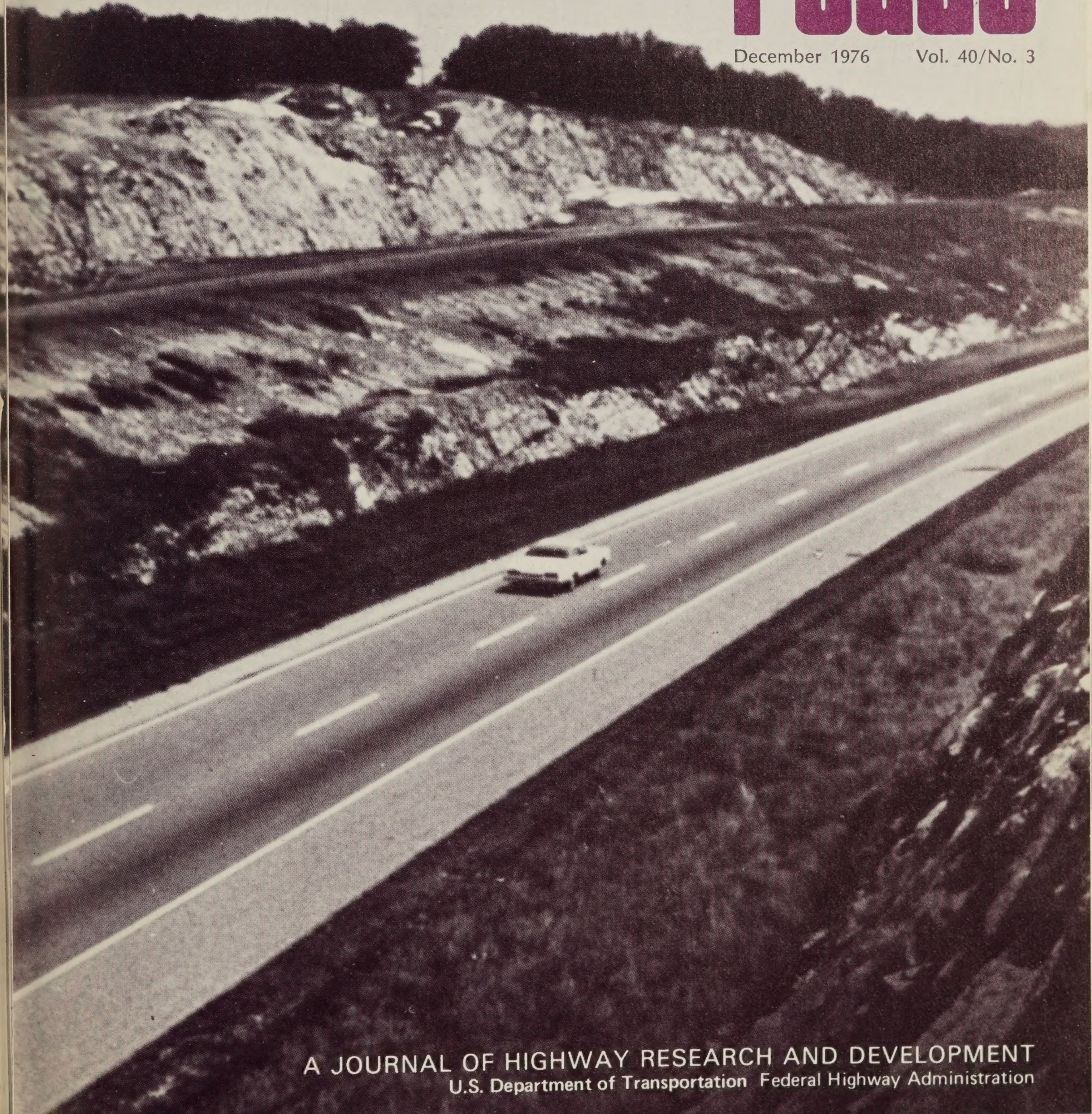

A JOURNAL OF HIGHWAY RESEARCH AND DEVELOPMENT

U.S. Department of Transportation Federal Highway Administration

public roads

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December 1976 Vol. 40/No. 3

COVER:
I–295 northbound, Cranston, R.I.—only necessary ledge cuts were made so this segment of highway would blend in with the surrounding environment.

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IN THIS ISSUE

Articles

Departments

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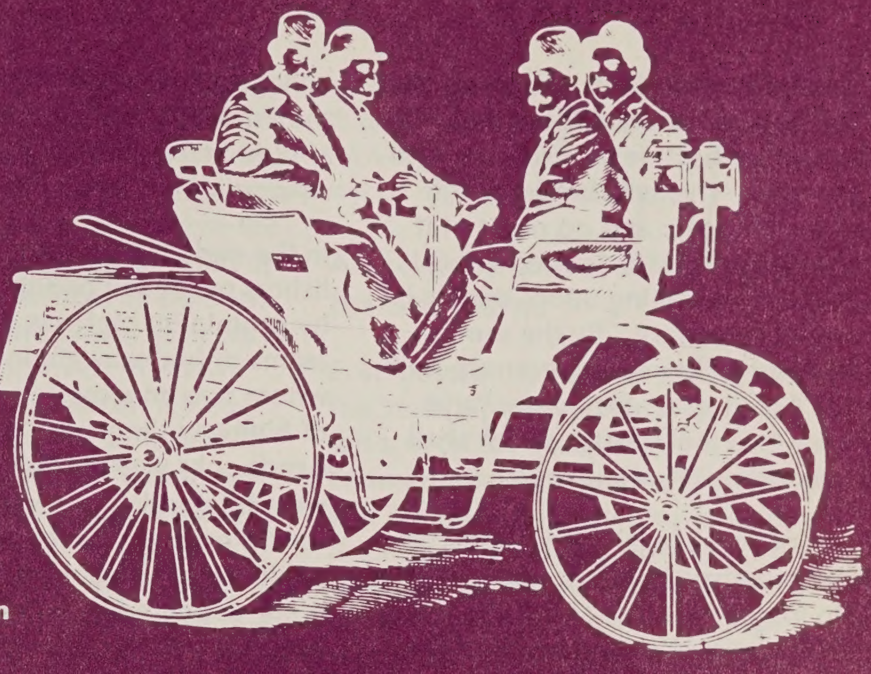
The Secretary of Transportation has determined that the publication of this periodical is necessary in the transaction of the public business required by law of this Department. Use of funds for printing this periodical has been approved by the Director of the Office of Management and Budget through March 31, 1981.



Highway Design for Motor Vehicles—A Historical Review

Part 8: The Evolution of Highway Standards

by [1] Frederick W. Cron

This is the last in a series of eight historical articles tracing the evolution of present highway design practices and standards in the United States. The Introduction and Part 1: The Beginnings of Traffic Measurement were published in vol. 38, No. 3, December 1974. Part 2: The Beginnings of Traffic Research was published in vol. 38, No. 4, March 1975. Part 3: The Interaction of the Driver, the Vehicle, and the Highway was published in vol. 39, No. 2, September 1975. Part 4: The Vehicle-Carrying Capacity of the Highway was published in vol. 39, No. 3, December 1975. Part 5: The Dynamics of Highway Curvature was published in vol. 39, No. 4, March 1976. Part 6: Development of a Rational System of Geometric Design was published in vol. 40, No. 1, June 1976, and Part 7: The Evolution of Highway Grade Design was published in vol. 40, No. 2, September 1976.

[1] Frederick W. Cron's biography appears on page 100 of this issue.

Highway standards in the United States are the distilled essence of the experience and judgment of hundreds of engineers and administrators—a consensus of many informed people. Highway standards change continually, though slowly, to adjust to changes in vehicle and driver capabilities, as well as the changing economic status of society.

Earliest Standards in Legal Form

For the earliest standards the informed consensus was expressed in laws. The Twelve Tables of 450 B.C. (the earliest Roman legal code) legally distinguished roads by their width. The *semita* or foot path was only 1 foot [2] wide; the *iter* for horsemen and pedestrians was 3 feet; the *actus* for a single carriage was 4 feet; and the two-lane *via* was 8 feet wide. (*1*) [3] These specific dimensions were modified in the course of the evolutionary changes all standards seem to experience, the *via* being established as about 12 feet wide when straight or 16 feet when crooked. (2)

[2] The Roman foot was about 11.66 English inches or 295 mm.

[3] Italic numbers in parentheses identify the references on page 100.

A Norse law of the year 950, required that a main road should be as wide as a spear was long, the shaft of which rested on the ground and the head of which a mounted man could reach with his thumb. (*3*)

Under the laws of Napoleon I, French highways of the first class were required to have rights-of-way of 20 meters, with the 7 meters in the center stoned or paved. Roads of the second and third classes had rights-of-way 15 and 10 meters wide, while roads of the fourth class—the town and local service roads—were required to be only 8 meters wide. (2)

In England in 1839, the legally prescribed width for turnpike roads at the approaches to populous cities was 60 feet. The less important or byroads were 20 feet wide for carriage roads, 8 feet for horse roads, and 6½ feet for footpaths. (2)

The act authorizing construction of the National Road in 1806 specified that the road should have a right-of-way of 80 feet, be paved with stone to a width of 20 feet, and that the gradient should not exceed 5 degrees from the horizontal (8.75 percent).

According to the 1845 statutes of New York State, ordinary public roads were required to be at least 3 rods or 49.5 feet wide: "This is to be the width between fences; and no more of it need be worked, or formed into a surface for travelling upon, than is deemed necessary." By the same statute turnpike roads were required to be 4 rods or 66 feet wide, "and twenty-two feet of such width shall be bedded with stone." (2)

Some of these legal road standards were negative: Rhode Island's original State highway law of about 1900, for example, restricted the width of the metaled surface to 14 feet.

The standards governing size, weight, and maximum speed of vehicles are still in legal form in most countries, but the determination of other standards has for the most part been delegated to highway departments and similar administrative bodies.

The Repositories of Good Practice

The writings of Trésaguet, Telford, and McAdam are a possible exception to the rule that standards are established by consensus. The reports and treatises of these eminent roadbuilders were so widely circulated and read in the 19th century that their recommendations became in effect international standards.

Other repositories of the standards of good practice were the manuals of organizations such as the French Corps of Bridges and Roads and the U.S. Army Corps of Engineers, and books compiled by eminent professors of civil engineering in technical colleges. These last, in particular, kept the art of roadbuilding alive in the United States during the "dark age of roadbuilding" from 1860 to 1880.

In the early 20th century a few publishers produced handbooks authored by prominent highway engineers or teams of highway engineers, each of whom was an expert in some aspect of highway theory or practice. These handbooks were widely used and were influential in developing a consensus on many aspects of highway engineering. Revised editions generally appeared at 5-year intervals to keep up with rapidly changing developments. These comprehensive handbooks, with the equally valuable college textbooks on engineering, still play an important part in refining, winnowing, and concentrating the fruits of research and practice which eventually find expression in standards.

Beginning in 1891 the *Good Roads Movement* revived interest in roadbuilding. In 1891 the State of New Jersey authorized financial aid to counties to help them improve their roads. In 1893 Massachusetts authorized the construction of State highways and set up a State highway department to do the job. Other States, mostly in the East, soon followed this example, and by 1910 over 25 States had their own highway systems and departments. These highway departments drew up *standard cross-sections* to guide their engineers and designers, and those of the wealthier, more advanced States were widely copied by other States and by counties and cities. This tendency to treat the practices of important States as "standards" was deplored by many engineers:

> It is perhaps somewhat unfortunate that the word "standards" should have been chosen to designate these plans. Strictly interpreted, the meaning would indicate that the standard design was the best design. This is by no means the case—nor is it intended to mean this. Standards are merely recommended designs which are to be adhered to unless conditions indicate that a variation in the design would meet them better.
>
> As a rule they are designs prepared by engineers of wide experience . . . and they represent the crystallization of ideas tempered by mature judgment and years of observation. . . .
>
> There is, however, a grave danger attendant on the use of standards of any kind. The temptation is to neglect the detailed study of local conditions and use a standard structure. This often results not only in an unwarranted increase in . . . cost, but may result in a type of construction which fits but poorly the location where used. (4)

This is good advice today.

Influence of the Federal Government on Standards

Until 1916 there was no central clearinghouse in the United States for road policy and standards. The Federal Government had relinquished responsibility for roads in 1838, turning the Old National Pike, which had been built with Federal funds, over to the States through which it passed to be operated as a toll road. Each State, county, and city had its own standards and rules for roadbuilding. Consulting engineers, college professors of engineering, and the private publishers of engineering handbooks played a vital role by disseminating the best feature of current experience.

With the Federal Aid Road Act of 1916 the Federal Government again asserted an influence on road policy, but with regard to physical standards this influence was muted. The 1916 Act authorized grants-in-aid to the States for roadbuilding and directed that the Secretary of Agriculture and the State highway department of each State should agree upon the roads to be onstructed "and the character and iethod of construction" and further, ıat the Secretary "shall approve only uch projects as may be substantial ı character."

he Secretary, operating through the ureau of Public Roads (BPR), inter-reted the word *substantial* with dmirable flexibility. The BPR recog-ized that:

> ı improvement which is substantial for one ənsity and kind of traffic may not be sub-antial for another. . . . the types of roads hich it is desirable to construct in New York, assachusetts, and Pennsylvania are not suit-ɔle or necessary for Nevada, Idaho, and the akotas. . . . the decision as to the type of ɪad which the Secretary will approve for a ven locality has been based in every case ɔon the traffic which is using the existing ɪad and which is estimated will use the ıproved road. (5)

he result of this policy was that the PR "approved roads of all types and idths, from graded earth roads to concrete, brick, or bituminous concrete, narrow as well as wide." Over 66 percent of these roads were earth, sand-clay, or gravel and located largely "in sections of the country where the pioneering work required to open up new territory yet remains to be done. Earth, sand-clay, and gravel surfaces have also been approved in many instances for projects which it is intended at a later date to surface with a more durable material. Whatever money is expended upon such projects for grading and drainage, which represent the major work involved in them, is money well spent for permanent improvement." (5)

The requirement that plans for Federal-aid projects be approved by the Secretary gave the BPR a unique opportunity to review standards in all parts of the United States. The BPR in turn was able to influence design policy in the States by passing on to all the highway departments those designs that had proved effective or economical. This liaison was not, however, equivalent to recommending or enforcing standards.

ıe *National Pike.*

AASHO Becomes Clearinghouse for Standards

Even before the Federal Aid Road Act, the State highway departments had felt a need for a way to come together in an atmosphere free from political and commercial pressures to discuss the many legislative, economic, and technical problems which all of them faced as a result of the headlong motorization of highway traffic that was taking place in the United States. In answer to this need they formed the American Association of State Highway Officials (AASHO) in 1914. Initially, AASHO's Committee on Standards confined itself to disseminating information on design to its members, but in 1928 it proposed that the Association adopt "standards of practice" to guide the member States in technical matters in which some uniformity from State to State was urgently needed. As a result, on March 1, 1928, AASHO approved its first four standards which read as follows:

- That wherever practicable shoulders along the edges of pavements shall have a standard width of not less than 8 feet.
- That on pavements 10 feet shall be considered as the standard width for each traffic lane.
- That the crown of a two-lane concrete pavement shall be 1 inch.
- That no part of a concrete pavement shall have a thickness of less than 6 inches, and that all unsupported edges shall be strengthened. *(6)*

Cautiously, almost reluctantly, AASHO moved to fill what had become a widening gap between the technical knowledge of highway engineering and actual design practice by issuing "Standards of Practice" and "Policies" from time to time which summarized the state of the art and set forth what its members considered to be good practice. These standards and policies were revised or updated over the years to keep pace with the evolution of highway design.

The AASHO standards and policies are not obligatory on the States, but they have acquired such prestige over the years that they have in effect become national standards. In practice, each State establishes its own standards which are generally in accordance with the AASHO recommendations but may vary. After approval by the BPR these standards are required to be used for roads financed by Federal-aid funds in that State. The AASHO standards have influenced the practice of many other countries, especially those of the Western Hemisphere.

Today all countries have their own highway standards. There are strong similarities between these national standards, which should not be surprising since motor vehicles and their drivers are quite similar the world over. The activities of international associations such as the Permanent International Association of Road Congresses, the International Road Federation, and the Pan American Highway Congresses have produced a healthy cross-fertilization of engineering thought and practice which has found expression in national road standards. Another strong influence in recent years has been the activity of international engineering consultants who do a worldwide business, have an opportunity to study and evaluate standards in different countries, and introduce values being used elsewhere. The international lending organizations and the agencies for bilateral and multilateral economic aid exert yet another leveling influence on standards.

Standards and Road Classification

In the past, road standards were often closely tied to the roads' strategic and political importance. In France, for example, Napoleon's decree of December 16, 1811, established Imperial roads of the First Class extending from Paris to the more important cities on the frontier, roads of the Second Class from Paris to the less important frontier cities, and roads of the Third Class joining the interior cities. Roads of the First Class had a right-of-way width of 20 meters of which 7 meters in the center were stoned or paved. These dimensions were reduced to 15 meters and 6 meters for Second Class roads and 10 meters and 5 meters for Third Class roads. Also, steeper grades could be used on roads of the Second and Third Classes. A Fourth Class of local roads and village streets had even lower standards. (2)

To some extent, the standards for the German Autobahnen and the low-traffic sections of the U.S. Interstate System were established in the same manner, that is, by the supposed importance of the road, rather than by the requirements to serve present and forecasted traffic.

For most roads such a system is inadequate for classifying road standards, because it does not recognize the great variations in traffic volume that occur from place to place on the same road. The same can be said of functional systems such as "primary" and "secondary" and "feeder"; nevertheless, function is probably still the most widely used basis for classifying road standards.

Traffic Is the Primary Determinant for Standards

Most engineers acknowledge that traffic is the primary determinant for road standards, overriding all others. Apparently this was first recognized by AASHO, which set up five *classes* of roads according to their traffic volumes in its standards of September 30, 1931:

Class	Traffic volume (ADT)
A	4,000 or more
B	750 to 4,000
C	300 to 750
D (minor trunkline)	300 maximum
E (local roads)	200 maximum

For each class AASHO recommended "desirable standards of practice applicable, except under extraordinary or special conditions" for width of right-of-way, pavement, shoulders and bridges, gradient limits, pavement type, and minimum radius of curvature.

AASHO's standards for 1941 introduced two new elements into standards classification: *design speed* and *character of traffic*. The design speed was an indirect recognition of the influence of topography on highway design. Passing and nonpassing sight distance and horizontal curvature were directly related to design speed so by selecting a lower design speed the designer automatically adjusted his design to rougher topography.

The *character of traffic* parameter was an attempt to recognize the effect of trucks and buses on design standards. Roads were to be classified as *passenger traffic (P), mixed traffic (M),* or *truck traffic (T)* roads, according to

/hether the percentage of trucks and uses in the traffic stream was low, noderate, or large. High-speed roads f the *T* type justified pavement /idths as great as 24 feet, while low-peed, passenger-type roads might et by with as little as 16 feet of avement.

ASHO introduced topography directly as a design parameter in its 1945 tandards for interstate highways and econdary and feeder roads. These tandards assumed three types of opography—*flat, rolling,* and *mountainous*—with varying design speeds nd gradients for each type. A second nnovation was the sanctioning of hat amounted to two levels of design: one using "minimum" standards nd the other "desirable" standards. lassification by type of traffic—*(P), M), (T)*—was abandoned at this time nd its design function has since been ccomplished by expressing heavy ehicles as a percentage of the total DT. With these new features the ontent of geometric road standards ecame essentially what it is today.

Highway Bridge Standards

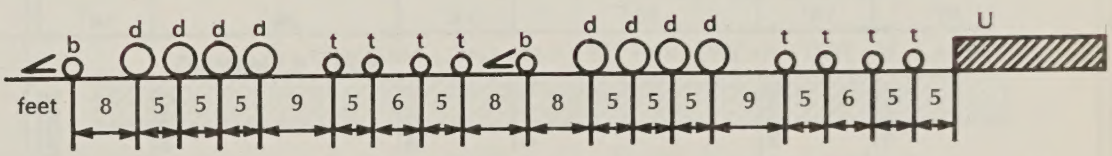
I the 1800's, bridges were described r classified by their shape—trabeate beam-shaped, arcuate or bow-shaped, or suspension—and by the aterial of which they were made. hey were designed to carry the reatest loads that could be placed pon them in addition to the dead ad of the structure itself. The greatest possible live load for a highway idge was a crowd of people, equal 70 pounds per square foot of deck. drove of cattle was 40 pounds per uare foot and a double row of avily loaded wagons, with horses, as 600 pounds per running foot, or pounds per square foot. For railad bridges a heavy freight train weighed half a ton per running foot and a row of steam locomotives 1 ton per running foot. There were rules of thumb for estimating the dead-load weights of bridges (of which there were dozens of types and sub-types) and for the "increase of weight by velocity" (impact stress). (2)

The sizing of bridge members depended not only on the estimated dead and live loadings but also on the assumed strength of the materials of which they were made. Materials, especially metals, were less reliable then than now, so it was common practice to subject bridges to test loads before acceptance. In France, iron railroad bridges were required by law to sustain a test load of 5,000 kg per linear meter for structures under 20 meters span and 4,000 kg per meter for longer spans, but not less than a total of 100 metric tons. For highway bridges, in France and elsewhere, test loads were 200 to 400 kg per square meter of deck depending on the importance of the bridge, and the probability of heavy loads. (2)

In the late 1800's a number of American railroads published "General Specifications" to govern the design, fabrication, and erection of their bridges. Consulting bridge engineers and manufacturers such as the American Bridge Company also published specifications, among which were those of Theodore Cooper, first published in 1894. In the United States these standard specifications became the repositories of the fundamentals of the rapidly evolving science of bridge design—the essence of what had been learned in nearly a half-century of experience with modern structures and materials.

Standard Live Loadings Introduced

Vehicles on railroads and highways were of many sizes and wheel arrangements, so Cooper and others introduced *standard live loadings* to help designers. Cooper's standard steam railroad loadings of 1901 assumed two consolidation locomotives, each with its tender, pulling a train of unlimited length. This produced a series of concentrated wheel loads at fixed spacings, followed by the train, assumed to be a uniformly distributed load (fig. 1).



Load in lbs. on one pair of wheels for each track

Class	Truck (bogie) b	Driver d	Tender t	Train load lbs. per lin. ft. U
E 27	13,500	27,000	17,550	2,700
E 30	15,000	30,000	19,500	3,000
E 35	17,500	35,000	22,750	3,500
E 40	20,000	40,000	26,000	4,000
E 50	25,000	50,000	32,500	5,000

Figure 1.—Cooper's standard live loading for steam railroad bridges.

The loading class was designated by the letter E (for engine) followed by two digits which represented the load on one pair of driving wheels in thousands of pounds. Thus, Cooper's E–50 loading meant the load imposed by two locomotives each with four pairs of drivers, each pair carrying 50,000 pounds, and the two locomotives pulling a train weighing 5,000 pounds per linear foot. (7) There were at least four systems of standard railroad loadings other than Cooper's each differing slightly from the others in the distribution and magnitude of the concentrated loads, but these are seldom used today, Cooper's loadings having become the "standard."

Standard live loadings simplified the work of bridge designers. More importantly, they provided an easily understood measure of the capacity of bridges to support loads, and therefore the train capacity of lines and systems.

Cooper also published standard live loadings for highway bridges. These varied with the bridge class, of which there were four, ranging from *Class A* for city bridges capable of carrying the heaviest loads, down to *Class D* for light duty, country roads. Class A loading, for example, assumed a concentrated load of one 24-ton wagon on two axles 10 feet apart positioned anywhere on the deck, or a uniformly distributed load of 100 pounds per square foot. (7)

For each of these classes vertical and horizontal clearances were specified. The usual vertical *headway* dimensions were 14.0 to 15.0 ft for Classes A, B, and C, and 12.5 ft for Class D. Horizontal clearance was specified as 14 inches greater than the width of roadway between wheelguards or curbs. (7)

In 1919 the Illinois Highway Department's standard loading for concrete bridges was a uniformly distributed load of 125 pounds per square foot, or the concentrated loads imposed by a 24-ton steam traction engine, whichever produced the greatest stress. (8) This loading was essentially the same as Cooper's Class A highway loading, except for the increase in the alternative uniformly distributed load. When Illinois adopted this specification, steam plowing engines were fairly common; but after 1918 they were replaced by gas or oil tractors weighing only half as much.

Standard Bridge Specifications Issued by AASHO

In 1921 AASHO created a Committee on Roads and Bridges to promote more uniform practice in the design and construction of highway bridges. This Committee published preliminar specifications in mimeograph form from time to time, which were used extensively by many States in prepar

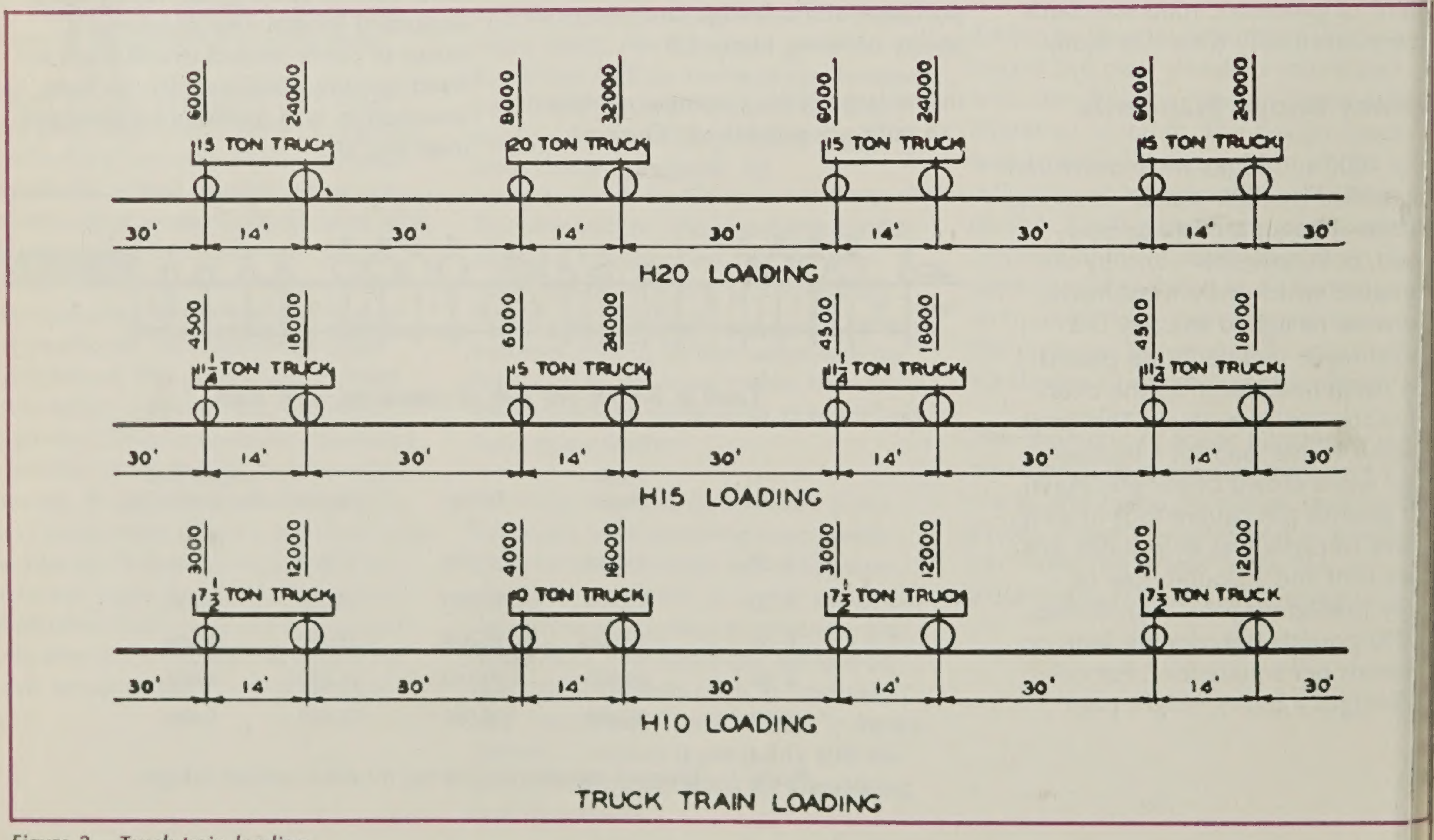


Figure 2.—Truck train loading.

ng their own bridge specifications. 'he Committee assembled these pre-minary specifications and much addi-.ional material into a book which ASHO published in 1931 under the itle "Standard Specifications for High-vay Bridges and Incidental Struc-ures." This publication was very well eceived in the United States and in a emarkably short time it became the tandard for practically all new high-vay bridges. It was also widely used r copied abroad.

he 1931 Specifications proposed four tandard classes for highway bridges:

:lass AA—Bridges where the passage f very heavy loads is frequent.

:lass A—Bridges for normally heavy affic with occasional very heavy oads.

:lass B—Temporary or semi-perma-ent structures for light traffic with ccasional normally heavy loads.

:lass C—Bridges for both highway nd electric railroad traffic.

hese specifications also proposed ree standard live loadings to sim-lify the computation of stresses:

20 Loading for Class AA Bridges

15 Loading for Class A Bridges

10 Loading for Class B Bridges

or spans under 60 feet, the H 20 ading assumed a 20-ton truck with -foot wheelbase to be on the bridge, receded and followed by an indefi-ite number of 15-ton trucks, all paced 30 feet apart (fig. 2). For spans 60 feet or longer the Committee rovided an equivalent H 20 uniform ading of 640 pounds per linear foot lane combined with a single con-entrated load which could be placed nywhere on the span to produce aximum stress (fig. 3). H 15 loading as three-fourths of H 20 loading, and 10 was one-half of H 20 loading.

For two-lane bridges without railway tracks the AASHO specifications required a curb-to-curb deck width of at least 18.0 feet with not less than 6 inches horizontal clearance beyond the curb. The minimum vertical clearance was 14.0 feet.

The AASHO Committee on Bridges and Structures has kept the Standard Specifications up to date by frequent revisions, the latest being the eleventh, issued in 1973. The most extensive changes occurred in the 1944 edition which introduced two extra-heavy live loadings to provide for the larger and heavier vehicles such as tractor-semi-trailer combinations; also, the 1944 specification required greater horizontal clearances.

Most countries have their own bridge specifications or permit the use of other established and recognized specifications such as those of the British or French governments or those of AASHO.[4] Standard design loadings are important features of all these specifications. These standard live loadings are much alike in principle, but the configurations of the design vehicles and their axle loads vary from country to country, as do the alternate uniform lane loadings which can be used in lieu of the design vehicles. Basically, all are variations of

[4] AASHO is now AASHTO, the name having been changed in 1974 to "American Association of State Highway and Transportation Officials."

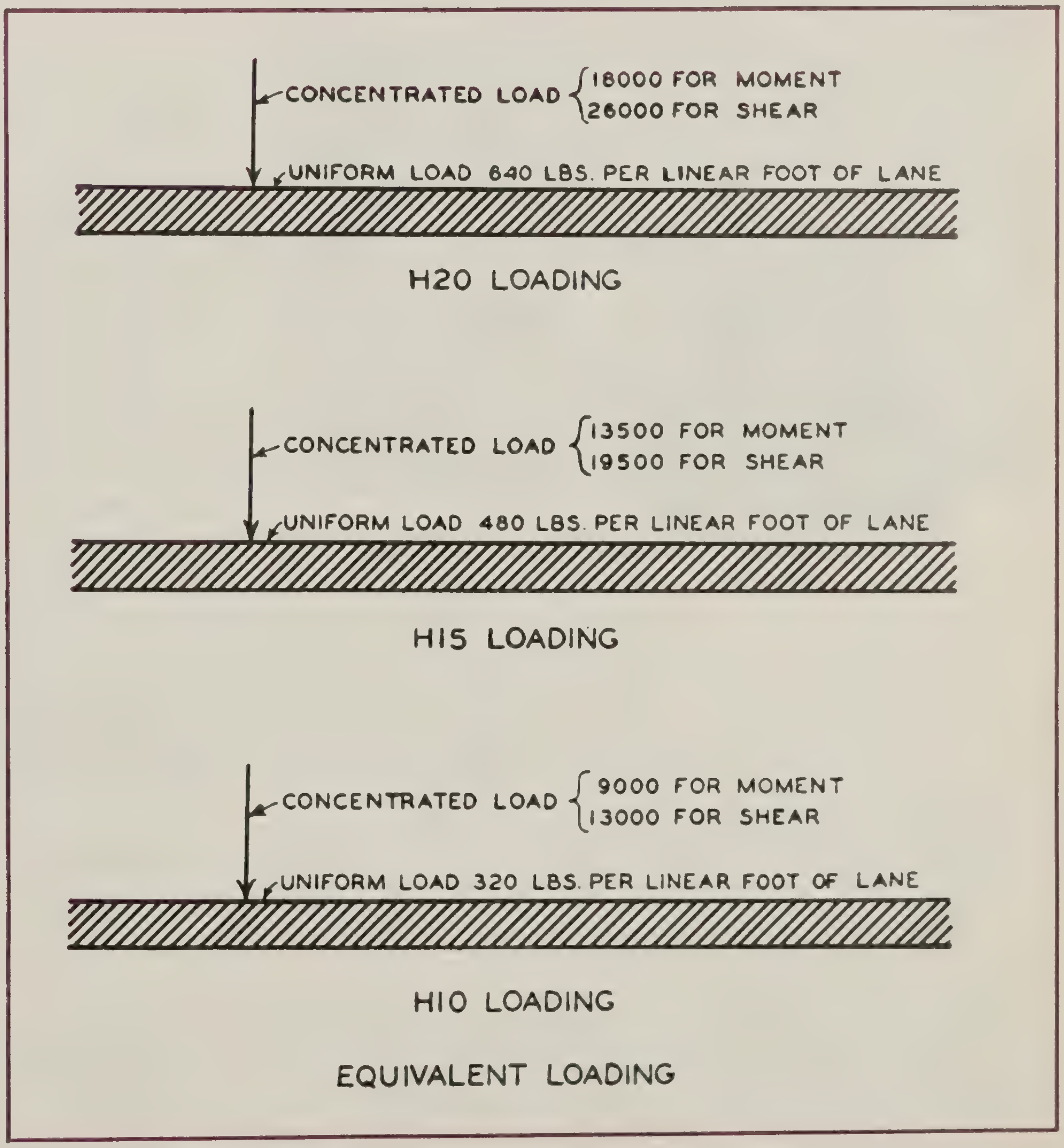


Figure 3.—Equivalent loading.

three general types; the type of loading which produces maximum stress controls the design:

- A simplified "design vehicle" of two or more axles with specified axle loadings and spacings. One or more of these vehicles may be placed anywhere on the bridge to produce maximum stresses in the bridge members.
- A uniform lane loading of a specified weight per square foot or square meter of deck.
- A single extremely heavy axle load (or knife-edge load or wheel load) which can be positioned anywhere on the bridge to produce maximum stress, usually in conjunction with a uniform lane loading.

The American Association of State Highway and Transportation Officials' specifications are the nearest approach to a world standard for bridge design. They have been adopted or copied in about two-thirds of all countries.

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(9) "Standard Specifications for Highway Bridges and Incidental Structures," *American Association of State Highway Officials*, 1931, pp. 174, 175.

This is the final part in an eight-part series of historical articles by Frederick W. Cron. Mr. Cron joined the Bureau of Public Roads in 1928, and thereafter worked for 28 years on the design and construction of national park roads and national parkways in the East. During World War II he served 3 years as airport engineer with the Corps of Engineers in Alaska. After the war, he was Design and Construction Engineer for the Bureau's Philippine Division under the foreign aid program, and, subsequently, he became Regional Engineer for the Bureau's Region 15. In 1961 he was assigned as Regional Design Engineer of Region 9, from which position he retired in 1969. Mr. Cron was recalled from retirement in 1970 by the Federal Highway Administration to serve as field manager for the Alaska Transportation Corridor Study, and in 1974–75 was one of the authors of FHWA's Bicentennial History of Public Roads.

In recognition of his contribution to *Public Roads* and the Federal Highway Administration, Mr. Cron was awarded the Administrator's Award for Superior Achievement "for a notable addition to the technical history of motor highway design." This award is the highest honor given by the Federal Highway Administrator and it was presented to Mr. Cron on behalf of the Federal Highway Administrator by Dr. Gerald D. Love, FHWA's Associate Administrator for Research and Development, in Denver, Colo., on October 18, 1976.

1976 Guatemala Earthquake Damage to Highways and Bridges

by [1] James D. Cooper

ntroduction

The Guatemala earthquakes of February 4 and 6, 1976, :aused extensive damage to two major highway bridges: .a Asuncion located in the northeast part of Guatemala City ınd Rio Agua Caliente located approximately 17 miles (27 <m) northeast of Guatemala City on Route CA9. A third najor bridge, Incienso, located in the northwest section of Guatemala City suffered some damage. Other bridges vithin Guatemala City performed well, suffering little or 10 damage. Some evidence of displaced bearing pads, ettled aprons on abutment slopes, and minor impacting ıt abutments was evident on a few of these structures. 3ridge damage outside Guatemala City was generally conined to the area of approximate surface faulting along the Motagua River northeast of the city (fig. 1). Eleven bridges ılong the 184 miles (296 km) of Route CA9 between Guaemala City and Puerto Barrios on the east coast experinced varying degrees of damage, mostly to bearings.

he earthquake caused additional extensive damage, nostly to nonengineered structures along the Motagua River Valley. The approximate location of surface faulting left lateral), the shaded area in figure 1, follows the Motagua River and Route CA9, and extends approximately 49 miles (240 km) from just east of Gualan to northwest f Guatemala City. A maximum 55 in (1,397 mm) of fault isplacement was reported north of Guatemala City. (*1*) [2]

The National Earthquake Information Service, Denver, Colo., reported the February 4 earthquake as being 7.5 on the Richter scale with the epicenter at latitude 15.27° N, longitude 89.25° W, 99 miles (159 km) northeast of Guatemala City between the towns of Gualan and Los Amates. Reportedly, ground shaking in Guatemala City lasted between 20 and 30 seconds. Two seismoscope records were obtained and no strong-motion accelerograph records were recorded. The February 6 earthquake is reported as being a 5¾ to 6 magnitude event with a preliminary epicentral location of 14.3° N, 90.5° W. Most damage was caused by the February 4 event.

Inspections of damage to the Rio Agua Caliente, La Asuncion, and Incienso bridges, and to the bridges and roadway along Route CA9, the Atlantic Highway, were made by a reconnaissance team from the Earthquake Engineering Research Institute, Oakland, Calif., on March 1–6, 1976, 1 month following the earthquakes. (2)

This article is a condensation of the report "Bridge and Highway amage Resulting from the 1976 Guatemala Earthquake," Report No. HWA–RD–76–148, *Federal Highway Administration*, Washington, D.C., lay 1976.

Italic numbers in parentheses identify the references on page 107.

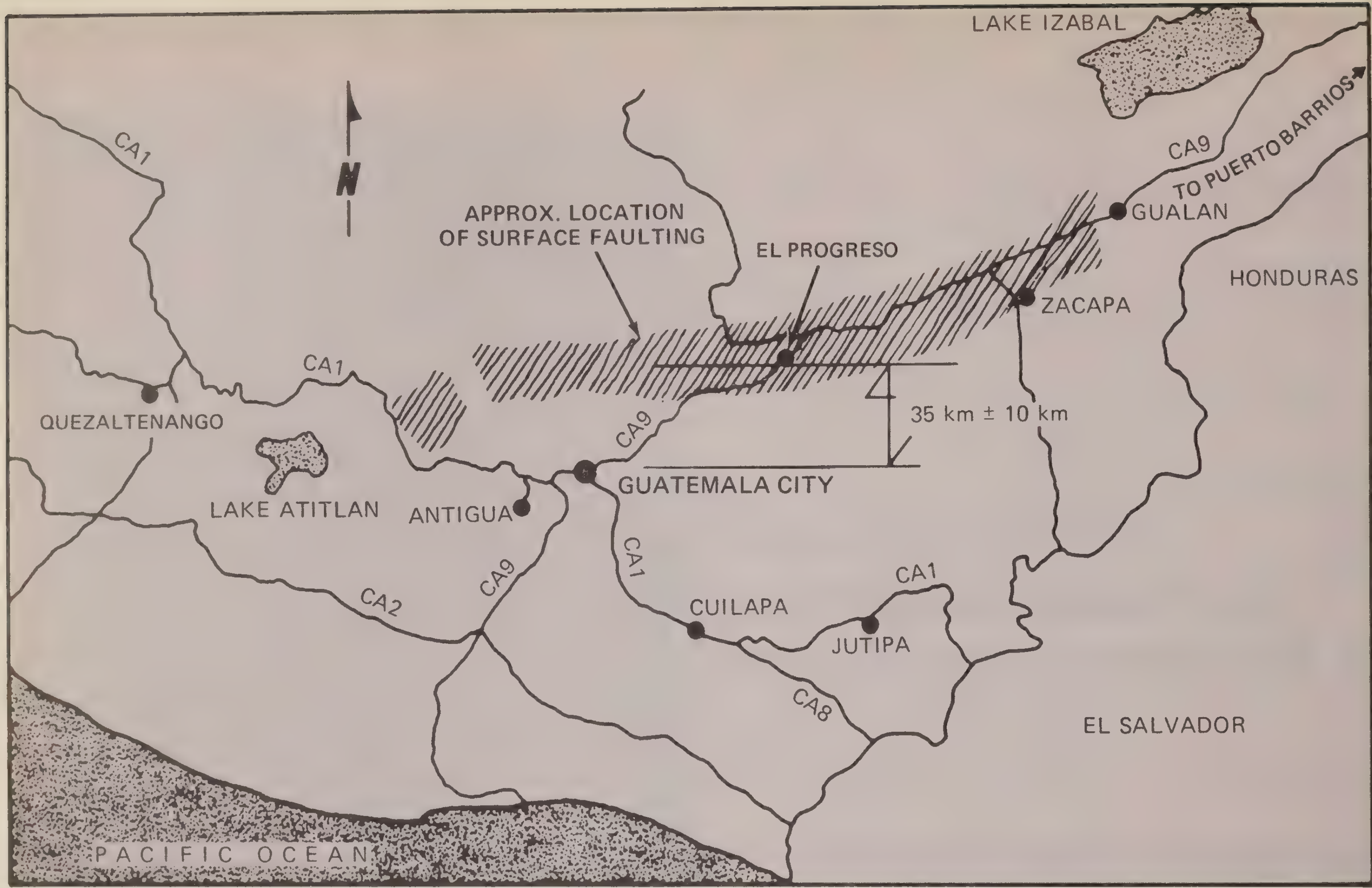


Figure 1.—Location map of Guatemala.

Rio Agua Caliente Bridge

The most severely damaged bridge was the important Rio Agua Caliente Bridge, located approximately 17 miles (27 km) northeast of Guatemala City on Route CA9, the Atlantic Highway. This highway serves as the major transportation link between Guatemala City and the economically strategic port at Puerto Barrios on the east coast of Guatemala.

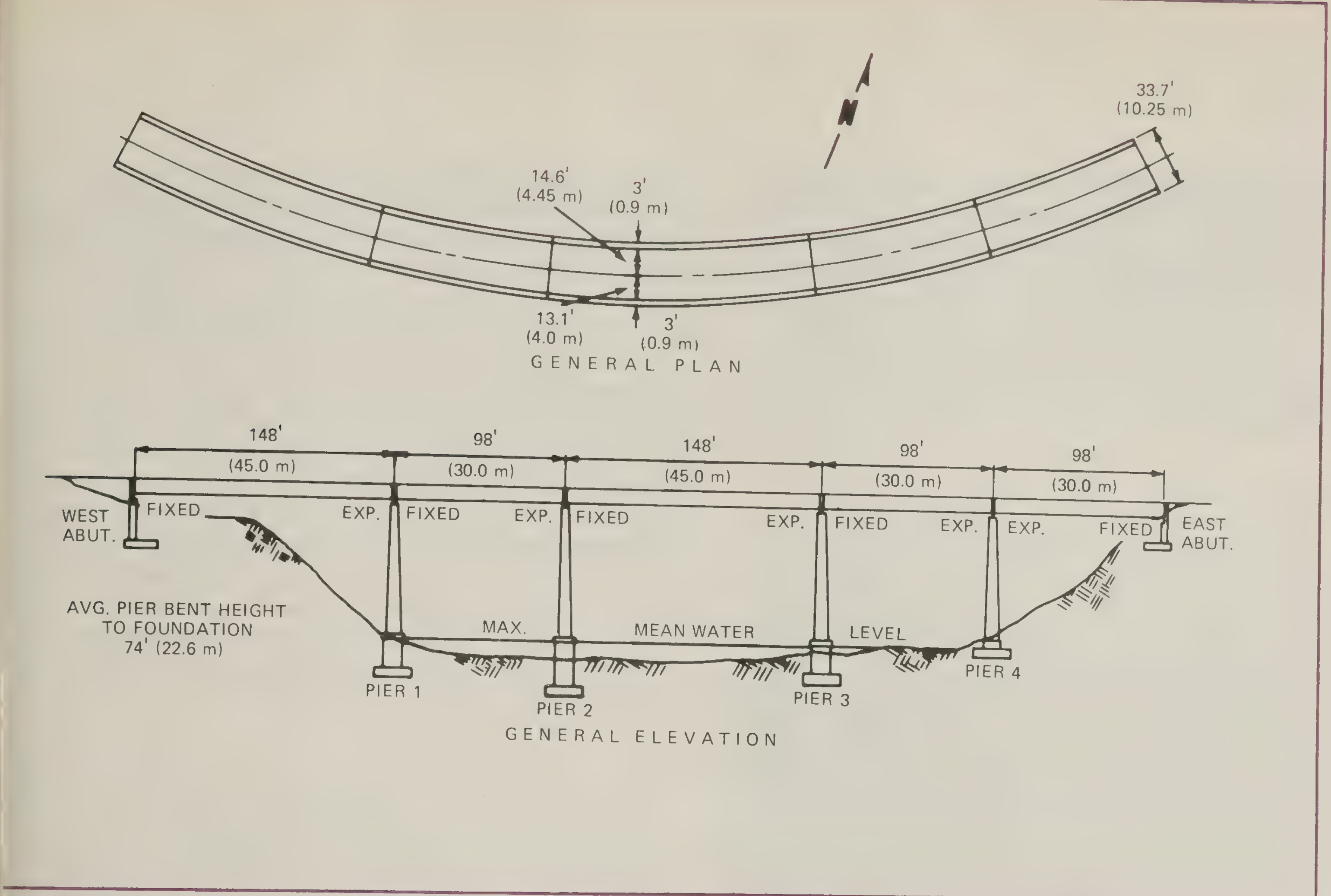
A simple plan and elevation view of the five-span, simply supported steel plate girder bridge is shown in figure 2. The bridge was designed in accordance with the 1953 AASHO Standard Specifications for Highway Bridges using an earthquake factor of 10 percent dead load. (3) The span lengths from the west abutment are 148 ft (45 m), 98 ft (30 m), 148 ft (45 m), 98 ft (30 m), and 98 ft (30 m).

The straight steel superstructure spans were designed using two 8-ft (2.4 m) deep, built-up exterior plate girders, two interior 18 WF 50 stringers (wide flange steel beams) with spirals for composite deck action, 30 WF 108 floor beams, and bottom lateral bracing. The concrete deck slab, curved in plan with 8.5 percent superelevation, is 6.5 in (165 mm) thick with a 2 in (51 mm) asphalt concrete overlay. Total deck width, including parapets, is 33.7 ft (10.3 m).

The superstructure rests on four reinforced concrete pier bents which vary in height between 72 and 76 ft (21.9 and 23.2 m) above maximum high water level. The pier bents are tied into pedestals which rest on spread footings. Total depths of pedestals and spread footings vary between 10 and 33 ft (3 and 10 m) below maximum high water level. The end spans rest on abutment walls on spread footings.

Figure 3.—Rio Agua Caliente Bridge—general view of collapsed spans.

Figure 3 shows a general view of the bridge from the north. The earthquake caused the three center spans to fall off the bearing supports. Pier bents and abutments appeared to be undamaged except for minor spalling where the girders had fallen against the columns and tie beams.



igure 2.—Rio Agua Caliente Bridge—general plan and elevation.

igure 4.—Rio Agua Caliente Bridge—displaced and fallen girders.

xtremely minor hairline cracks were noted around the ase of the columns of pier No. 2.

he fallen second and fourth spans appear to have initially ›st support on the south side of the bent, that is, along ıe outside radius of the deck curvature. As the south side f the superstructure fell, the bearings on the north side of ıe bent gave way allowing the spans to rotate down to ıeir final position.

Significant longitudinal pier motion occurred, propelling the girder of span No. 2 approximately 7 ft (2 m) beyond the centerline of pier bent No. 2. Pier rocking was evident because the soil at the base of pier No. 2 was compressed leaving approximately a $\frac{1}{2}$-in (13 mm) void between soil and foundation. This could account for approximately 1.5 in (38 mm) of longitudinal displacement about the centerline at the top of the pier bent, assuming simple rocking about the base of the foundation. The only visible damage to pier No. 3 was spalled concrete from girder impact above the lower tie beam on the left column and on the upper right side of the lower tie beam. In general, the pier bents and abutments appeared to be structurally undamaged and plans were being made to reuse them when the superstructure is rebuilt.

The bridge collapsed principally because of the bearing details used and the lack of longitudinal and transverse superstructure restraint at the supports. Figure 4 shows the fixed bearings in place atop the free standing piers and the twisted girders, stringers, floor beams, and lateral bracing of the collapsed spans. An expansion rocker can be seen resting on the tie beam in the foreground. The girder in the foreground appears to have walked approximately 5 in (127 mm) on the expansion bearing. Extension of anchor bolts was noted at fixed bearings on the abutments. If appropriate hinge restrainers had been in place across the expansion joints, thus tying the structure together, collapse could possibly have been avoided.

It is interesting to note that a curved railroad steel truss bridge located approximately 1 mile (1.6 km) north of the Rio Agua Caliente Bridge and oriented in the same direction, suffered no apparent damage. Although the railroad structure was not accessible for inspection, the passage of a freight train at the time of the Agua Caliente inspection showed that the structure was operational. The rails probably provided adequate structural continuity to resist the earthquake motions.

La Asuncion Bridge

The La Asuncion Bridge, a three-span, continuous plate girder bridge in the northeast section of Guatemala City, was severely damaged during the earthquake. The general plan and elevation views of the bridge are shown in figure 5. The span lengths are 148 ft (45 m), 196 ft (60 m), and 148 ft (45 m). The deck is 48 ft (15 m) wide. The piers are single, reinforced concrete columns with cap beams. The west pier is 86 ft (26 m) above grade and the east pier is 82 ft (25 m) above grade. Foundation type is unknown. A general view from the east abutment is shown in figure 6.

During the earthquake, the superstructure rotated, in plan, about the east pier causing the bearing supports to dislodge. The west end of the bridge deck displaced 39 in (991 mm) laterally to the south while the east end displaced 20.5 in (521 mm) laterally to the north (fig. 7) as measured from the once continuous median delineator on the approach roadway and deck. Bridge bearings again failed allowing the superstructure to rotate. The bridge deck dropped 8.3 in (211 mm) as it displaced and dislodged

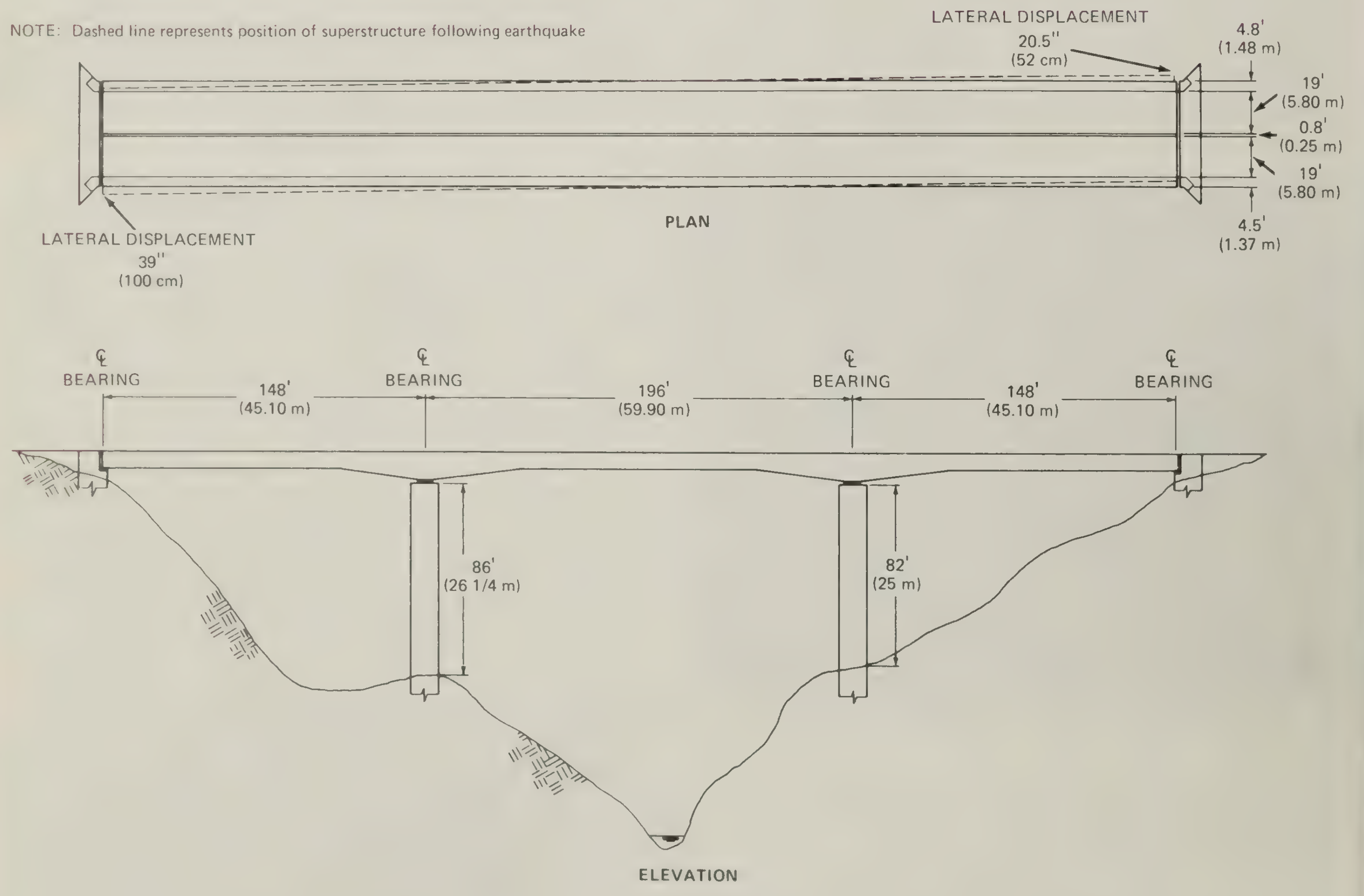


Figure 5.—La Asuncion Bridge—general plan and elevation.

from the rocker bars at the west abutment. The only noticeable damage to the steel superstructure was slight buckling of the bottom lateral bracing and end diaphragm at the west abutment.

The approach roadway itself was badly cracked, and damage to the west abutment and wing walls indicated the presence of extremely high backfill forces. The foundation at the west abutment was offset laterally 2.5 in (63.5 mm) relative to the south wing wall. The evidence indicated the complete failure of the west abutment. There was, however, no indication of foundation failure at the east abutment; and the piers, as viewed from both abutments, appeared undamaged.

The La Asuncion Bridge appears to be structurally sound and should be able to be jacked back into place once the damage to the west abutment has been corrected.

Incienso Bridge

The Incienso Bridge is located in the northwest section of Guatemala City (fig. 8). Constructed in 1974, 3 years after the disastrous earthquake in San Fernando, Calif., it incorporated the use of seismic hinge and abutment restrainers. The performance of this eight-span prestressed concrete bridge provided evidence that the use of new seismic design techniques in new construction can provide adequate structural resistance against major earthquakes. Although minor damage occurred, the bridge remained in service following the earthquake.

Figure 6.—La Asuncion Bridge—east abutment.

The three main spans were twin segmental, precast, prestressed concrete box girders, 203 ft (62 m), 400 ft (122 m), and 203 ft (62 m) long, and were constructed by the balanced cantilever method. The main span is supported on reinforced concrete piers 262 and 230 ft (80 and 70 m) high. The five side spans, each 92 ft (28 m) long, are constructed with seven prestressed concrete I-beams supported on reinforced concrete pier bents. The total deck width is 82 ft (25 m). The three main spans suffered no visible damage.

Figure 7.—La Asuncion Bridge—deck offset.

Figure 8.—Incienso Bridge.

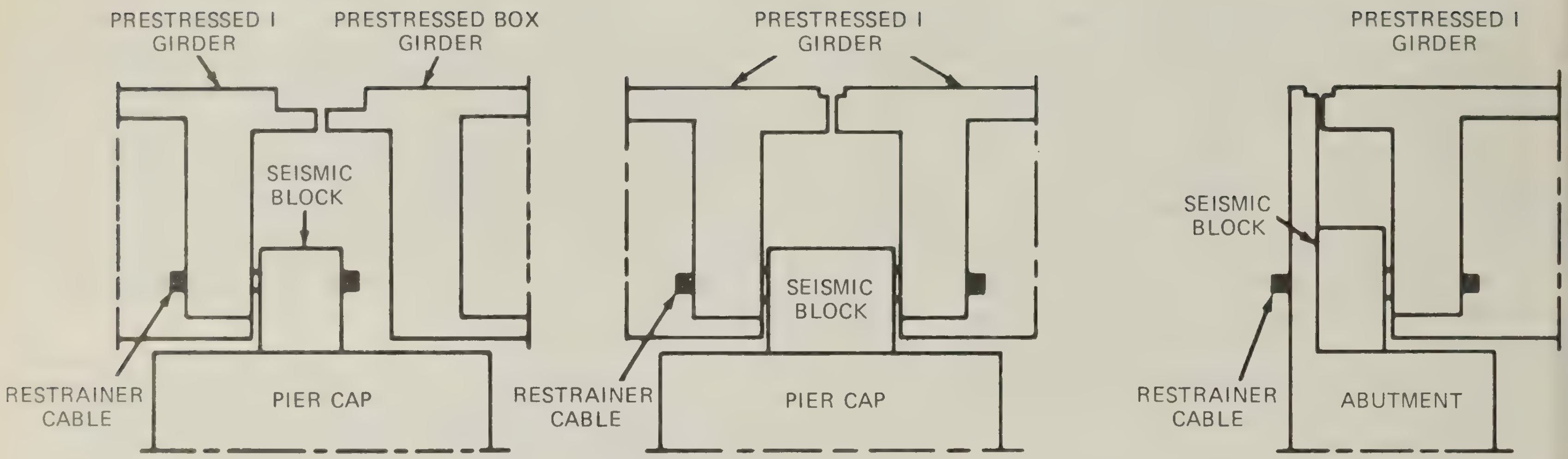


Figure 9.—Incienso Bridge seismic restraint.

The generally good performance of the bridge is attributed to the use of seismic restraint mechanisms, that is, hinge restrainers, which tied each line of prestressed I-girders together across the piers (fig. 9). A total of 12 restrainers were placed across each joint. Each restrainer cable was prestressed to 45 metric tons and had a 63 metric ton tensile strength. The only restrainer failure occurred across the west abutment, where all 12 cables failed in combination tension and shear. The failure of the restrainer cables at the west abutment resulted in the walking of the neoprene bearing pads.

Additional restraint was provided through the use of abutment tie back cables (fig. 10). It was reported that there were seven tie back cables across each abutment, although only five cable caps were noted across the abutment wall. Each cable was reported to have a tensile strength of 180 metric tons. There was no evidence of abutment tie back cable failure.

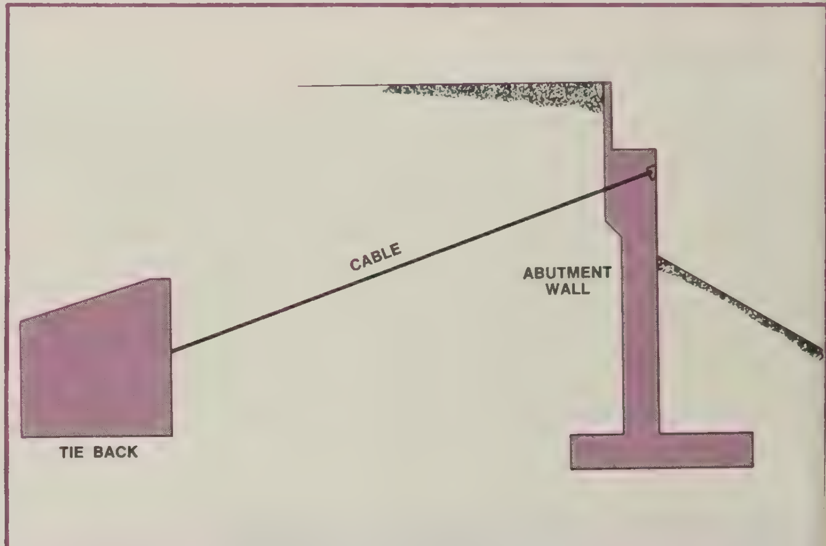


Figure 10.—Incienso Bridge abutment restraint.

The Atlantic Highway—CA9

The Atlantic Highway is one of the most economically strategic highways in Guatemala, extending from Guatemala City to the port at Puerto Barrios, 184 miles (296 km) to the northeast. The road was closed following the earthquake due to severe landslides, damaged bridges, and road sloughing.

Bridges

There are approximately 42 bridges along the Atlantic Highway, including the Rio Agua Caliente. Eleven suffered varying degrees of damage. Typical structures include deck, through, and pony trusses; steel girders with composite decks; and prestressed and reinforced concrete girder bridges. The predominant types are simple span trusses supported on rocker bearings. Excluding the Rio Agua Caliente Bridge, the most common damage was lateral displacement of the decks, impacting of deck against abutment walls causing cracking and spalling, and tipping or falling of rocker bearings.

Roadways

The Atlantic Highway is a two-lane asphalt roadway 22 ft (6.7 m) wide. Significant road sloughing occurred between Guatemala City and the Rio Agua Caliente Bridge site, typically on fill areas across steep erosional cuts. Damage was held to a minimum where retaining walls were constructed at the sides of the road. The most severe landslides, which closed the highway, occurred approximately 31 miles (50 km) northeast of Guatemala City covering approximately 6 miles (10 km). Future earthquakes can be expected to cause similar traffic disruption along CA9 because of the unfavorable terrain.

Figure 11.—Motagua fault.

Figure 11 shows the Motagua Fault Zone passing across the Atlantic Highway approximately 2 miles (3 km) northeast of El Progreso. The roadway across the fault zone was totally pulverized. There was a 39.4-in (1,000 mm) left lateral displacement at the fault as measured from the projection of the white centerline in the background of figure 11 to the white centerline in the foreground. There was no indication of vertical fault displacement at this site.

Conclusions

The Guatemala earthquakes vividly demonstrated the vulnerability of bearing assemblies commonly used to support typical highway bridges, not only in Guatemala but also in the United States. The types of bearings which failed have not been designed to withstand the large dynamic forces and motions imposed by earthquakes. While this type of bearing may be avoided in future construction in seismic risk areas, the crucial issue is what can or will be done to improve the seismic resistance of existing bridges which have this vulnerable detail. Recent research has identified retrofitting techniques which can reduce susceptibility to damage at a relatively small cost. *(4)*

The Guatemala earthquakes provided the first full-scale test of a structure whose design had apparently been modified to incorporate some of the lessons learned from bridge damage in the 1971 San Fernando, Calif., earthquake. The performance of the Incienso Bridge demonstrated that bridges which are appropriately designed and, in particular, tied together can withstand major earthquakes.

An attempt should be made to identify the numerous bridges which are located in seismically vulnerable areas of the United States. Importance and seismic vulnerability factors can be developed for each of these structures so that as funding becomes available, they can be retrofitted.

It is impossible to totally prevent landslides from occurring in areas of unfavorable terrain. However, critically important routes in rough terrain should be identified and plans made to provide either quick opening or alternate emergency routes following an earthquake.

Acknowledgments

The author wishes to express his appreciation and sincere thanks to Ingeniero Ernesto Argueta Pineda and his staff at the Guatemala Departamento de Puentes, Direccion General de Caminos, who provided valuable technical data and assistance during the inspection of damaged bridges. The author is also greatly indebted to the Earthquake Engineering Research Institute (EERI), Oakland, Calif., and Henry J. Degenkolb for providing names of key contact persons in Guatemala.

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1974 Brake Performance Levels for Trucks and Passenger Cars

by Paul A. Winter

A decline in braking performance of cars and trucks has been observed by the Bureau of Motor Carrier Safety, Federal Highway Administration. This article, a condensation of a final report (1),[1] describes brake testing on more than 1,500 passenger cars and trucks in common use on the highways of Maryland, Michigan, and California during the summer and fall of 1974. Measurements included the distance required for an emergency stop from an initial speed of 20 mph (32 km/h). The test corresponds with the braking performance tests established by the Federal and most State governments for in-use motor vehicles.

The article tabulates the braking performance for each of the various vehicle configurations tested, and compares the results measured in 1974 with the results of similar tests conducted in earlier years.

[1] Italic numbers in parentheses identify the references on page 115.

Introduction

Starting in 1941, the Bureau of Public Roads undertook a research program to determine, at periodic intervals, the brake performance levels of motor vehicles operating on the highway system in the United States. The studies were repeated in 1949, 1955, and 1963. In 1967 when the U.S. Department of Transportation was established, the Bureau of Public Roads was reorganized as the Federal Highway Administration (FHWA). The most recent series of tests was conducted in 1974 by the Bureau of Motor Carrier Safety (BMCS) within the FHWA.

The BMCS testing was done near the locations in Maryland, Michigan, and California which were used for the testing in 1949, 1955, and 1963. The information obtained from the series is expected to be used in the following ways: (1) promote improvement in the general level of brake performance; (2) serve as a basis for revising brake performance standards; (3) provide current motor vehicle brake performance data that can be used to establish highway design standards such as stopping sight distance; and (4) show different levels of actual brake performance for the different types of vehicles using the highways.

Scope of Research

With the assistance of State officials, BMCS conducted braking performance tests on 1,200 single-unit trucks and combination vehicles. For comparison, 366 passenger cars were also tested. All vehicles were selected at random from general highway traffic. For each, a complete description was recorded and three emergency stops were made from 20 mph (32 km/h). Drivers were advised the tests were voluntary and that no punitive action would be taken regardless of the stopping performance of their vehicles' brake systems.

The braking performance was measured in terms of brake system application and braking distance (BSABD)—that is, the distance traveled between the point at which the driver starts to move the braking controls and the point at which the vehicle comes to a complete stop. (The terms BSABD and "stopping distance" are used interchangeably throughout this article.)

Test Sites

In Maryland, the testing was conducted near Elkton on northbound U.S. 40, a four-lane divided highway. In Michigan, the testing was conducted at the Fowlerville Weigh Station on westbound I–96. In California, passenger cars were tested on a parking lot at the Del Mar Fairgrounds in Del Mar, and trucks were tested at the San Onofre Weigh Station on southbound I–5.

A single lane approximately one-quarter to one-half mile (0.4 to 0.8 km) long was established at each test site. The States' scale facilities were used to obtain axle weights of all commercial vehicles tested. Axle weights were not measured on passenger cars. Skid tests at each site showed that all test surfaces had similar frictional characteristics, the average coefficient of sliding friction being 0.65.

Instrumentation

The primary instrumentation was a test wheel equipped to measure speed and distance with accuracies of ±0.1 mph and ±0.1 ft (±0.16 km/h and ±0.03 m), respectively.

Test Procedures

In each State uniformed officers selected passenger cars from the flow of traffic. Test procedures were carefully explained and drivers preferring not to participate in the test continued their journey. Commercial vehicles were selected by test personnel at State weighing scales near the test sites. Axle weights were recorded as participating drivers drove their vehicles across the scales enroute to the test sites.

At the test sites, both passenger cars and commercial vehicles were photographed and inspected to insure that no vehicle defects which might cause the vehicle to be unsafe for testing were present and that all cargo items were adequately secured.

Once the test wheel and associated equipment were installed on a vehicle, the driver was informed of the exact test procedures. Each stop was made upon the observer's direction when the vehicle was as close as possible to the test speed of 20 mph (32 km/h). The driver then applied his brakes and maintained the vehicle's maximum braking capacity. After the vehicle had stopped completely, the observer recorded the brake system application and braking distance, the speed at which the vehicle was traveling when the stop was initiated, and brake pedal reserve or brake system air pressure remaining.

Data Analysis Procedures

In the analyses of test data, vehicles were classified by vehicle type, manufacturers' Gross Vehicle Weight Rating (GVWR), and actual weight of the vehicle measured at the time of the test. Braking performance results were compared with the performance requirements of the Federal Motor Carrier Safety Regulations (FMCSR) (2), the Uniform Vehicle Code (UVC) (3), and with the results of the previous studies. These comparisons are presented, primarily in the form of cumulative frequency curves (using smooth-line approximations), and percentile plots derived from these curves.

All analyses were done using standard statistical packages (4) and "canned" utility programs on an IBM 360 Model 65 computer.

Confidence Intervals

The classification of passenger cars represents samples of passenger cars being driven on public highways. In evaluating braking performance of the entire population of cars within each classification, the means for each sample classification were used to determine the interval in which the population mean could be expected to fall with some degree of confidence. The confidence interval selected was 95 percent, meaning if 100 samples were taken from the population, 95 of the sample means would be within the computed interval.

Passenger Car Results

Cumulative frequency distribution curves offer a convenient method for showing the braking performance of motor vehicles. Figure 1 compares the cumulative frequency distribution for passenger cars tested in 1974 with those for the earlier tests in 1955 and 1963. These curves show, for the test year indicated, the percentage of all passenger cars tested which can stop in a given distance or less. For example, 50 percent of the vehicles tested in 1974 could stop in 22.4 ft (6.8 m) or less. This represents an increase over the 50 percent levels in 1955 (20.0 ft—6.1 m) and 1963 (19.7 ft—6.0 m). (5)

Prior to 1975, the National Committee on Uniform Traffic Laws and Ordinances recommended in the UVC that all passenger cars stop in 25 ft (7.6 m) or less from a speed of 20 mph (32 km/h). The FMCSR still impose an identical requirement on passenger cars that are used for commercial purposes in interstate or foreign transportation. The data show that 87 percent of the passenger cars tested were capable of meeting this requirement. In 1963, 97 percent of passenger cars tested stopped within the 25 ft (7.6 m) requirement. (5) There are several factors which, when combined, can account for this decline in braking performance.

A significant increase in the weight of passenger cars since 1963 is perhaps the largest single factor contributing to increases in stopping distances. Using data from the National Automobile Dealers Association (6), 17 makes and models manufactured in 1963 which had readily identifiable counterparts still being manufactured in 1974 were examined. Without exception, the 1974 model weighed 8 to 28 percent more than the 1963 model, the average increase being 17.5 percent.

A second factor leading to increased stopping distances, particularly for full-sized passenger cars, is the increased use of vacuum booster (power) brakes. Eighty-nine percent of the full-sized passenger cars tested in 1974 were equipped with power brakes, while only 33 percent of those tested in 1963 were so equipped. (5)

The overall impact of vacuum boosters on braking performance of large passenger cars is favorable. Primarily, a vacuum booster allows reduced driver pedal effort, and enables greater braking capacity at high speeds. However, for the 20 mph (32 km/h) stops performed in this study, vacuum boosters have two drawbacks.

First, power brake systems take slightly longer from the instant the brake pedal is applied to achieve full braking effort at each wheel than do conventional hydraulic brake systems without vacuum booster assist. This slight increase in system response time contributes to a slightly increased stopping distance.

Second, vacuum boosters make it easier for drivers to apply the brakes hard enough to lock all four wheels. Maximum frictional force between a tire and the roadway is available at the point just prior to skid. If this condition of impending skid can be maintained during a stop, the shortest stopping distance will result. However, once a wheel actually begins to skid, available frictional forces are decreased, causing stopping distance to increase.

A final factor which has possibly contributed to increased stopping distances for passenger cars concerns the substantial conversion to disc brakes on the front wheels of passenger cars manufactured since 1963. A disc brake system tends to respond to brake pedal effort slightly faster than does a drum brake. Many manufacturers install disc brakes on front wheels and conventional drum brakes on rear wheels. With this configuration, it is necessary to install a metering device in the disc portion of the system to prevent front wheels from locking before the rear brakes apply. Overall, the metering device

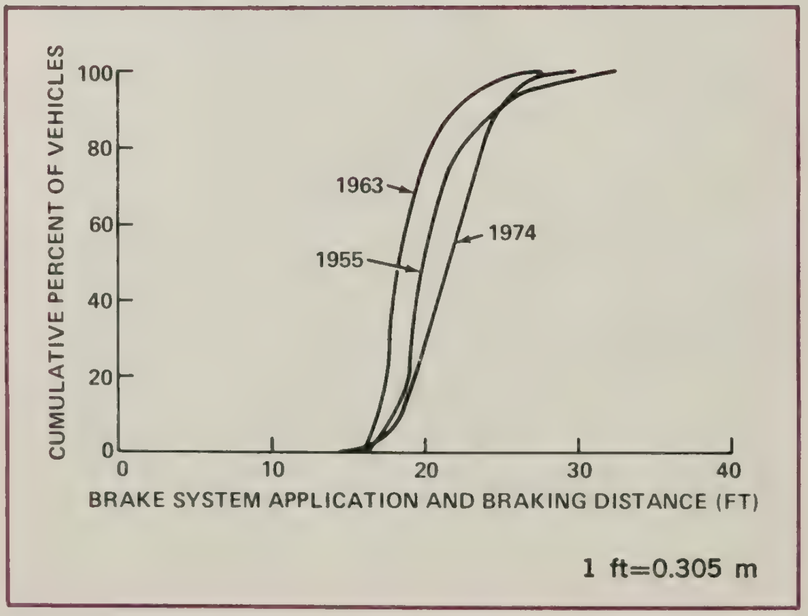

Figure 1.—Braking distance by year (passenger cars)—cumulative frequency distribution. (5)

allows for smoother braking and improved vehicle control. In the case of a panic stop from 20 mph (32 km/h), however, the metering device causes a slight delay in front brake application time with a corresponding increase in stopping distance.

It should be emphasized that the stopping distance required for a 20 mph (32 km/h) panic stop is only one measure of a passenger car's overall braking performance. The degradation observed between the 1974 test and earlier tests in this measure may be more than offset by improvements in the other aspects of passenger car brake performance previously mentioned.

Truck Results

In an ideal situation, all motor vehicles using the highways would be capable of stopping within approximately the same distance from a given speed. Unfortunately, stopping distance performance of trucks has always lagged considerably behind that of passenger cars. This lag is compensated for partially by the forward-sight advantage in trucks due to the driver's position above the roadway. It is unlikely that equal braking performance for cars and trucks will be achieved in the foreseeable future.

From a brake design standpoint, trucks present engineering problems not found in passenger cars. For example, a typical tractor-semitrailer combination (three-axle tractor, two-axle semitrailer) weighs less than 25,000 lb (11,340 kg) when empty, and can legally be loaded in most States to 73,280 lb (32,240 kg). This represents a weight variation of about 300 percent from the empty condition to the fully loaded condition. The brake system for this type of vehicle must be sufficiently aggressive to enable acceptable emergency stopping performance when fully loaded, and yet not be so aggressive that smooth stops, without locking wheels, are impossible when the vehicle is empty.

Table 1.—Comparison of test results with FMCSR and UVC stopping distance requirements

Vehicle type	FMCSR requirements (20 mph)	Vehicles within FMCSR requirements (1974)	UVC requirements (20 mph)	Vehicles within UVC requirements: 1955 tests (6)	1963 tests (6)	1974 tests
Single units	*Feet*	*Percent*	*Feet*	*Percent*	*Percent*	*Percent*
2-axle≤10,000 lb	25	69	30	84	97	88
2-axle>10,000 lb	35	75	40	84	95	89
3-axle	35	29	40	53	75	64
Combinations						
2–S1	40	80	50	81	97	100
2–S2	40	78	50	80	94	97
3–S2	40	65	50	64	92	96
3–S3, 3–S4, 3–S5, 3–S7, and 3–S8	40	55	50	—	—	95
2–1, 2–2, 2–3, 3–2	40	48	50	38	86	90
Twin trailer combinations	40	44	50	41	71	74

1 mph=1.6 km/h
1,000 lb=454 kg

The maximum weight variation for a passenger car would be more on the order of 50 percent, as in the case of an intermediate-sized car weighing 2,600 lb (1,180 kg) loaded with six, 150-lb (68 kg) passengers and 400 lb (180 kg) of luggage.

Most States have adopted the braking performance requirements found in the UVC for the various truck categories. The UVC serves as a guide for all States in determining their respective motor vehicle codes. Trucks operated in interstate or foreign commerce must also comply with the stopping distance requirements in the FMCSR.

When the tests were last conducted in 1963, the stopping distance requirements in the FMCSR were identical to those in the UVC. However, because of the large percentage of vehicles capable of stops shorter than those required by the UVC in 1963, the FMCSR were made more stringent effective on July 1, 1972.

In 1975, after the test work for the final report was complete, the UVC requirements were modified to specify an upper limit of 40 ft (12 m) from 20 mph (32 km/h) for all vehicles. This limit corresponds to the FMCSR requirements for combination vehicles.

Table 1 shows the FMCSR stopping distance requirements from 20 mph (32 km/h) for the various truck categories, and the percentage of each vehicle type tested in 1974 which could meet or exceed its respective requirement. A similar comparison is shown for the pre-1975 UVC 20 mph (32 km/h) stopping distance requirements for the tests conducted in 1955, 1963, and 1974. (5)

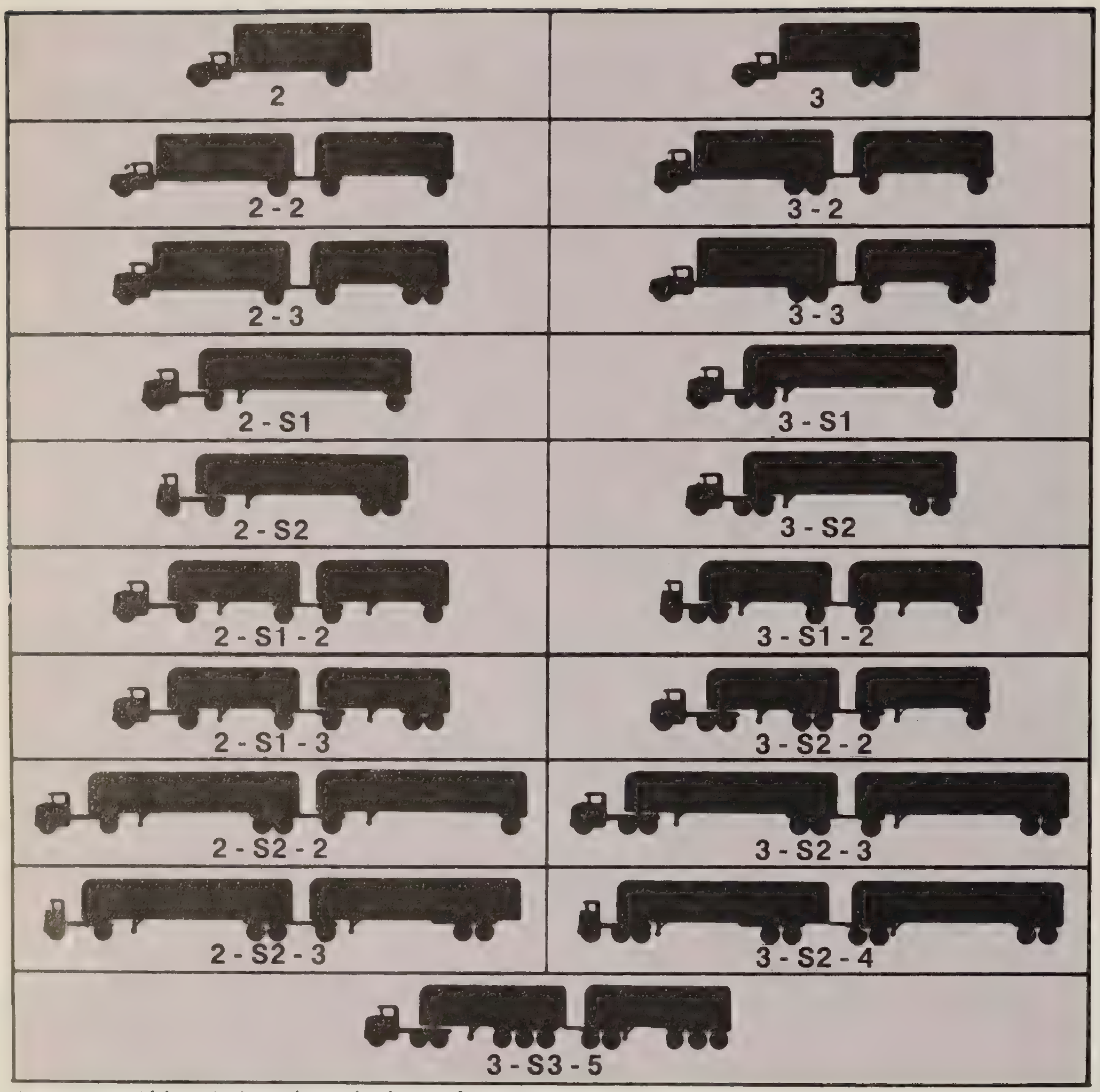

Figure 2.—Abbreviations for vehicle configurations.

Figure 2 explains the notation used in the study to represent various vehicle configurations. For example, 3–S2 refers to a three-axle power unit (3) pulling a semitrailer (S) having two axles (2); the last digit in a three-vehicle combination code, such as 3–S2–2, refers to the number of axles on a full trailer.

As can be seen in table 1, the truck configuration which performed the worst in meeting either the FMCSR or the UVC requirements was the three-axle single unit. While the performance of two-axle single units also declined, the three-axle single unit showed the largest degradation in performance since the 1963 testing.

Combination vehicle configurations showed modest improvements since 1963 with respect to the UVC requirements, particularly the "twin trailer" combinations (truck tractor pulling a semitrailer and a full trailer). It should be noted, however, that the performance of the trucks tested in 1974 with respect to the FMCSR requirements leaves substantial room for improvement, particularly for twin trailer combinations and straight trucks pulling full trailers.

The fact that the majority of vehicles within each vehicle configuration tested in 1974 can meet the respective FMCSR requirements (more than 50 percent of the three-axle single unit vehicles tested in 1963 could do so) indicates that the FMCSR requirements are reasonable and well within manufacturers' design capabilities. However, a large percentage of vehicles within each vehicle configuration failed to meet the respective FMCSR requirements. This suggests that increased attention should be paid to brake system maintenance by motor carriers.

Figure 3 shows the trend in braking performance for various truck configurations since testing began in 1941. The 15th, 50th, and 85th percentile performance levels are indicated for each truck configuration. In each test year, 15 percent of the vehicles tested of a given configuration could stop in the distance indicated by the bottom line or less; 50 percent could stop within the distance bounded by the middle line; and 85 percent within the distance shown by the upper line.

Figure 3 shows a degradation in braking performance in at least two of the three percentile levels for every truck configuration except 2–S2 and 3–S2. The 2–S2 and 3–S2, which are two of the most widely used vehicle configurations, both demonstrated a slight improvement at the 85th percentile level since 1963, remained constant at the 50th percentile, and deteriorated somewhat at the 15th percentile.

In the passenger car portion of this article, several vehicle design changes which have been implemented since 1963 were cited that could have contributed to somewhat increased stopping distance for passenger cars from 20 mph (32 km/h). There have been no corresponding changes in truck design during this period which could account for the increases in observed truck stopping distance.

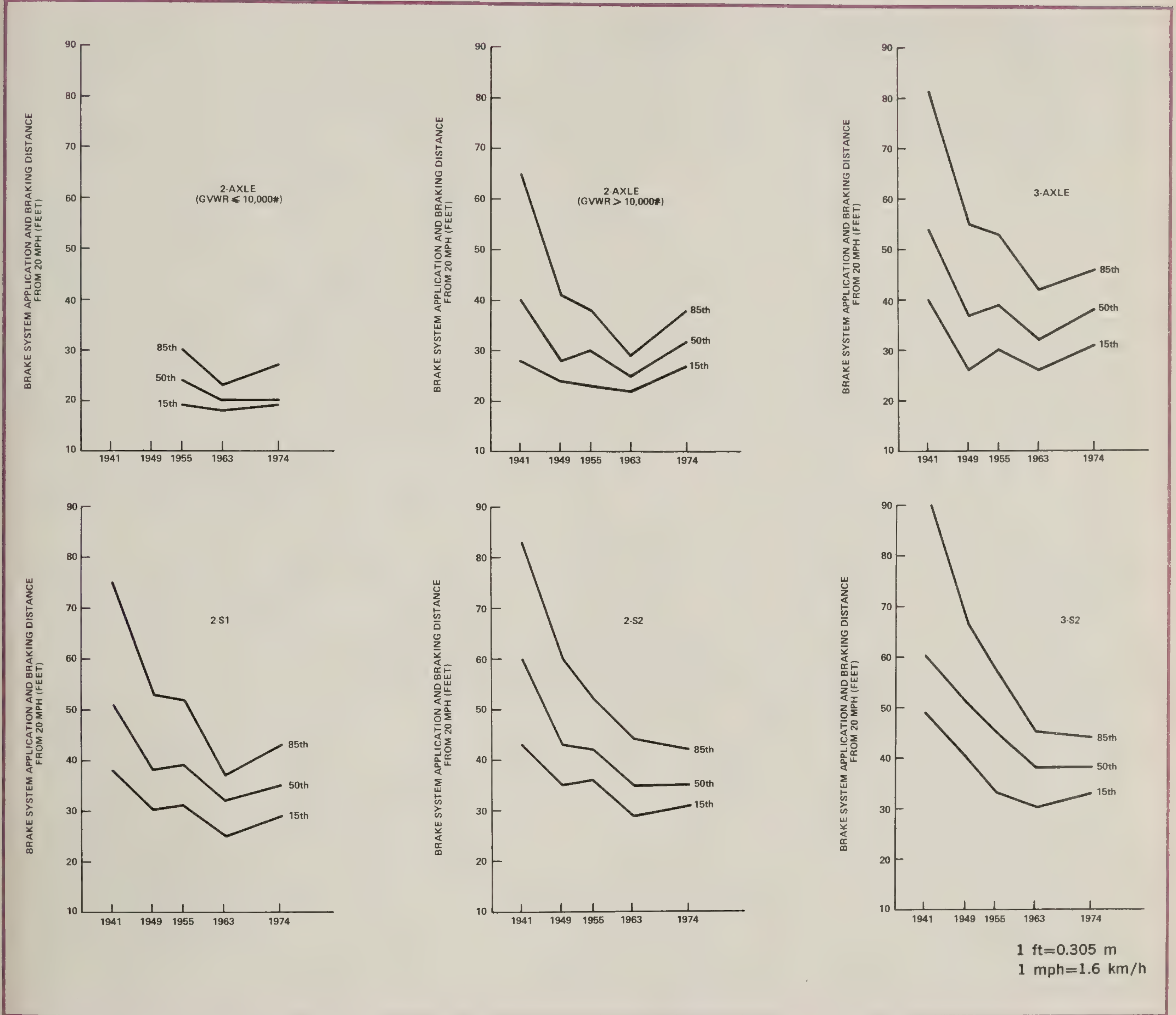

Figure 3.—Percentile levels of brake performance by year. (5)

Effect of Brake System Maintenance on Stopping Distance

In brake testing conducted during the late 1940's and early 1950's (7), the Bureau of Public Roads investigated the effect of maintenance on stopping distance. A total of 63 heavy duty commercial vehicles were selected from large operating fleets. The braking performance of these vehicles was measured, both in the "as received" condition and after needed maintenance and repair work had been done.

A simple brake adjustment on these vehicles resulted in a weighted average reduction in stopping distance from 20 mph (32 km/h) of 9.0 percent. Completion of all other adjustments and repairs resulted in a total weighted average reduction in stopping distance of 15.3 percent. In other words, almost 59 percent of the total improvements that could be made in the stopping performance of a typical vehicle resulted from merely adjusting the brakes.

VEHICLE TYPE	WEIGHT RANGE 1000's OF #	NUMBER TESTED	BRAKING DISTANCE RANGE (FT) 10 20 30 40 50 60 70
2-AXLE ≤ 10,000 LBS	4.3-10.0 (Av=8.3)	132*	18.5 Av=25.6 49.5
2-AXLE > 10,000 LBS	10.0-39.0 (Av=16.0)	279	21.0 Av=32.7 64.3
3-AXLE	14.3-63.4 (Av=31.6)	45	25.7 Av=39.3 57.8
2-S1	10.7-44.3 (Av=27.1)	62	23.3 Av=35.8 49.0
2-S2	19.3-63.9 (Av=32.5)	146	20.7 Av=36.4 80.0
3-S2	21.5-78.4 (Av=44.3)	427	25.9 Av=38.4 75.3
2-S3, 3-S1	19.9-71.8 (Av=32.3)	9	27.4 Av=37.0 46.7
2-1, 2-2, 2-3, & 3-2	15.4-79.3 (Av=34.8)	29	30.6 Av=42.5 60.8
3-S3, 3-S4, 3-S5, 3-S7, & 3-S8	29.9-137.1 (Av=76.4)	20	28.5 Av=40.3 54.9
TWIN TRAILER COMBINATIONS	23.3-188.4 (Av=67.8)	51**	30.3 Av=43.8 83.1

* - INCLUDES 98 2-AXLE VEHICLES FOR WHICH GROSS WEIGHT IS NOT AVAILABLE.

** - INCLUDES 3 SPECIAL PERMIT VEHICLES IN MICHIGAN WHICH EXCEEDED THE 136,000 LB GROSS WEIGHT LIMIT FOR 11-AXLE COMBINATIONS.

▼ - DENOTES AVERAGE

1,000 lb=454 kg

1 ft=0.305 m

Figure 4.—Summary of weight and braking distance observations by vehicle type.

Weight and Distance Observations

Figure 4 summarizes the number of vehicles tested, the weight range and average weight within the grouping, and the braking distance range and average braking distance for each commercial vehicle type. For example, 427 vehicles of the 3–S2 configuration were tested. These vehicles had gross combination weights ranging from 21,460 to 78,400 lb (9,734 to 35,562 kg), with an average weight of 44,260 lb (20,076 kg). The stopping distances for the group ranged from 35.9 to 75.3 ft (10.9 to 23.0 m) with the average vehicle stopping within 38.4 ft (11.7 m).

Summary of Findings

The 1974 test results showed a decline since 1963 in braking performance from 20 mph (32 km/h) for passenger cars and most truck categories.

The decline for passenger cars can be attributed primarily to weight increases and brake system design changes since 1963.

Single unit trucks, as a group, showed the largest decline from the 1963 braking performance levels. Most categories of combination vehicles (for example, tractor-semitrailers) either declined or remained essentially the same in braking performance during this period. Large percentages of commercial vehicles on the highways are not capable of meeting the applicable stopping distance requirements from 20 mph (32 km/h) specified in FMCSR. It is believed the majority of these vehicles could meet or exceed the FMCSR requirements if properly maintained.

Conclusions and Recommendations

It must be concluded that a general deterioration has been allowed to occur since 1963 in the braking performance of commercial motor vehicles, the first such deterioration observed since the testing began in 1941. This deterioration is most pronounced, about 27 percent, for two-axle single unit vehicles having a GVWR in excess of 10,000 lb (4,536 kg). The braking performance of three-axle single unit vehicles and 2–S1 combination vehicles showed a degradation of about 14 percent since 1963. The braking performance of 2–S2 and 3–S2 combination vehicles has remained essentially constant.

A careful review of the variables which could have caused the measured increases in stopping distances suggests that the quality of brake system maintenance is responsible. Accordingly, it is recommended that motor carriers devote increased effort to brake system maintenance, particularly to brake adjustment. Likewise, it is recommended that Federal and State vehicle inspection programs put increased emphasis on activities dealing with brake system maintenance and adjustment.

Finally, it is recommended that tests similar to those conducted in 1974 be repeated every 5 years for two reasons:

- To determine whether the negative trend observed in 1974 has been reversed.
- To determine the effect on vehicles-in-use of Federal Motor Vehicle Safety Standard No. 121 which became effective for new air-braked vehicles shortly after the test work for this report was completed.

Acknowledgments

Federal Highway Administration personnel who contributed to the success of this project include Donnell W. Morrison, Asriel Taragin (now retired), James Bremner, Walter Hannigan, Lynn J. Runt, Thomas Kelley, Dane Ismart, Ronald Ashby, William B. Leysath, Darwin D. Garvin, and Harry Krashen.

The following State organizations provided both personnel and material assistance without which the field research could not have been successfully accomplished: Maryland Department of Transportation; Maryland State Police; Michigan Department of State Highways and Transportation; Michigan Public Service Commission; Michigan State Police; and Department of California Highway Patrol.

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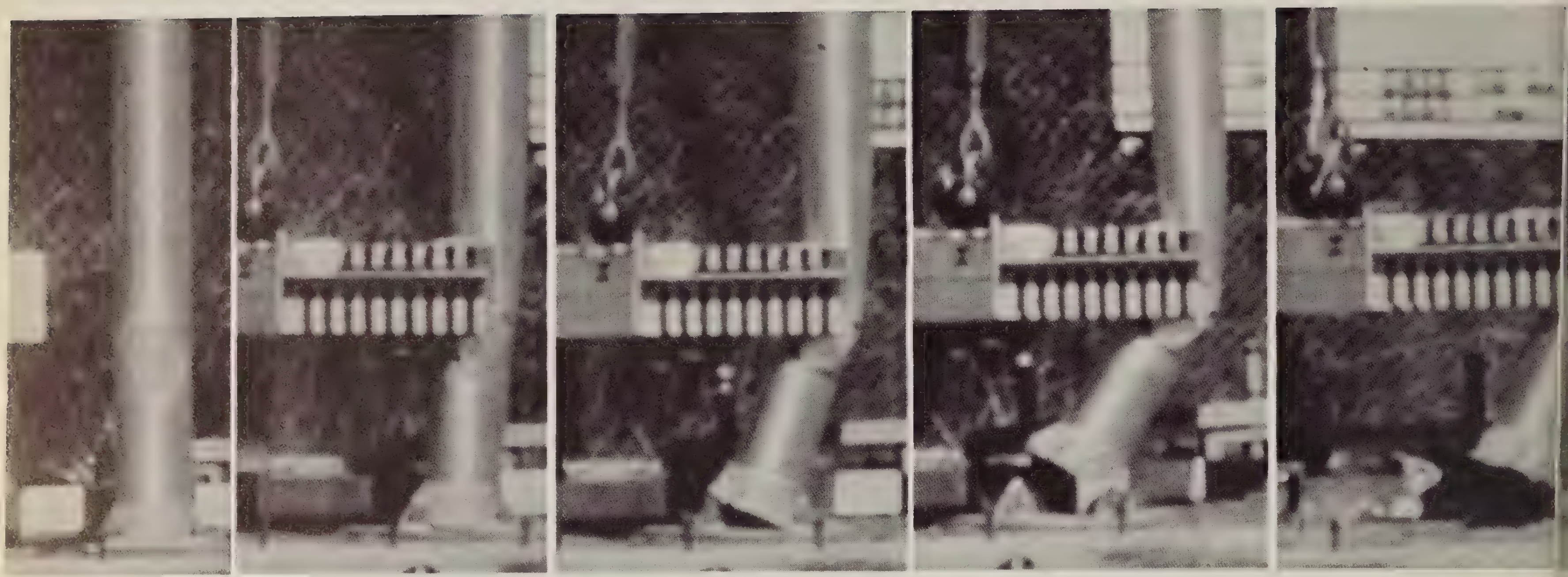
ERRATA

In the article "Highway Design for Motor Vehicles—A Historical Review. Part 7: The Evolution of Highway Grade Design," in the September 1976 issue of *Public Roads*, vol. 40, No. 2, the vertical and horizontal axes in figure 3, p. 84, and the vertical axis in figure 4, p. 85, were not included.

The vertical axis in figure 3 should read "SPEED UPGRADE—MILES PER HOUR" with a scale of 0 to 45 by 5 mph increments. The horizontal axis should read "DISTANCE UPGRADE—FEET" with a scale of 0 to 3,500 by 250-ft increments.

The vertical axis in figure 4 should read "SPEED ON GRADES—MILES PER HOUR" with a scale of 0 to 50 by 5 mph increments. The value at which the lines begin at the vertical axis is 47 mph.

The two figures, reproduced in their entirety, are available upon request from *Public Roads* Magazine, HDV–14, Federal Highway Administration, Washington, D.C. 20590.



The Development of Acceptance Test Criteria for Frangible Sign and Luminaire Supports

by Douglas B. Chisholm

The frequency and severity of single vehicle ran-off-the-road accidents involving rigid sign and luminaire supports has prompted a Federal Highway Administration research program to evaluate the impact safety performance of these structures. The program features new testing procedures, analytical simulations to predict impact performance, and a new impact test facility using a crushable impact face with a variable mass pendulum.

Accidents involving signs, luminaires, and utility poles account for a high percentage of fixed object collisions in which a vehicle runs off the road. Every State has some compilation of fixed object collision data, and these represent about one-third of all fatal crashes in the Nation. (*1*) [1]

Recognizing the need to design highway structures with lower impact severity, the Federal Highway Administration's (FHWA) Office of Traffic Operations prepared a memorandum giving impact test standards for breakaway luminaire supports. (2) The memorandum stated that a luminaire support that yields or breaks away causing a change in vehicle momentum of 1,100 lb-s (4.893 kN-s) or less was considered to possess acceptable breakaway features. The implied test procedure involved a full-scale test of a 4,000-lb (17.792 kN) vehicle at 40 mph (64 km/h).

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[1] Italic numbers in parentheses identify the references on page 120.

This 1,100 lb-s (4.893 kN-s) momentum change corresponds to a 6-mph (9.7 km/h) change in velocity for a 4,000-lb (17.792 kN) vehicle. Available facts suggested 11 mph (17.7 km/h) as a threshold of injury for head-on collisions with standard weight vehicles involving rigid obstacles.

Laboratory tests were thought likely to produce a more consistent measure of pole resistance to vehicle impact than the costly full-scale vehicle impact test. To allow a pendulum test to substitute for a full-scale test, an interim acceptance level of 400 lb-s (1.780 kN-s) change in momentum of the 2,000-lb (8.896 kN) impacting weight was set forth when tested at 20 mph (32 km/h) impact speed and 20-in (510 mm) striking height. (3)

This implied that a pole producing a 1,100 lb-s (4.893 kN-s) change in vehicle momentum in a full-scale test would produce a 400 lb-s (1.780 kN-s) momentum chan in a rigid mass laboratory dynamic test. The difference is due, in part, to vehicle crush characteristics.

The shortcomings of these criteria have become obvious. Research by FHWA's Office of Research, Structures and Applied Mechanics Division, has shown that in addition t the overly simplified test conditions in the two criteria, there are other factors contributing to vehicle momentum change during impact (*4*):

Figure 1. A.—The side of a Datsun 240Z following an impact with a motorist aid call box structure. B.—The pole base and foundation is torn from the soil. C.—Note the short 2.5-ft (0.77 m) foundation. D.—The remaining pole segment.

- Vehicle velocity at impact must be specified since change in velocity is a function of initial velocity.

- Vehicle crush stiffness affects change in velocity—the lower the stiffness, the greater the deformation needed to reach the level of force required to activate the breakaway mechanism. As the vehicle crushes, it absorbs kinetic energy, thereby slowing the vehicle. Vehicle side structures are typically one-third to one-half as stiff as the frontal structure.

- Soil mounted signs, luminaires, or other dynamically similar structures behave differently than rigidly mounted structures used in full-scale vehicle and laboratory tests, especially in small structures when foundations are displaced during impacts.

- Scatter in the test data indicates the need for more than one test.

- Mechanisms such as striking blocks that distribute the impact load over a large target area cause more rapid loading than would occur in service.

- Orientation of target structure to the point of force application is critical since premature buckling may occur in one direction and not in another.

- Large breakaway structures can prove less resistant on impact than small breakaway structures if the peak force required to activate the breakaway mechanism in the small structure exceeds the soil-bearing capacity for lateral dynamic loads. This may be a problem for small structures such as call box support foundations designed only to resist wind and ice loads which are much lower than impact loads (fig. 1).

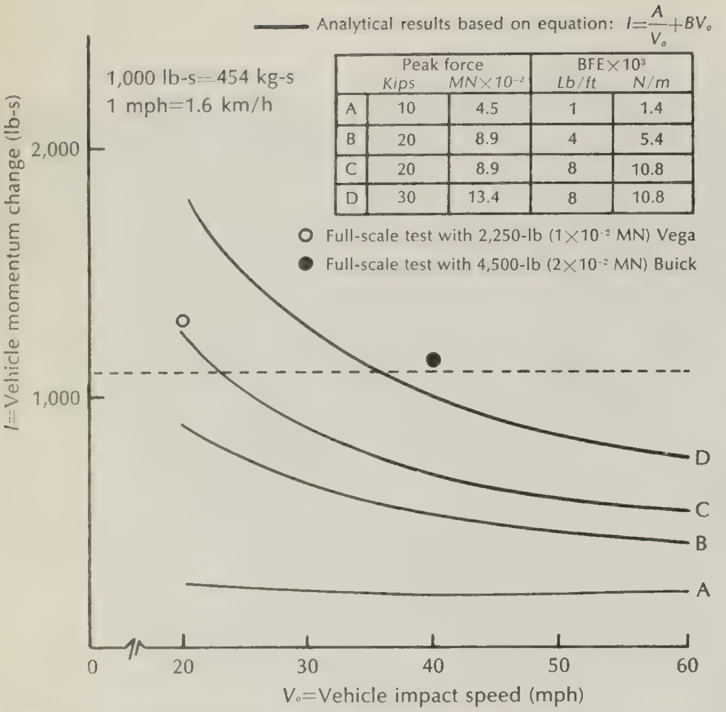

Figure 2.—Vehicle impacts with luminaire supports—8-in (203 mm) diameter shaft, 30-ft (9.22 m) height, 0.188-in (4.77 mm) wall, cast 356–T6 shoe base—manufactured by HAPCO, Inc.

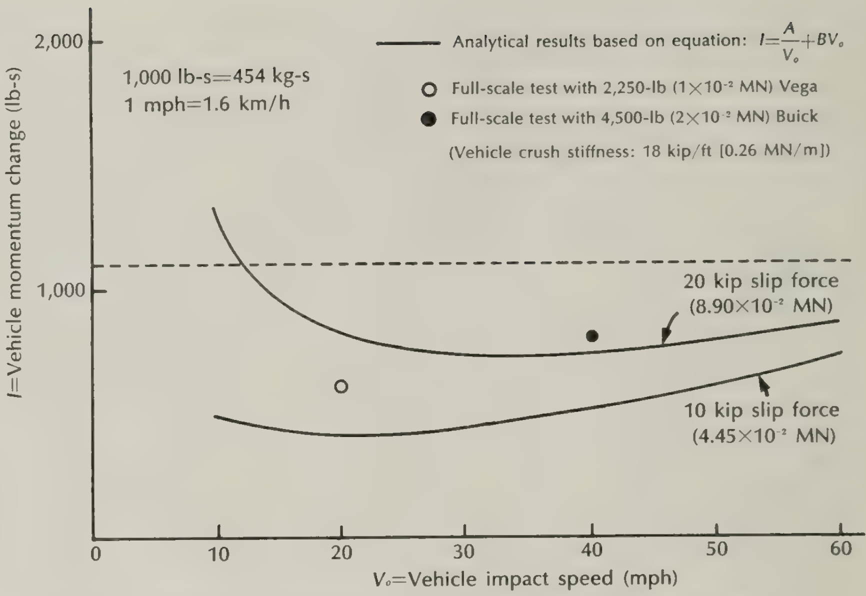

Figure 3.—Vehicle impacts with slip base sign—12WF45 legs.

$$I=M\,\Delta V=\frac{-\beta\omega F^2}{2KV_o}-\frac{8}{3}\,\frac{(BFE)}{\gamma V_o}-\frac{M_p\alpha R^2}{R^2+D_o^2}\,V_o$$

$$=\frac{A}{V_o}+BV_o$$

Where,

$$A=-\frac{\beta\omega F^2}{2K}+\frac{8}{3}\,\frac{(BFE)}{\gamma}$$

and

$$B=-M_p\,\alpha\,\frac{R^2}{R^2+D_o^2}$$

Definition of symbols:

M_p=Mass of the impacted structure—lb (N).

R=Radius of gyration of impacted structure about center of gravity—ft (m).

D_o=Moment arm of the impact force about the center of gravity of the impacted structure—ft (m).

α, γ, ω=Empirical factors to account for velocity change during each of three impact phases.

β=An empirical factor to account for nonlinear vehicle crush behavior. In all cases this factor is nearly equal to unity as found by full-scale tests.

F=The peak force required to initiate breakaway mechanism in the pole—lb (N).

K=Vehicle crush stiffness (averaged as a single parameter)—lb/ft (N/m).

V_o=Impact velocity—ft/s (m/s).

BFE=Breakaway impact energy, or energy required to produce full separation of breakaway base mechanism—lb/ft (N/m).

I=Momentum change of impacting vehicle in lb-s (N-s).

This simplified expression, which agrees fairly well with more sophisticated simulation results, shows that the change of vehicular momentum during impacts with sign supports, luminaires, or utility poles can be characterized by two components.

In the first component, vehicle momentum change (currently the measure of acceptance of the test article) is *inversely* related to vehicle velocity. In the second, vehicle momentum change is *directly* proportional to vehicle velocity. Returning to the first component, when the value of the impact velocity is low, a large vehicle momentum transfer can be expected, especially if A is larger than B.

Most aluminum luminaire supports are in this category, that is, low-speed collisions are significantly more severe than high-speed collisions as shown in figure 2. With heavy sign structures—support mass weighing >45 lb/ft (656 N/m)—as in figure 3, the reverse may be true if the signs are of the slip-base breakaway type. The open circles in each figure denote full-scale test results. The momentum change values are based on accelerometer data.

The previous equations show that structures such as sign supports, luminaires, and utility poles all have a characteristic impact velocity at which the vehicle momentum change is minimized. That velocity is given by:

$$V_m = (A/B)^{1/2}$$

For years luminaire acceptance tests have been conducted at impact speeds (40 mph—64 km/h) which produced momentum transfers considerably lower than would be expected at lower speeds such as 20 mph (32 km/h). For aluminum supports ranging in height from 30 to 45 ft (9.1 to 13.7 m) and weighing from 170 to 350 lb (756 to 1,557 N), the variation in V_m is not rapid from 35 to 60 mph (56 to 97 km/h). For sign supports (weighing in excess of 600 lb—2.66 kN) the opposite trend is seen; that is, as the vehicle impact velocity increases, the momentum change increases.

This oversimplifies the problem but illustrates a key point: a single impact velocity cannot adequately reflect the impact performance of sign supports, luminaires, or utility poles. Other factors are important. One, as shown by Owings and Adair (6), is the peak force, F, required to activate a breakaway device. The constant A is seen to depend on this force squared. For example, a shoe base luminaire support with a 30-kip (0.1334 MN) peak force requirement will result in an impact nine times more severe than a slip base or breakaway coupling having a peak force requirement of 10 kips (0.0445 MN).

The American Association of State Highway and Transportation Officials (AASHTO) has set restrictive criteria for breakaway roadside structures including sign support structures and luminaire supports. (7) The new AASHTO criteria require that a 2,250-lb (10 kN) vehicle impacting such structures at velocities between 20 and 60 mph (32 and 96 km/h) suffer not more than 1,100 lb-s (4.893 kN-s) change in momentum.

The impact test criterion is eight times more severe than the one given in reference 3 on the basis of kinetic energy available to activate the device, yet fails to identify test procedures. Procedures are expected to be outlined soon defining test conditions which incorporate the AASHTO criteria and guidelines for interpreting the results.

The simulation work of Edwards et al. (5) and Owings and Adair (6) verifies that as the mass of the target structure increases, so does the severity of impact at higher speeds. The component of the momentum change equation which dominates the result for such structures at high speeds is the BV_o term (recalling that $I = M\,\Delta\,V = \frac{A}{V_o} + BV_o$).

For luminaire supports weighing under 300 lb (1.334 kN), lower velocity (40 mph—64 km/h) impacts are critical for lightweight vehicles. For large sign structures with support weights from 450 to 650 lb (2.000 to 2.900 kN), velocities greater than 40 mph (64 km/h) at impact can result in a change in vehicle momentum greater than 1,100 lb-s (4.893 kN-s) even for optimally designed slip bases. This distinction in dynamic performance will need to be considered for the popular, large diagrammatic signs.

Research by Ensco, Inc., in an FHWA contract, "Safer Sign and Luminaire Supports," (8) clearly shows the need for criteria which take target mass into account. It is conceivable that a lightweight vehicle could impact a 12WF45 (658 N/m) breakaway sign support at 20 mph (32 km/h) and pass the AASHTO specification requirement, but not at 60 mph (96 km/h). The specifications need to be more specific.

Recent full-scale tests conducted at Texas Transportation Institute using both 2,250- and 4,500-lb (10.0 and 20.0 kN) vehicles impacting a dual-legged 12WF45 sign structure (on a single support) at 20 and 40 mph (32 and 64 km/h) support the trends shown in figures 2 and 3.

Figure 4.—The FHWA's Riverdale, Md., pendulum facility for highway structures impact research.

This research should develop a more comprehensive acceptance test specification for sign supports and luminaires. Research facilities such as the FHWA pendulum shown in figure 4 are being employed with crushable impact modules

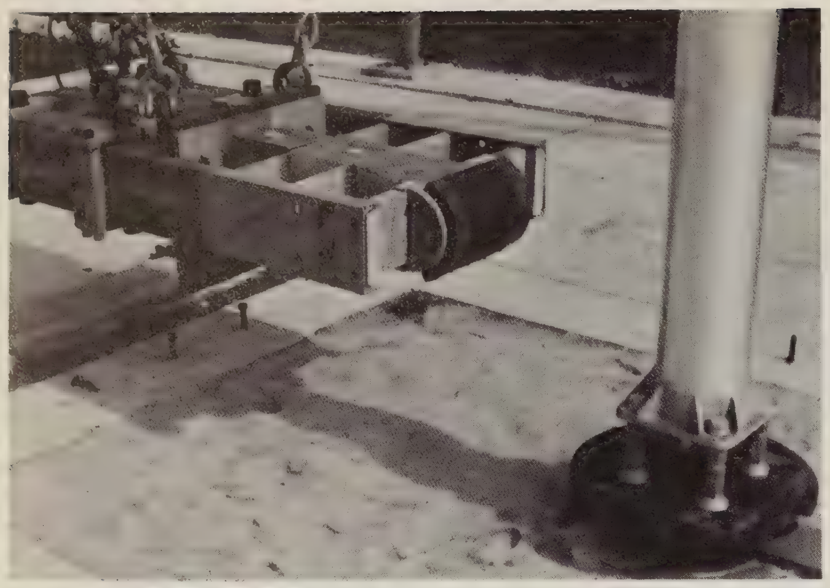

Figure 5.—Crushable nose section of the FHWA pendulum facility with luminaire support mounted on breakaway couplings.

(fig. 5) made of aluminum honeycomb to simulate full-scale vehicle tests, without going to the test track.

Such facilities should provide close correlation with full-scale tests when the crush characteristics of the honeycomb are matched to those of the test vehicle. Work to date provides encouragement that such correlation can be achieved for the lower velocities; and that these results, coupled with careful simulation for higher velocity impacts, will lead to the development of performance characteristics for a wide range of roadside structures. The result will be safer highways.

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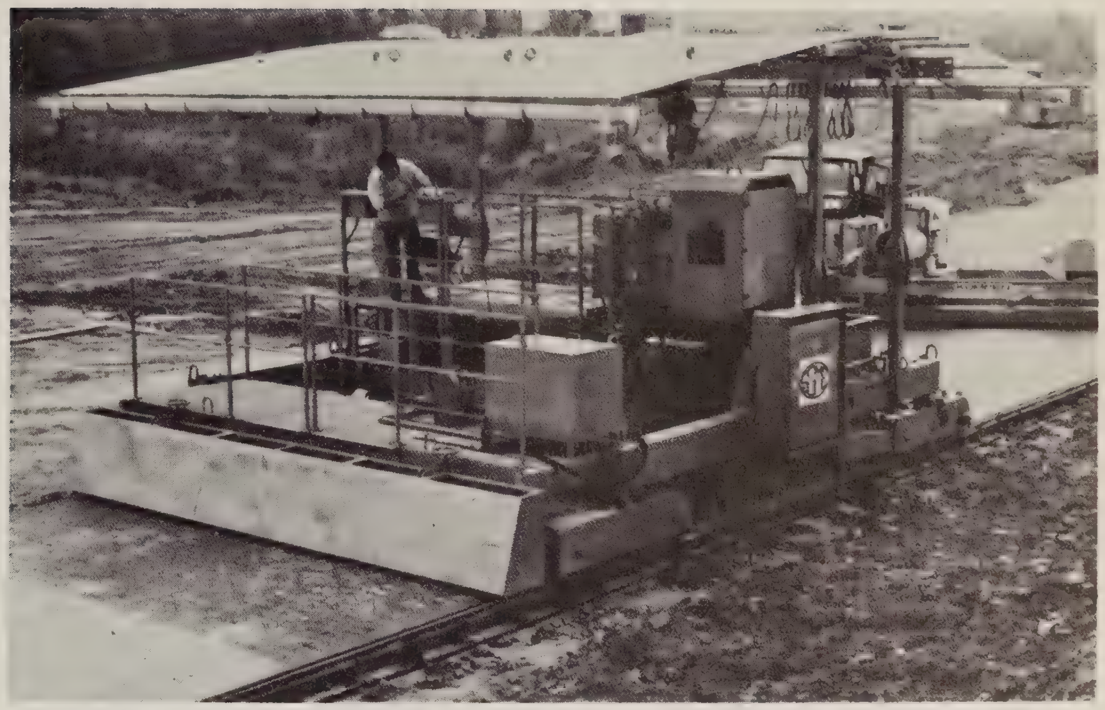
New Publication

Transportation System Management: A Bibliography of Technical Reports contains a listing of readily obtainable technical reports on operational transportation improve- ments. Descriptions and availability information on over 150 reports dealing with low-capital, short-range, or polic oriented urban transportation improvements are included. The reports are classified into nine sections. The first section, General, includes transportation management overviews, survey reports on the various operational approaches and strategies for improved transportation efficiency, and demonstration program reports. The remaining sections contain reports focused on the following areas: Preferential Treatment for High Occupancy Vehicles, Traffic Operations, Parking Management, Transit Improvements, Transit Managemen Pooling and Paratransit, Pedestrians and Bicycles, and Transportation Demand Management.

The bibliography may be purchased for $6.75 in papercop and $2.25 in microfiche from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 257273).



Fresh concrete is given a durable, rough macrotexture by sprinkling with chips from non-polishable stone. (See fig. 4.)

Cement Concrete Roads in Belgium

by [1] G. Van Heystraeten

The main roads network of Belgium is characterized by a high density, heavy loading, and a variable, temperate, maritime climate. These conditions have caused Belgium to become one of the leading countries in cement concrete roadbuilding. The usual concrete composition is rich in cement and corresponds to a fairly stiff mixture. For acceptance, it is tested for compression strength on a statistical basis. Construction includes both fixed form and slip-form pavers with the latter requiring special adaptation to Belgium's particular mix composition.

Three finishing techniques for obtaining a rough macrotexture of fresh concrete have been investigated on test sections: deep transverse grooving, concrete chipping, and concrete stripping. An investigation has also been conducted and improvements made in connection with the use of the curing compounds.

The transverse joints are systematically dowelled and the use of prefabricated dowel supports is compulsory. The bottom insert has been abandoned and the top groove is now sawed, at least on the major roads. The longitudinal joints are anchored and the load transmission is achieved by a tongue-and-groove joint.

Standards for continuously reinforced concrete pavements have been derived.

[1] A lecture sponsored by the University of Illinois at Urbana and the U.S. Army Construction Engineering Research Laboratory at Champaign, Ill., and a lecture at the Fairbank Highway Research Station, Federal Highway Administration, Washington, D.C., November 1975.

Introduction

Belgium has an area of 11,776 square miles (30,500 km^2). The relief is mainly characterized by a succession of plains and plateaus. In the north the lowland reaches 164 ft (50 m) above sea level, in the central part low plateaus range from 164 to 984 ft (50 to 300 m) above sea level, and in the south the hills of the Ardennes are as high as 2,297 ft (700 m). The climate is temperate maritime. From north to south the annual precipitation varies from 31.5 to 55 in (800 to 1,397 mm), the number of days of frost ranges from 45 to 120, and the number of days of snow from 10 to 35. One of the main characteristics of the climate is that it varies greatly from one day to the next and from one year to the next.

The number of vehicles in circulation in Belgium in 1974 was 2,502,000 non-commercial vehicles, 19,000 buses, and 268,000 commercial vehicles. This is a ratio of one vehicle to every four inhabitants.

Table 1 shows the mileage of paved roads and the percentages for cement concrete roads. With 4.8 miles of roads per square mile (3 km of roads per km²) it may well be said that the road network is particularly dense.

Figure 1 is a map of the freeway network which, in addition to national traffic, is also subjected to a large volume of traffic in transit. Due to its central location, Belgium is one of the most important points of intersection for traffic in Western Europe.

Table 1.—Mileage of paved roads in Belgium (1975)

Road type	Mileage	Percent concrete roads
Freeways	629	38
State highways	6,582	14
Provincial roads	842	24
Local and rural roads	49,720	≈ 25
Total	57,773	

1 mile=1.6 km

Another key factor in the design of pavements is the authorized axle load:

- The authorized axle load is 14.3 tons (13 metric tons) as against 9 tons (8.2 metric tons) in the United States.
- The authorized weight per dual axle is 22 tons (20 metric tons), as against 15.5 tons (14.1 metric tons) in the United States.

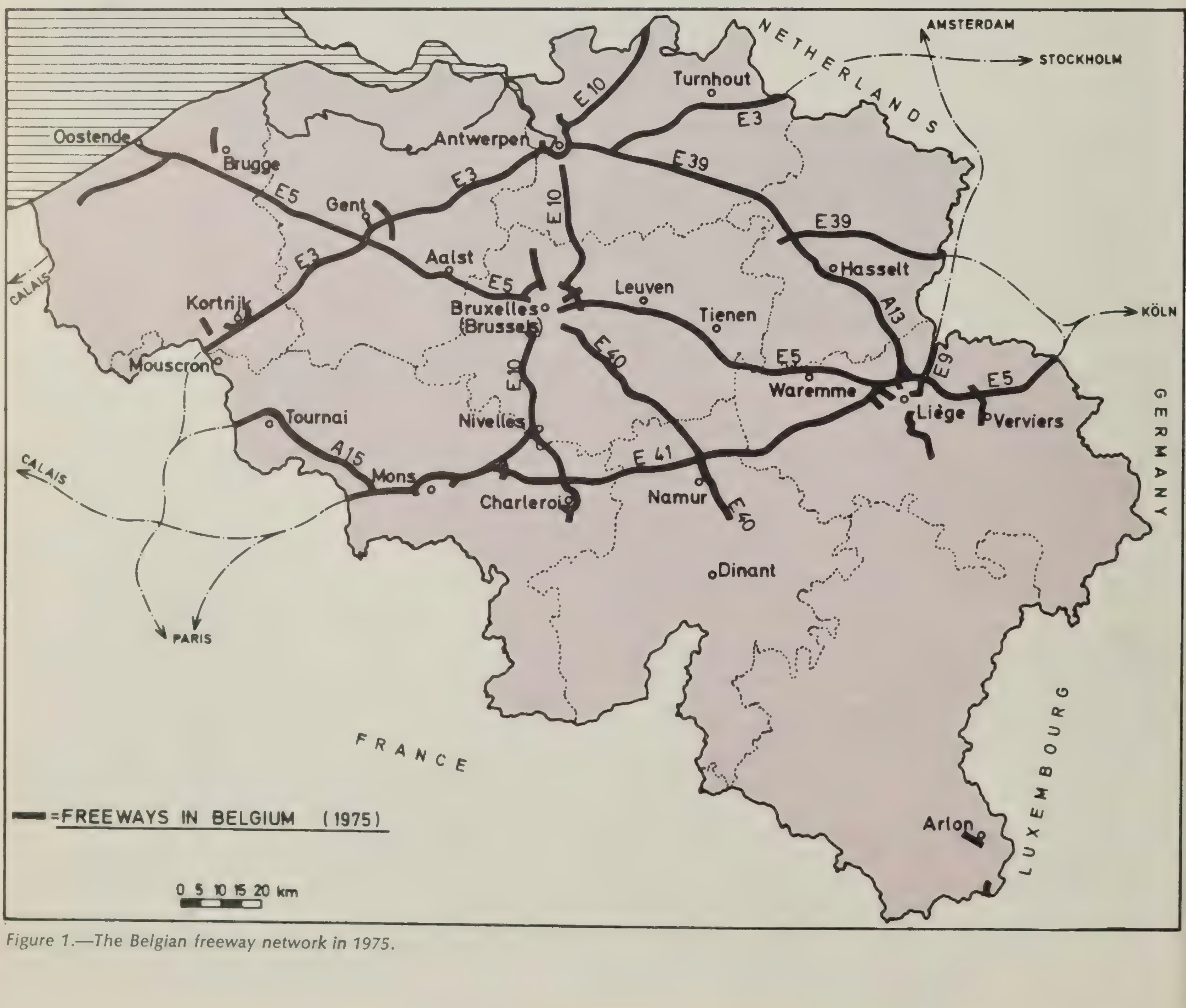


Figure 1.—The Belgian freeway network in 1975.

Considering that many vehicles are illegally overloaded, it may be concluded that the Belgian road system is one of the most loaded in the world, both in number and in weight of vehicles.

The Concrete

Materials

Adequate supplies of basic materials are available and an important cement industry (7.5 million metric tons [8.3 million tons] in 1974, 20 percent of which are exported) is well established along with many limestone, porphyry, and sandstone quarries. Various parts of the country have nearby sources of crushed river gravel. The rivers also serve as an important source for the coarse sand used in cement concrete. Materials which are not of natural origin, such as blast furnace slag, are not approved for use in road construction.

Mix design

The following basic principles of mix design have not changed to a great extent in the last 25 years:

- A more or less uniform gradation obtained by mixing three sizes of crushed gravel with coarse natural sand.
- A maximum size of 1¼ in (32 mm square sieves) aggregate.
- An amount of cement ranging from 6.3 to 7.2 bags/cubic yard (350 to 400 kg/m³) of compacted concrete.
- A water/cement ratio of 0.40 to 0.42.

Thus a classical composition is as shown in table 2 or otherwise expressed as 64 percent stone, 19 percent sand, and 17 percent cement by weight of the dry mixture. On the whole, that amounts to 40 percent of mortar (sand, cement, and water)—which is similar in consistency to a stiff mixture.

Quality of the concrete

The quality of the hardened concrete is checked by testing only the compression strength, measured after 90 days on 4-in (100 mm) high disks cut from roadway cores with a 15.5 in² (100 cm²) cross section.

The Belgian Ministry of Public Works, as provided by the specifications for freeways and state highways (*1*),[2] wants to be statistically sure that for 95 percent of the pavement the compressive strength shall exceed the minimum threshold value of 870 psi (6,000 kPa). Thus the minimum value to be obtained for the average of all the results depends on the standard deviation. For example, for a standard deviation of 104 psi (720 kPa)—a frequently observed standard deviation—the average compression strength ought to be as high as 1,044 psi (7,200 kPa). Failure by a contractor to comply with this requirement will result in a monetary penalty and may even result in the work not being accepted. Most contractors find it is easy to satisfy the requirements due to the high cement content, the high compactness—dry volumetric weight of 144 pcf (2,300 kg/m³)—and the comparatively low water content.

Table 2.—Classical composition of Belgian concrete

Size designation	Pounds/yard³	Kg/m³
¾ in to 1¼ in	1,265	750
¼ in to ¾ in	632	375
No. 12—¼ in	590	350
Sand	801–717	475–425
Cement	590–674	350–400
Water	253–270	150–160

1 in=25.4 mm

[2] Italic numbers in parentheses identify the references on page 131.

The compressive strength requirements for provincial and local roads are, of course, less severe, averaging at 56 days 870 psi (6,000 kPa), and even less, say 580 psi (4,000 kPa), for rural roads. (2) Thus for these rural roads, the concrete contains less cement, 5.4 bags/cubic yard (300 kg/m³), and the sand content is higher.

Construction

Paving train

Until 1970 concrete pavements were placed using only the conventional paving train on fixed side forms. Since 1970, the slip-form paver has been introduced as a result of an extensive program providing for building 155 miles (250 km) of cement concrete freeways in 1971 and 1972.

Two types of slip-form pavers appeared on the Belgian market:

1. The Société Générale de Matériel d'Entrepreneurs (SGME) machine, manufactured in Belgium, having a working width ranging from 23 to 40 ft (7 to 12.25 m).
2. The CMI Corporation machine, imported from the United States, capable of laying widths of concrete ranging from 23 to 26 ft (7.0 to 7.9 m). The equipment comprises a separately operating concrete spreader and the finishing machine itself.

The first operations quite clearly showed that the Belgian concrete and the slip-form paver could not be reconciled unless concessions were made on both sides: either change the composition of the Belgian concrete or adapt the machine, originally designed for United States concrete.

The Ministry of Public Works required that the limits within the existing standards set for the concrete—such as compressive strength and density—shall be maintained; the composition was only slightly changed. The mortar content was increased from 40 to 45 percent through a slight increase in sand and cement, and a water reducer was added. Thus a slump of about 1.2 in (30 mm) is obtained. The stringent requirements relating to compressive strength and density exclude the use of air-entraining additives. Nevertheless, because of the high cement content, no frost damage has been found. Modifications were made on the machines themselves, the most important of which was an increase in the number of vibrators, from 12 to a minimum of 18 which are needed for a working width of 24.6 ft (7.5 m).

Furthermore, the first trials with slip-form pavers showed that the guide wire level adjustment technique resulted in a lag in the response time which was detrimental to the evenness. The best results were obtained by profiling the supporting surfaces as well as possible and abandoning the guide wire. This is one of the reasons for placing an asphalt layer between the lean concrete and rich concrete resulting in the concrete freeway cross section as shown in figure 2. Experience has shown that the traditional paving train on fixed side forms provides the smoothest riding surface.

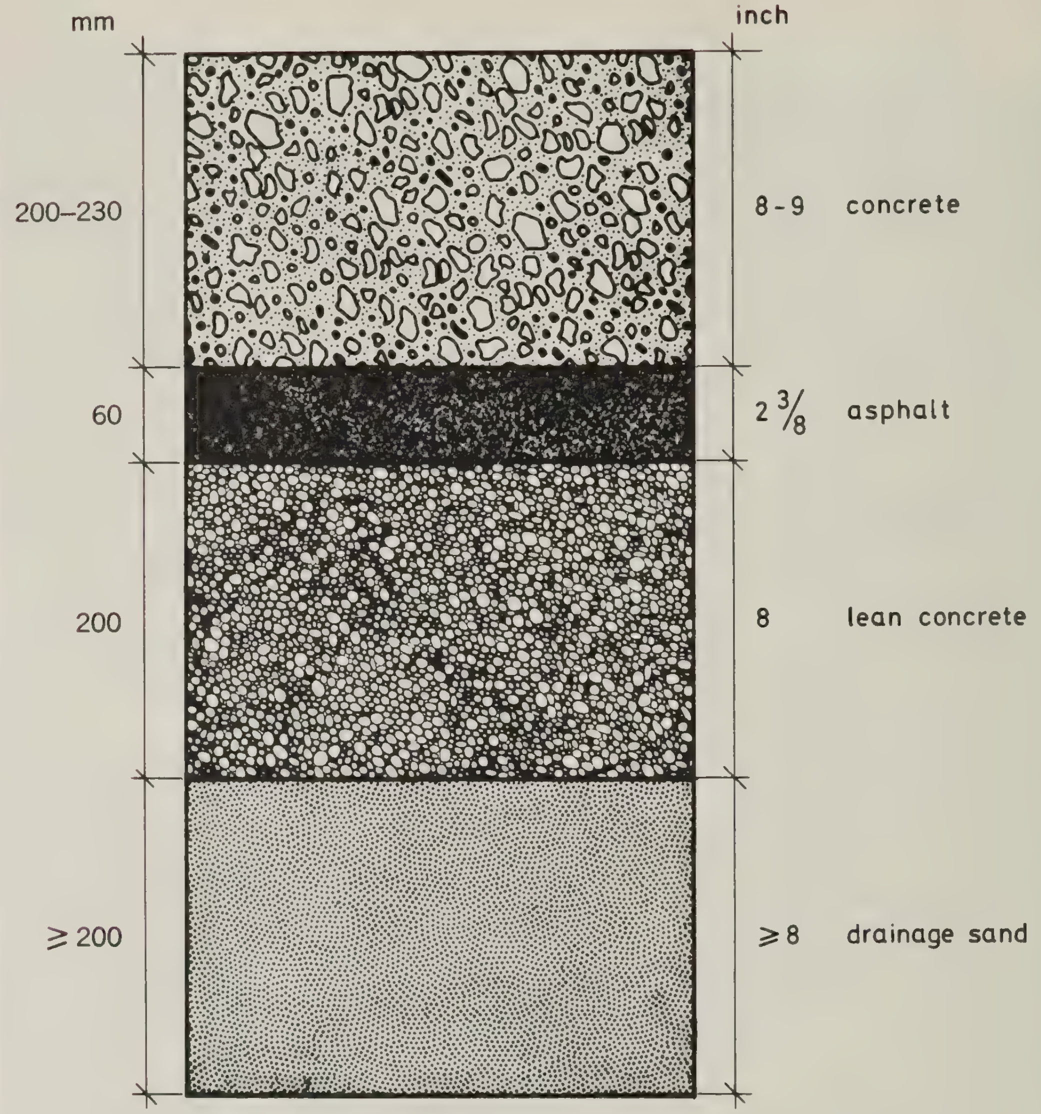


Figure 2.—Typical structure of a Belgian concrete freeway—8 in (200 mm) concrete in case of CRC.

Although there are now nine slip-form paving machines in Belgium, the conventional paving train is still in use especially for two-way roads. The lack of a slip-form paver which has sufficient maneuverability in sharp bends and capable of widths less than 24.6 ft (7.5 m) is the principal reason for using two systems. A prototype of a machine with greater flexibility is under study.

Finally, for rural roads, where the evenness requirements are much less severe, the paving is nearly always completed with an appropriate asphalt-finisher.

Surface treatment

Belgium did not include a skid resistance performance requirement in its specifications until 1960. Consequently, no restrictions were placed on polishing resistant aggregates or on finishing the fresh concrete. Polishable limestone was used in the regions with limestone quarries and a number of contractors brushed the concrete with an ordinary household brush, more to cover up any irregularities than to provide for anti-skid properties.

Nevertheless, between 1950 and 1960, the Belgian Road Research Center laid test sections to isolate the effects of different aggregates and finishing techniques on the performance of the pavement.

By repeated testing of the sections, the importance of the following skid resistance factors became evident:

- The microtexture of the pavement, specifically the polishability of the surface aggregates. This factor was evaluated by the test called "Polished stone value" developed by the British Transport and Road Research Laboratory. (*3*) This test consists of subjecting the sample of stones to be investigated to an accelerated polishing operation, after which the degree of polishing is measured using a pendulum. The more resistant the surface aggregates are to polishing (harsh surface), the better the skid resistance (fig. 3).

- The macrotexture, that is the perceptible relief in the surface. The "Sand patch test" has been developed to evaluate this type of texture. With a rough macrotexture, the decrease of the friction coefficient with speed remains limited (fig. 3).

At the same time three finishing techniques for obtaining a rough macrotexture in fresh concrete have been investigated:

1. Deep transverse grooving with a modulated brush.

2. Sprinkling the fresh concrete with chips from non-polishable stone.

3. Brushing away the surface mortar in order to expose the stone skeleton even before opening to traffic. This process is called concrete stripping.

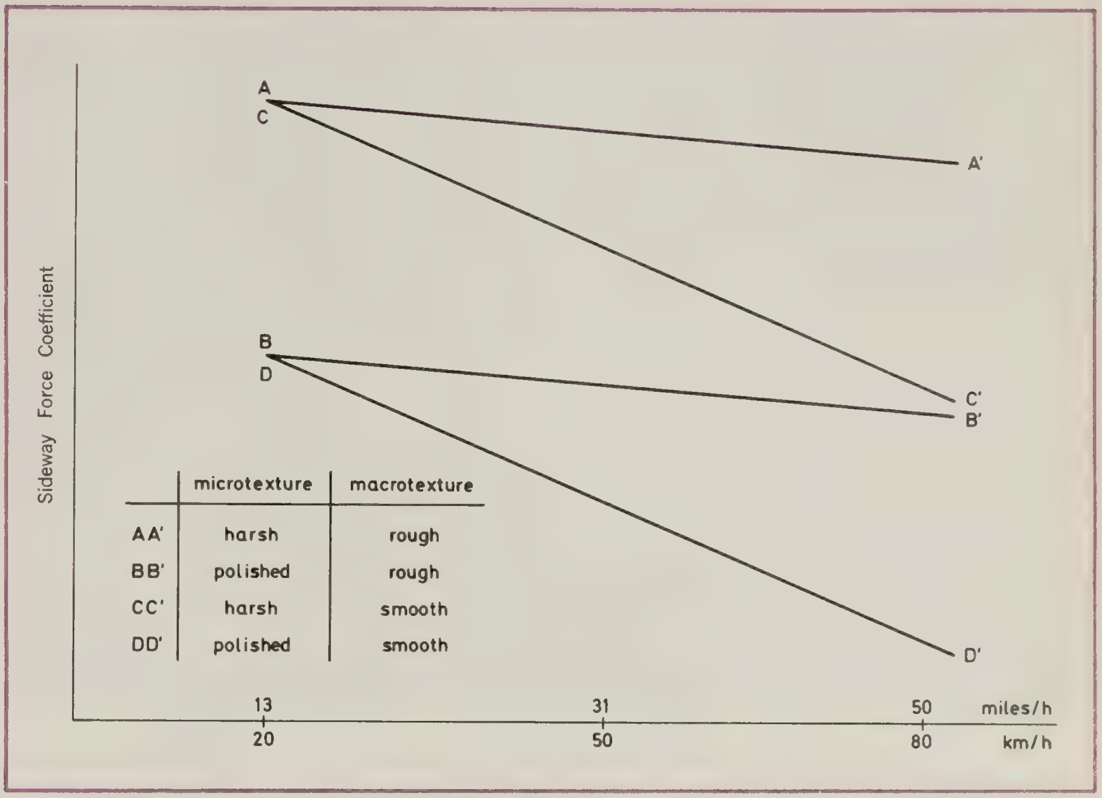


Figure 3.—Influence of the type of micro- and macrotexture on skid resistance.

All three techniques appear to be capable of providing the fresh concrete with a rough macrotexture. The deep transverse groove technique has, however, been selected because the mechanization seemed easier than for the other two techniques.

The mechanization of the transverse grooving process was developed in the 1960's. The SGME machine was introduced on the market in 1966, the machine developed by the Belgian Road Research Center in 1968 (*4, 5*), and the CMI machine in 1970.

In 1968, grooving by machines was made compulsory, the groove pattern standard set, and the sand patch test introduced as a means of checking the groove depth.

The most common groove pattern comprises 0.16- to 0.25-in (4 to 6 mm) wide grooves, spaced 0.8 to 1.2 in (20 to 30 mm) from centerline to centerline with a depth of 0.20 to 0.28 in (5 to 7 mm). The use of limestone with a Polished Stone Value (PSV) less than 0.50 was not permitted. In addition to these measures, there was the contractual introduction of a severe requirement in connection with the Sideway Force Coefficient (SFC) which has to remain satisfied during a guarantee period of 3 years.

Earlier reports have documented (*4, 5*) that the deep transverse groove technique gives both efficient and durable results. Numerous test sections exist which have already been subjected to 12 years of heavy traffic and still show the traces of the original grooves. Apart from a low sensitivity to speed of the SFC, other advantages of transverse grooving have been observed. The surface drainage is greatly improved, the splash and spray behind the tires are perceptibly reduced, and the headlight glare reflection of the vehicles on the pavement is reduced.

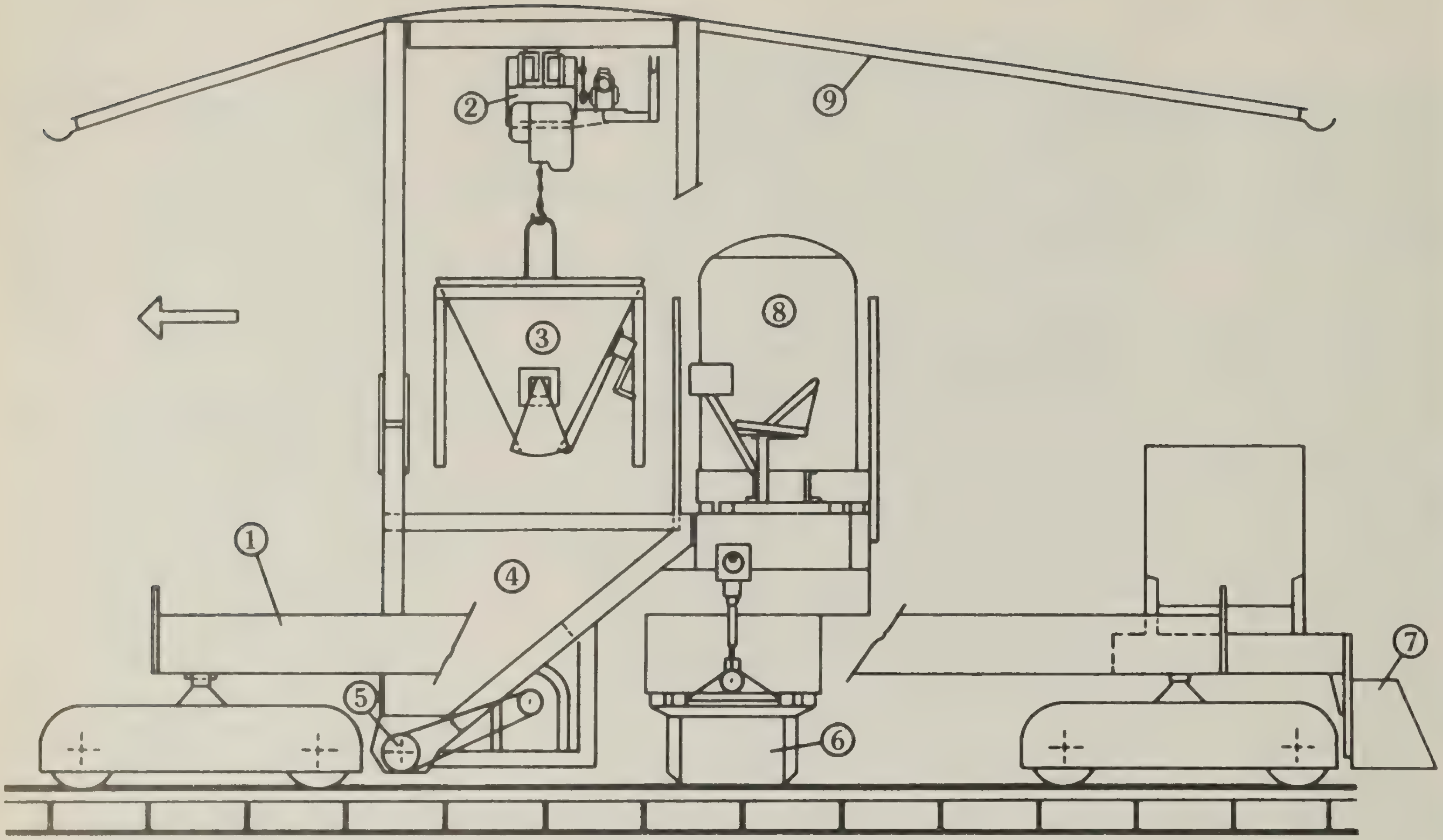


Figure 4.—Schematic view of the machine for spreading and embedding chips in fresh concrete.

The durability of the skid-resistance of deep transverse grooved cement concrete can only be assured if no polishable materials are used. To minimize the impact this might have on limestone quarries, the Belgian Road Research Center conducted an investigation on the mechanization of the previously mentioned fresh concrete chipping process. The chipping process would allow the use of polishable stones in the concrete (6) with the exception of the surface chips. Using this process, the microtexture of the concrete aggregates in the basic pavement structure does not influence the skid resistance.

Figure 4 shows a machine that has been designed and manufactured by the Belgian Road Research Center and SGME (7) for applying chips to the surface of fresh concrete. The rigid frame (1) carries a monorail with traveling carriage (2) and loading bucket (3) used to fill the storage bin (4) which feeds a distribution drum which deposits the chips on the pavement surface (5) followed by a vibrating tamper beam (6) and a curing compound spray bar (7). Power and control of the machine is provided by an electric generator and control desk located on the bridge (8) and protected from the weather by an awning (9).

The required spreading rate of 11 to 15 lb/yd^2 (6.0 to 8.1 kg/m^2) of 1/2 to 3/4 in (13 to 19 mm) size chips is obtained by adjusting the discharge opening and the rotation speed of the drum. Spreading the chips and embedding them by means of the vibrating tamper beam are carried out in one passage of the continuously advancing machine. The result obtained is shown in figure 5.

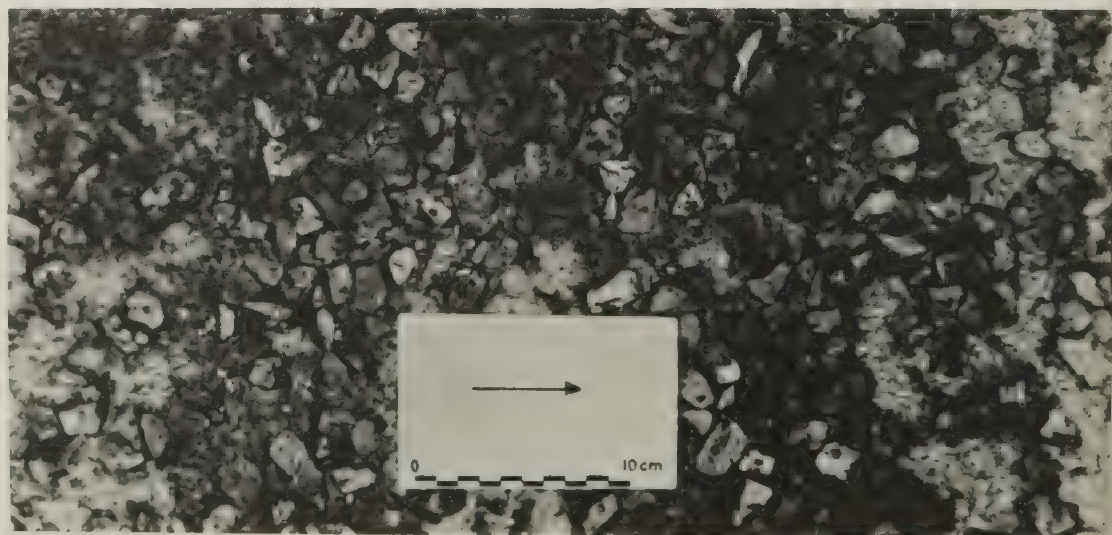


Figure 5.—Close up of a chipped concrete.

Since approximately 71,759 yd² (60,000 m²) have been laid using this machine in 1974 and 1975, the machine may be considered to be operational for work on fixed side forms. The satisfactory results achieved thus far are expected to bring the process into large-scale use in the near future. (*8*) [3]

Curing

Curing compounds were introduced in Belgium around 1956. Since then, encouraged by skillful pseudo-technical advertising from suppliers, contractors have used these products to replace the prescribed methods of straw mattings, canvas, wet earth, or sand without thoroughly evaluating the effectiveness of the curing compounds.

In 1970, the Center made an investigation of 38 products available on the European market, based on a method adapted from the C–156 ASTM. (9)

The study on these products showed that only 16 satisfied the criterion of a protection coefficient of at least 80 percent and only 18 had a reflecting coefficient of at least 40 percent. If additional standards are set—such as requiring that a good curing compound should have a drying time of less than 2 hours, that it should be easily sprayable, and that it not show disturbing reactions with water and calcium ions—it was to be expected that less than one-fourth of the 38 products investigated would be satisfactory. The investigation of these products has led to both a better selection and a better application of the curing compounds through the use of a mechanical spray process. The prescribed minimum amount is 0.23 lb/yd² (0.125 kg/m²), and the curing compound must be sprayed immediately after finishing the concrete.

At this time it is planned to require that a rolling roof 165 to 330 ft (50 to 100 m) long trail behind the paver. This requirement will prevent the concrete from washing out by rain when it is less than 2 hours old. A last means of protection for the fresh concrete is the prohibition of any traffic on the new pavement until a compressive strength of 580 psi (4,000 kPa) has been attained.

A side drawback of some of the curing compounds is the persistence durability of these products under traffic conditions. Indeed, it has been observed that the remaining film may have a detrimental influence on the skid-resistance. Thus, the persistence of several products—which may last for a number of years—is a supplementary criterion that requires more study.

[3] "A New Technique for Achieving Non-skid Cement Concrete Pavements by the Chipping of Fresh Concrete," by F. Fuchs. Oral presentation at the 55th Annual Meeting of the Transportation Research Board, Washington, D.C., 1976.

Figure 6.—Blow up of a concrete slab.

Pavement Joints

Transverse joints

Until 1957 Belgian concrete roads with nonreinforced slabs were always placed with expansion joints without dowels, spaced from 33 to 50 ft (10 to 15 m) apart, and placed on unbound base course. As long as traffic was not too dense and too heavy, this type of construction performed well. Since this time, however, important drawbacks have appeared:

- Differential settlement at the joints has greatly reduced the riding comfort and caused heavy damage at the edges of the joints.
- Water infiltration through the wide joints resulted in a reduction of the bearing capacity and pumped out the underlying foundation leading to many slab fractures.
- Slab blow up (fig. 6) occurred at the expansion joint during extended hot periods if the wooden plank used to form the joint was not kept vertical during paving.

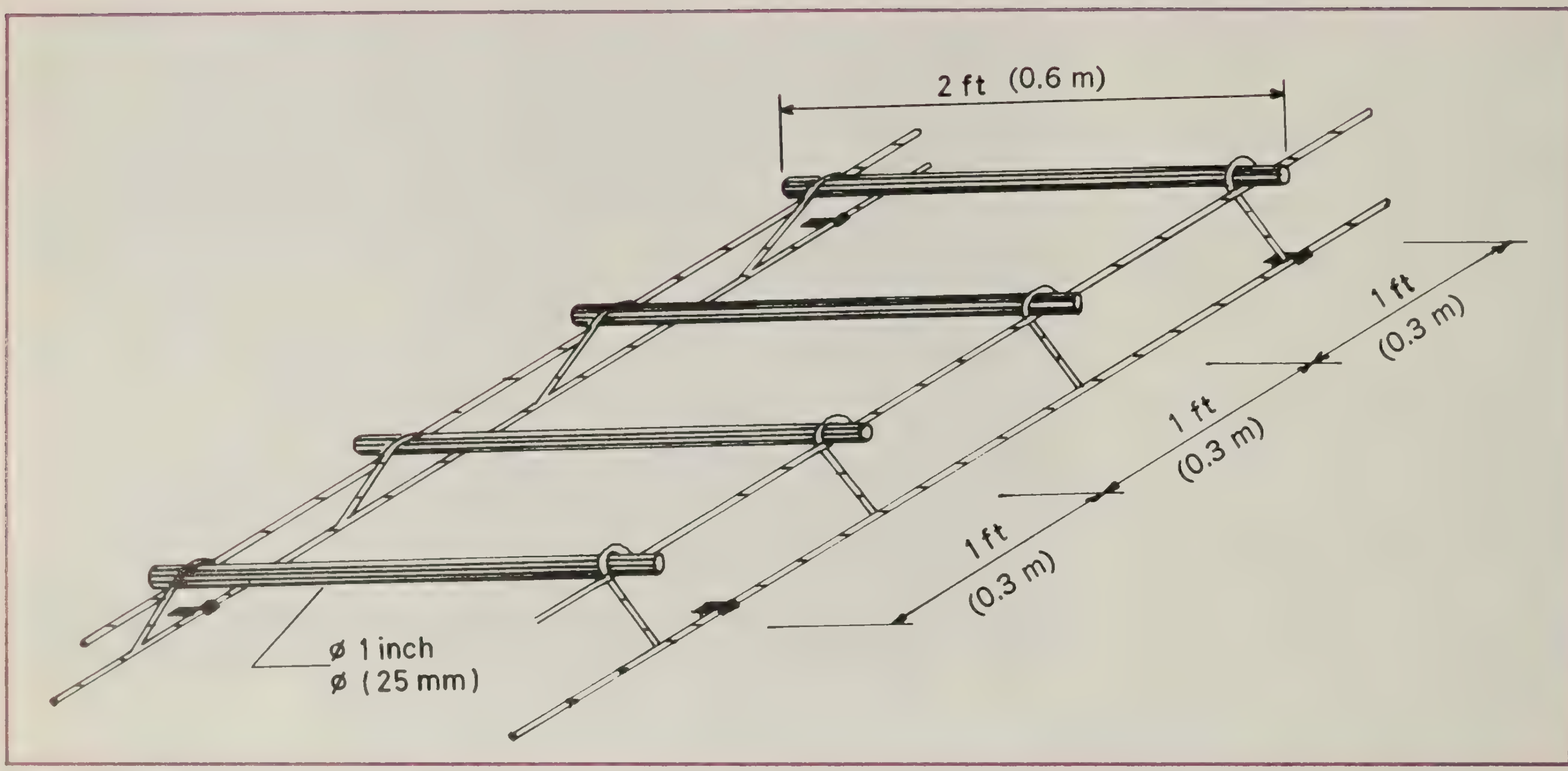


Figure 7.—Dowels placed on supports.

■ Penetration of hard stones into the joint resulted in spalling at the edges when the joint closed.

In order to overcome these disadvantages, stabilized base and dowelling were systematically introduced; expansion joints were replaced by contraction joints, except near bridges and crossings; and, at the same time, slab lengths were kept as short as 20 ft (6 m). (*10*) Dowels are 2 ft (0.6 m) long, have a 1-in (25 mm) diameter, and are placed 12 in (300 mm) on center. Early attempts to use dowels began by supporting them on a beam of precompacted concrete which did not insure that the dowels remained in place during the placement of the concrete. This technique was replaced by the use of prefabricated dowel supports, as shown in figure 7, which has been made mandatory. Insertion of the dowels after placing the concrete has found little application until now.

Figure 8.—Corrugated strip of asbestos cement as crack inducer.

The design of the contraction joints has also been changed since their initial use. In the beginning inserts at the bottom and at the top of the slab were provided. The insert at the bottom was achieved through embedding a 2-in (50 mm) corrugated asbestos strip (fig 8). This technique involves a risk of wild cracking originating at this insert.

The top groove for crack initiation was a cut in the fresh concrete and 0.16-in (4 mm) thick by 2.0-in (50 mm) high fiber plate was inserted. Finishing was completed afterward with a trowel, and quite often too much water was used to achieve better workability. The introduction of an oblique leveling beam made it possible to minimize the work done by hand; in addition, joints mechanically finished in this way showed much less spalling. The upper part of cut is milled to the depth and width necessary for inserting the joint compound. Nevertheless, a joint made in fresh concrete often gives reduced riding comfort.

Due to these problems, the bottom insert has been eliminated and the top groove is being cut by sawing. Determining the proper time to do the sawing is important. If you wait too long, there is a risk of wild crack formation; if you saw too soon, small stones may be pulled from the edges of the joint. The correct time at which to saw depends on a number of factors, such as concrete composition, saw type, and weather. The sawing depth is one-third of the concrete thickness; the width is 0.16 in (4 mm) maximum. Afterward this groove is widened by milling. The dimensions of this widening depend on the nature of the joint filling compound: hot or cold poured compounds or preformed neoprene inserts.

The provincial and local road system has followed this change in slab length and joint design with some delay, but the sawing of the top groove and the use of neoprene have not been applied because of the high total cost.

Figure 9.—View of a rural road just after construction—7 in (180 mm) of concrete directly on the subgrade.

For Belgian rural roads—mostly laid 7 in (180 mm) thick directly on the subgrade (fig. 9) using appropriate asphalt-finishers—it is essential that both construction and maintenance of the joints be as economical as possible. Economy is obtained by omitting the dowels and designing for transmission of loads between slabs through the interlocking of the asperities of the walls of the contraction joints. (2) For such roads, this type of load transmission is performing satisfactorily. The contraction joint is created by driving a thin plastic insert into the fresh concrete by means of a vibrating knife, after which the unevenness is smoothed away and the surface is equalized by tamping. This type of joint may be laid very rapidly and requires no joint-filling compounds, and consequently, no maintenance.

Longitudinal joints

Transmission of loads in the longitudinal joint between two concrete lanes laid separately is carried out with a tongue-and-groove joint and by using tie bars to prevent the joint from opening. The tie bars are 3 ft (1 m) long and are spaced 2.5 ft (0.75 m) on center. This joint is sealed by making a groove and filling it with an approved joint compound.

On roads where the various driving lanes are laid in one operation, a longitudinal joint is also created between each driving lane. This is carried out either during the paving operation by pushing a thin plastic insert into the fresh concrete or later by sawing a groove and sealing it. Experience with the plastic insert technique has not been good. Longitudinal cracks have appeared close to the joint. This seems to indicate that the plastic insert becomes folded while being unrolled and pushed in, and therefore does not penetrate sufficiently deep to induce a crack.

This phenomenon of erratic longitudinal cracks has also been observed in cases where the joint saw cut depth was insufficient (one-fifth or less of the concrete thickness). Therefore, at this time it is recommended to saw at least to one-third of the concrete thickness.

Continuously Reinforced Portland Cement Concrete

In addition to the nonreinforced cement concrete slabs, continuously reinforced concrete (CRC) is also used. This type of pavement was first constructed in Belgium in 1951 on a 1,950-ft (600 m) long section. In this installation, plain reinforcing steel with an elastic limit of 27,578 psi (4 MPa) was used at a rate of 0.5 percent for one lane and 0.3 percent on the other, of the cross section of the concrete.

Test sections were placed again between 1964 and 1970, where the reinforcement varied from 0.7 to 1 percent of steel with an elastic limit between 28,958 and 34,473 psi (4.2 and 5 MPa). The results of United States experience and of these tests finally resulted in standards being set in 1970 for the construction of CRC roads on a larger scale. (*11*)

The main features of Belgian CRC roads are as follows:

- The 8-in (200 mm) thick concrete is laid in one operation.
- The reinforcement is placed in advance of the pavers on the base using appropriate supports.
- The percentage of reinforcement is 0.85 and the reinforcement has an elastic limit of at least 34,474 psi (5 MPa).
- The longitudinal bars, which are located on top of the transverse bars, are lying 3.5 in (90 mm) below the top surface.
- The transverse bars are placed at a 60° angle to the longitudinal bars in order to avoid cracks along the transverse reinforcements.
- The transverse bars are spaced 27.6 in (700 mm) on center, the longitudinal bars 6 in (150 mm) apart on center. Their diameter is from 0.6 to 0.7 in (16 to 18 mm).

The reinforcement is achieved either with pre-welded meshes including the supports, or by installation of separate transverse bars with welded supports with field installation of the longitudinal bars placed on top of the transverse bars (fig. 10). (*12*)

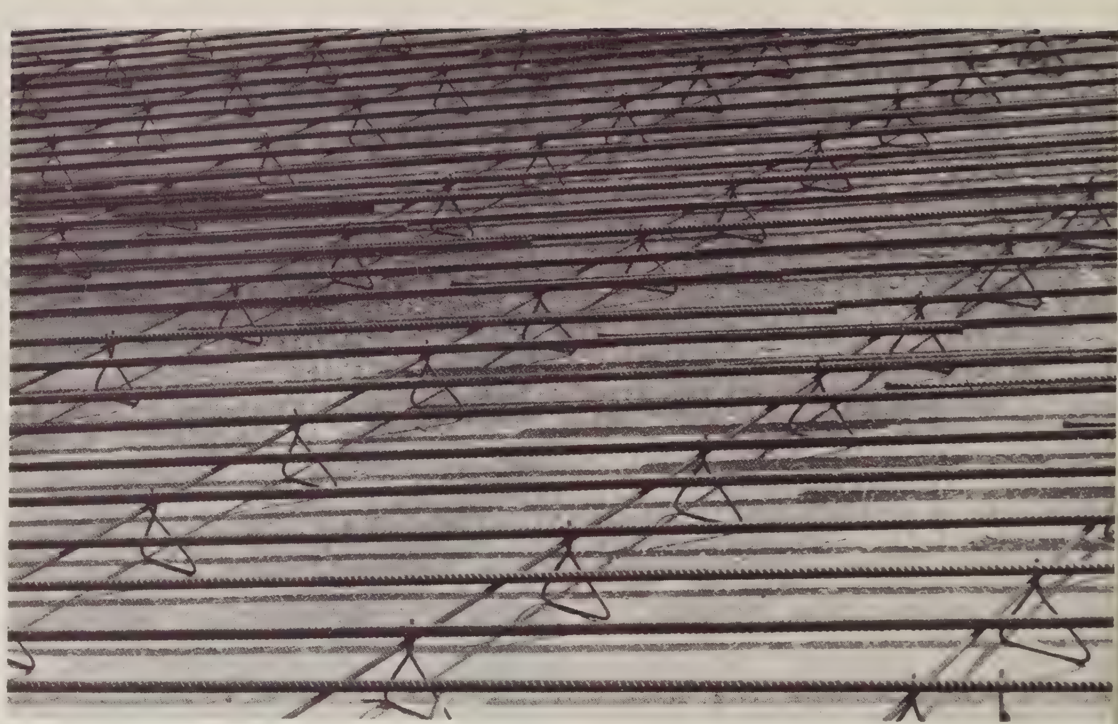


Figure 10.—Reinforcement bars placed on supports.

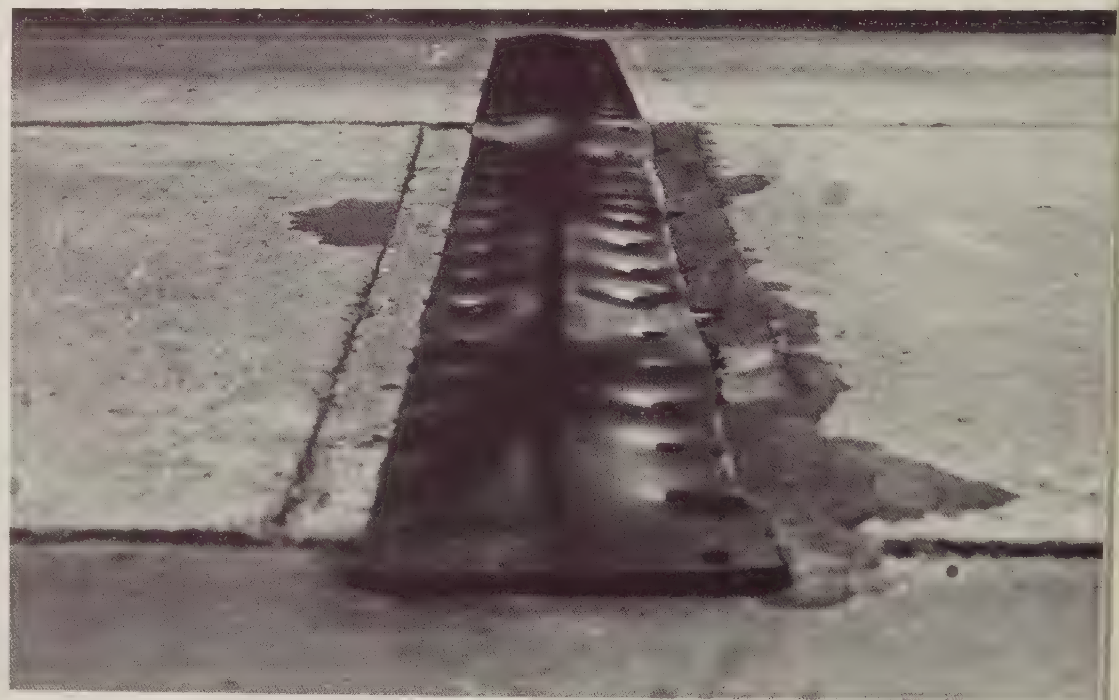


Figure 11.—Special expansion joint at the end of a CRC pavement. Note the damage incurred.

A successful technique for anchoring the ends of CRC pavement has been the use of anchoring abutments with six transverse walls. These abutments have proven to be cheaper and easier to build than the special expansion joints previously used (fig. 11). Furthermore, they insure better riding comfort for the user, do not require any maintenance, and last as long as the pavement. Belgium currently has 190 miles (300 km) of CRC pavement in service, chiefly on freeways.

Conclusions

The first cement concrete pavement in Belgium was placed almost 50 years ago. Today Belgium has 1,240 miles (2,000 km) of this type of road pavement for the freeways and state highways, and about 12,400 miles (20,000 km) for the local and rural roads. Although this mileage is not extremely high in absolute, it remains nevertheless true that, when seen in the European context, Belgium plays a leading part in the cement field and is the most advanced European country for laying continuously reinforced concrete pavements.

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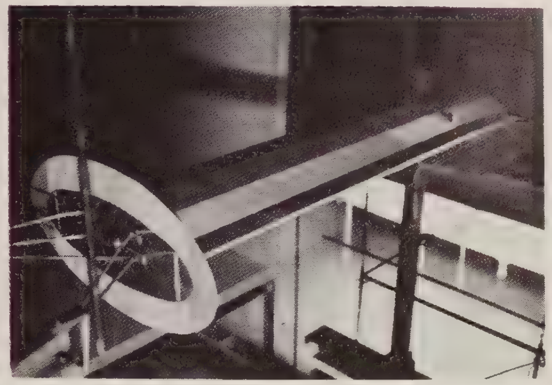
Recent Research Reports You Should Know About

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 Design and Control Division. The reports are available from the address noted at the end of each description.

Recent Methods in the Application of Test Results to the Wind Design of Long, Suspended-Span Bridges, Report No. FHWA–RD–75–115

by FHWA Structures and Applied Mechanics Division

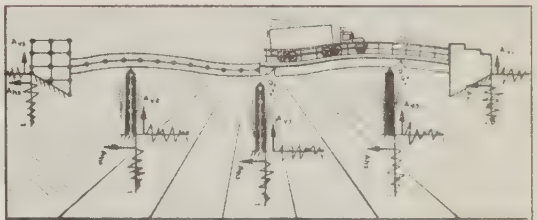
Wind tunnel studies of models of suspended bridge decks have been performed in many parts of the world, notably the United States, Canada, Japan, Great Britain, and France. The Federal Highway Administration's Fairbank Highway Research Station, McLean, Va., is excellently equipped for such work in its George S. Vincent



wind tunnel, which was especially designed to test freely oscillating bridge deck section models.

With the growth of interest in bridge models for aerodynamic investigation, a related body of theory has developed, both for the interpretation of tests and the subsequent dynamic analysis of the prototype bridges under wind. This report summarizes the main aspects of this body of theory which is oriented to the exploitation of the elastically suspended, freely oscillating deck model —a model which is particularly suited to the George S. Vincent wind tunnel. The methods outlined in the report should prove sufficiently general and attractive to find broader application wherever wind tunnel model studies of suspended bridge decks are required.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 250482).



Mathematical Earthquake Simulation

Structural Analysis and Retrofitting of Existing Highway Bridges Subjected to Strong Motion Seismic Loading, Report No. FHWA–RD–75–94

by FHWA Structures and Applied Mechanics Division

Interest in methods of retrofitting existing highway bridges to enhance their resistance to the severe forces of earthquake-induced strong ground motion increased in the United States following the 1971 San Fernando, Calif., earthquake which caused extensive damage to a number of modern freeway structures. This report is the result of a research study to identify and define, through structural analyses, practical techniques and criteria for retrofitting existing bridges. Seven different bridge structures situated in high risk seismic regions throughout the United States were selected as typical for study.

The report identifies potential seismic loads for each bridge based on soil conditions and seismicity at each site.

It also presents a simplified analysis procedure which is applied to each bridge to identify potential weaknesses or failure mechanisms, recommends retrofit measures for those structures requiring them, and describes and illustrates all known retrofit concepts having value for use in reducing damage. The basic concepts include installing superstructure horizontal and vertical motion restrainers for hinges and expansion joints, widening bearing areas, and strengthening columns and footings.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 255299).

Design and Construction of Compacted Shale Embankments, Volume 1 (Report No. FHWA–RD–75–61) and Volume 2 (Report No. FHWA–RD–75–62)

by FHWA Materials Division

The practice of building large, high embankments for today's modern highways from local deposits of shale has led to numerous problems such as excessive settlement and slope failures. This report contains an extensive study of the causes of these problems and an evaluation of remedial actions.

Volume 1, Survey of Problem Areas and Current Practices, shows that distress and failure of shale embankments is attributable to the physical and chemical weathering of certain shales when placed as a rock fill. The experience of some States, the U.S. Army Corps of Engineers, and the Bureau of Reclamation indicates that embankments constructed in thin, well-compacted lifts exhibit no problems.

Volume 2, Evaluation and Remedial Treatment of Shale Embankments, summarizes information obtained from State and Federal agencies concerning techniques to evaluate the stability of existing embankments and remedial treatments for distressed embankments. The relative merits of various laboratory and field sampling and testing techniques for evaluating stability are discussed and summarized in a flow chart of recommended methodology.

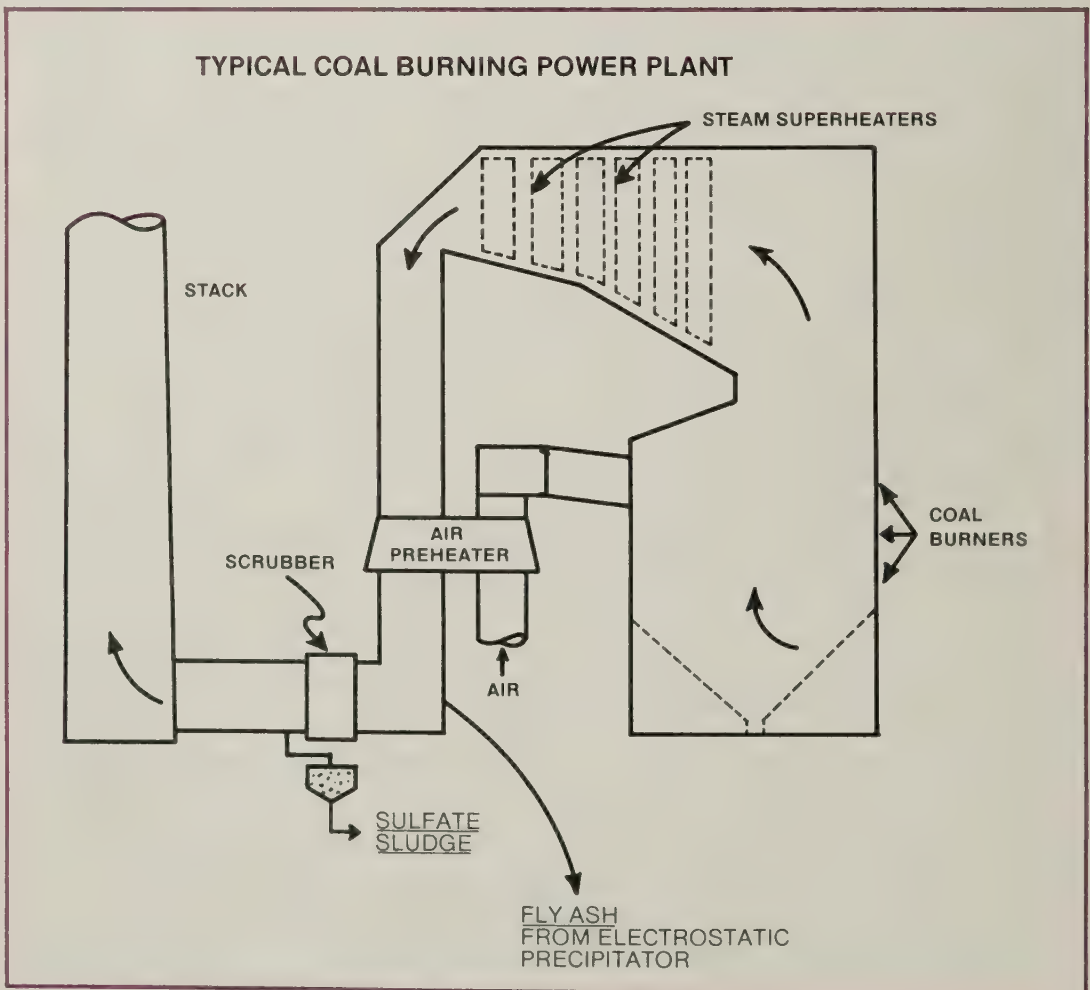
This report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Vol. 1—Stock No. PB 253120; Vol. 2—Stock No. PB 253121).

User's Manual for Sulfate Waste in Road Construction, Report No. FHWA–RD–76–11

by FHWA Materials Division

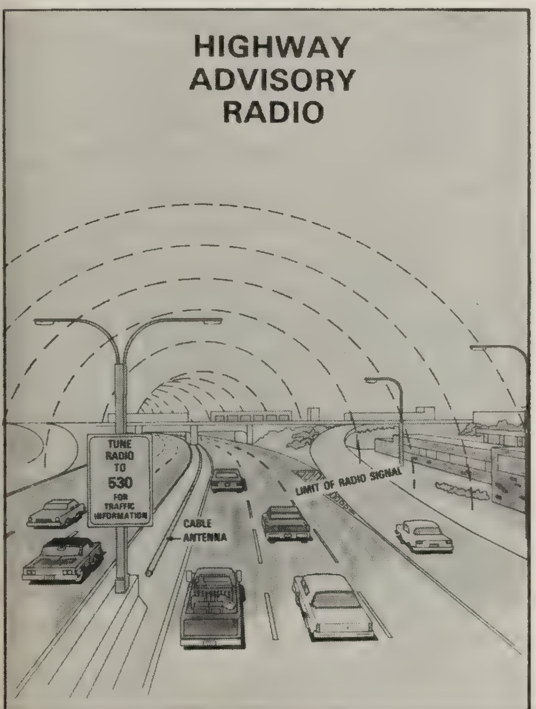
This manual presents background information on the possibility of using waste sulfates in highway construction. It gives known sources of waste sulfate and fly ash in the Eastern United States, laboratory test procedures for mix design, and typical specifications.

The report also contains a summary of the results of laboratory tests on compacted specimens composed of fly ash, waste sulfate, lime, and water. These tests include compressive



strength, tensile strength, California Bearing Ratio, permeability, freeze-thaw resistance, and wet-dry stability. While mixtures had acceptable strength properties, high California Bearing Ratio, and low permeability, the durability properties were marginal. This requires that care and proper precautions must be taken in using these mixtures for construction purposes.

This report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 257176).



Guide to Highway Communications Systems Technology and Design, Report No. FHWA–RD–75–101

by FHWA Traffic Systems Division

Highway communication technology and hardware are significant elements in modern traffic surveillance and control systems. The terminology, definitions, and general understanding of this subject are important to the traffic systems designer. This report was developed as a staff effort to review briefly the various technical considerations associated with the design and implementation of highway communications in traffic systems. It should serve as a technical aid in the understanding of the various elements and subsystems involved.

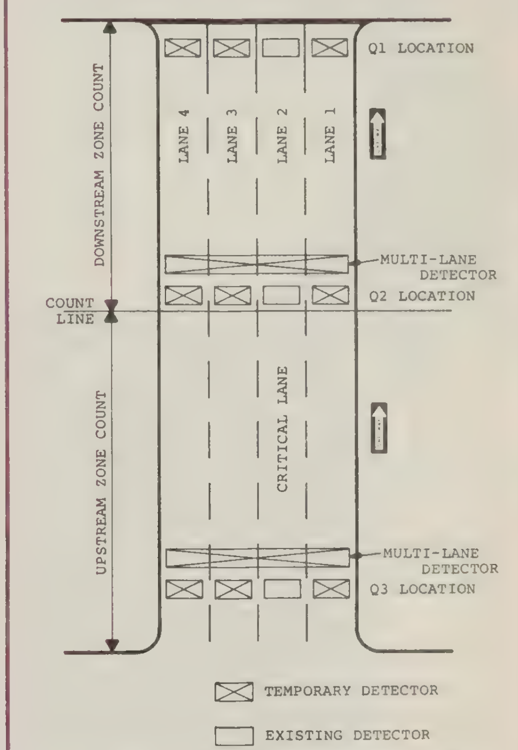
The report is organized under three major topics: Information and Data Sources, Displays, and Communications. Since a detailed coverage of communications was a major objective, additional emphasis is placed on this topic. The report also includes a summary and state-of-the-art discussion of low powered Highway Advisory Radio (HAR).

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 256244).

Locating Detectors for Advanced Traffic Control Strategies—Handbook, Report No. FHWA–RD–75–91

by FHWA Traffic Systems Division

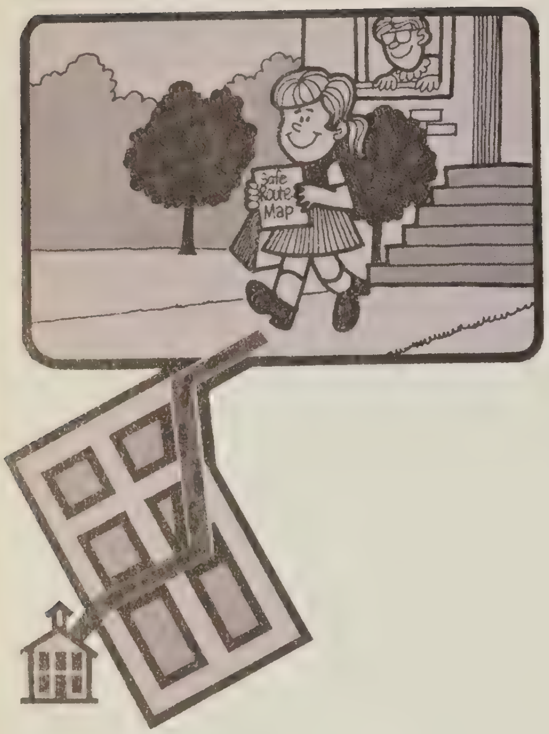
This report presents criteria and procedures for locating detectors to provide required surveillance data. The procedures relate to locating detectors at critical intersections, assessing which intersection approach in the network requires detectorization, and locating detectors on the intersection approach. Both latitudinal and longitudinal placement on the intersection approach are discussed. The procedures were developed as part of the research for the Urban Traffic Control System/Bus Priority System (UTCS/BPS) in Washington, D.C., but are applicable to more general location studies for traffic control systems. This handbook is a supplement to "Locating Detectors for Advanced Traffic Control Strategies—Technical Report," Report No. FHWA–RD–75–92, which is the final



Typical instrumented intersection approach

report for the detector locating project. The final report includes the study procedures and results for the UTCS/BPS network.

Both reports are available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Handbook—Stock No. PB 251182; Technical Report—Stock No. PB 251177).



School Trip Safety and Urban Play Areas, Volumes I-VII

by FHWA Environmental Design and Control Division

This report summarizes the research conducted during the School Trip Safety and Urban Play Areas research study. The study objective was the development of guidelines for the protection of young pedestrians (5–14 years old) walking to and from school, entering and leaving school buses, and at neighborhood play. These guidelines were based on field surveys of the young pedestrian and the driver concerning designated school zones and specific school crossing protective devices, as well as the play street user population, its needs, and the behavior of traffic in the play street area.

Volume I (Report No. FHWA–RD–75–104) provides an overview of the entire research effort. The other six volumes deal with the following topics: Vol. II—Student and Driver Perception of School Trip Safety and Traffic Control Devices (Report No. FHWA–RD–75–105); Vol. III—A Survey of Characteristics of the Urban Play Street (Report No. FHWA–RD–75–106); Vol. IV—An Analysis of Daylight-Savings Time Related to Student Pedestrian Safety Problems and Countermeasures (Report No. FHWA–RD–75–107); Vol. V—Guidelines for the Development of Safe Route Maps for the School Walking Trip (Report No. FHWA–RD–75–108); Vol. VI—Guidelines for Planning School Bus Routing and Scheduling (Report No. FHWA–RD–75–109); and Vol. VII—Guidelines for the Creation and Operation of Urban Play Streets (Report No. FHWA–RD–75–110).

All of the volumes are available from the Environmental Design and Control Division, HRS–40, Federal Highway Administration, Washington, D.C. 20590.

Visual Values for the Highway User

by Harvard University for FHWA Environmental Design and Control Division

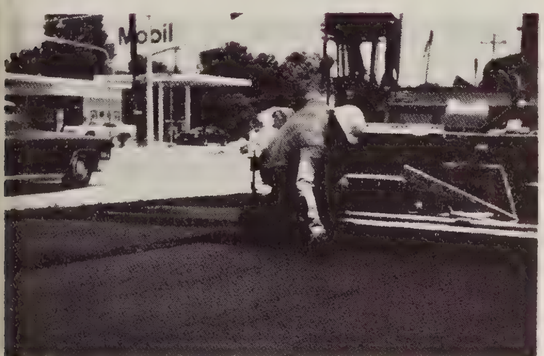
This report is a workbook intended primarily for the design engineer as a resource for understanding the applicability of esthetics in highway development. The workbook identifies many of the components of visual quality which should be considered in the highway planning and design processes. Various phases of the highway development process are used as chapters in presenting the material. The applicability of the guidelines is illustrated by means of a case study from location of a facility through its design. These guidelines for highway esthetics are applicable for most highway types—although a rural, scenic highway is used in the case study. The workbook is not intended as a finalized or proven process and should be tailored to local needs and conditions. It should be considered as a workable beginning point, not a finished product.

This report is available from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–00073–3).

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

The following are brief descriptions of selected items which have been recently completed by State and Federal highway units in cooperation with the Implementation Division, Offices of Research and Development, Federal Highway Administration (FHWA). Some items by others are included when they have a special interest to highway agencies. These items will be available from the Implementation Division unless otherwise indicated.

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

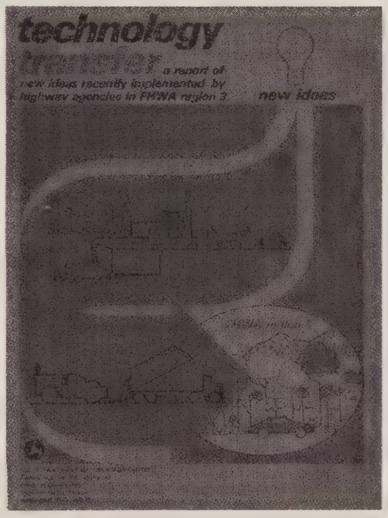


Riding on Refuse

by the Texas Transportation Institute and the FHWA Implementation Division

The use of incinerator residue as a highway construction material has been the subject of research and development efforts for several years. This film, prepared by the Texas Transportation Institute, illustrates the use of incinerator residue as the aggregate for a hot bituminous concrete base course. It clearly shows that standard equipment and construction methods can be used in the construction of a base course using incinerator residue. The section of pavement shown in the film is located in Houston, Tex.

The film is available from FHWA Regional Offices (see p. 131) or the Implementation Division.



technology transfer
a report of new ideas recently implemented by highway agencies in FHWA region 3
new ideas

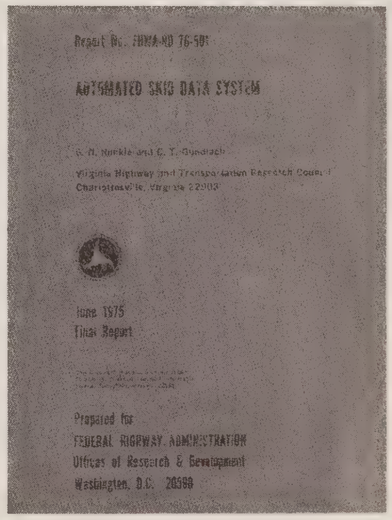
Technology Transfer—A Report of New Ideas Recently Implemented by Highway Agencies in FHWA Region 3

by the FHWA Implementation Division

This report contains listings of research that has been implemented recently or will soon be implemented by five highway agencies in Region 3 of the Federal Highway Administration—Maryland, Pennsylvania, Virginia, West Virginia, and the District of Columbia. Emphasis is placed on research results that have actually been implemented.

Implemented research items are listed by State and are grouped into specific categories: Economics and Other, Environmental Management and the Environment, Maintenance, Materials and Testing, Pavements (Design and Construction), Soils and Geology, Structures (Design and Construction), and Traffic and Safety. Items are identified by a descriptive title and accompanied by a short narrative and the name and telephone number of an individual familiar with the implementation.

The report is available from the Implementation Division.



Report No. FHWA-RD-76-501

AUTOMATED SKID DATA SYSTEM

June 1975
Final Report

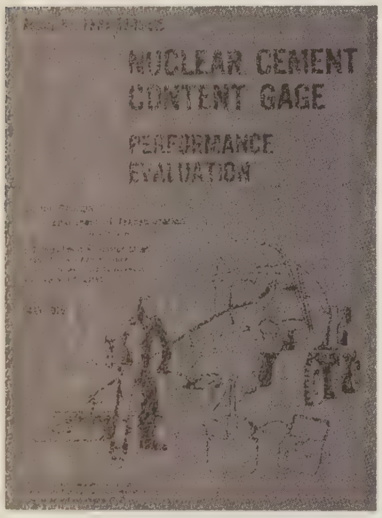
Prepared for
FEDERAL HIGHWAY ADMINISTRATION
Offices of Research & Development
Washington, D.C. 20590

Automated Skid Data System, Report No. FHWA–RD–76–501

by Virginia Highway and Transportation Research Council

The Data Systems and Analysis Section of the Virginia Highway and Transportation Research Council has been involved in the development of automated information systems for various types of roadway data for some time. The system discussed in this report is for the collection, storage, and retrieval of skid data collected during normal survey skid testing. Included in this report is a description of the data files maintained for skid data and the computer programs developed to edit input data, maintain data files, and provide skid data output listings. Instructions regarding the function and use of the programs and listings for each program are also given.

The report is available from the Implementation Division.



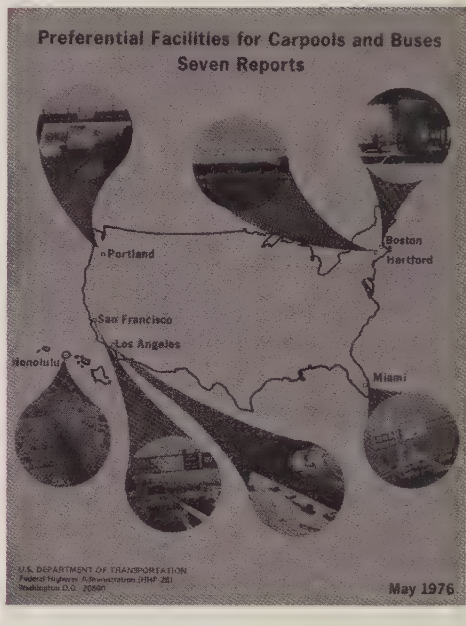
Nuclear Cement Content Gage Performance Evaluation (Georgia), Report No. FHWA–RD–76–525

by the Georgia Department of Transportation and the FHWA Implementation Division

The quality of concrete being placed on highway projects has long been of primary concern to highway departments throughout the country. A nuclear cement content gage which could accurately measure the cement content of concrete in its plastic stage would be a valuable tool in assuring the quality of the concrete that is being placed. Basic research for the development of such a gage was conducted for the Federal Highway Administration from 1968 to 1970 and a prototype gage was constructed. Several modifications have been made to the original gage and the nuclear cement gage used by the Georgia Department of Transportation is one of two modified prototypes which were constructed for a field performance evaluation. The evaluation was performed by establishing and verifying calibration curves and by field testing to determine the accuracy, reliability, and maintainability of the gage under routine use. This report describes the evaluation, presents the results and an analysis of the gage's performance, and includes several recommendations for minor design modifications to improve the ease of operation in the field.

The report is available from the National Technical Information Service, 5285 Port Royal Road, Springfield, Va. 22161 (Stock No. PB 247179).

New Publication



Preferential Facilities for Carpools and Buses contains seven recent reports on preferential highway projects and fringe parking facilities to serve carpool and bus passengers. The reports describe the operating features of many different types of priority treatment. Some of the reports also provide evaluation data on the effectiveness of the projects, or detailed information on the project design and operational control elements.

This publication does not provide a comprehensive overview of preferential treatment strategies. Rather, it supplements the growing body of technical literature on this topic by providing up-to-date status reports on established projects and descriptions of several recently implemented and innovative projects, such as the Banfield Freeway in Portland, I–95 in Miami, the Moanalua Freeway in Honolulu, and others.

This publication may be purchased for $1.10 from the Superintendent of Documents, U.S. Government Printing Office, Washington, D.C. 20402 (Stock No. 050–001–00112–8).

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 Research)—Performing State Highway 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 1A: Traffic Engineering Improvements for Safety

Title: Safety Aspects of Traffic Signal Control on Arterial Systems. (FCP No. 31A1624)
Objective: Investigate and develop appropriate guidelines for improving safety and efficiency of traffic signals on arterial systems.
Performing Organization: Honeywell, Inc., Hopkins, Minn. 55343
Expected Completion Date: June 1978
Estimated Cost: $82,000 (FHWA Administrative Contract)

Title: Evaluation of Bituminous Mix Produced by the Dryer-Drum Process. (FCP No. 41A2013)
Objective: Determine the advisability of permitting the use of the dryer-drum process on New Jersey Department of Transportation project and develop specification provisions and control requirements to govern the equipment's operation if such use is warranted.
Performing Organization: New Jersey Department of Transportation, Trenton, N.J. 08628
Expected Completion Date: November 1977
Estimated Cost: $57,000 (HP&R)

FCP Project 1C: Analysis and Remedies of Freeway Traffic Disturbances

Title: Evaluation of Urban Freeway Modifications. (FCP No. 41C3574)
Objective: Evaluate operation of lanes less than 12 ft (3.6 m) wide; use of shoulders as lanes; implementation of bottleneck solutions; operation, implementation, and control of preferential priority vehicle systems; and the effectiveness of low-volume incident detection system.
Performing Organization: Texas Transportation Institute, Austin, Tex. 78701
Funding Agency: Texas Highway Department
Expected Completion Date: August 1977
Estimated Cost: $115,000 (HP&R)

FCP Project 1L: Improving Traffic Operations During Adverse Environmental Conditions

Title: Alternative Highway Deicing Chemicals. (FCP No. 31L9022)
Objective: Determine the technical and economic feasibility of using chemicals other than sodium or calcium chlorides for deicing without compromising the integrity or utility of the highway itself or adversely affecting the environment.
Performing Organization: Bjorksten Research Laboratories, Madison, Wis. 53715
Expected Completion Date: June 1978
Estimated Cost: $95,000 (FHWA Administrative Contract)

FCP Project 1T: Advanced Vehicle Protection Systems

Title: Test and Evaluation of Heavy Vehicle Barrier Systems. (FCP No. 31T3024)
Objective: Perform full-scale vehicle impact testing of vehicles weighing up to 70 kips (0.31 MN) into guardrails and median barriers, and evaluate both existing and new concepts by simulation and test.
Performing Organization: Ultrasystems, Phoenix, Ariz. 85007
Expected Completion Date: December 1978
Estimated Cost: $549,000 (FHWA Administrative Contract)

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

FCP Project 2K: Metropolitan Intermodal Traffic Management

Title: Evaluation of Control Strategies and Demonstration of Bus Preemption of Traffic Signals in New Jersey. (FCP No. 42K3083)
Objective: Provide a bus preemption system at signalized intersections which minimizes total passenger hours of delay by determining the degree of person delay reduction of several preemption strategies using

both simulation techniques and field demonstration and testing.
Performing Organization: New Jersey Department of Transportation, Trenton, N.J. 08625
Expected Completion Date: June 1980
Estimated Cost: $112,000 (HP&R)

FCP Category 4—Improved Materials Utilization and Durability

FCP Project 4A: Minimize Early Deterioration of Bituminous Concrete

Title: Predicting Moisture-Induced Damage to Asphaltic Concrete—Field Evaluation Phase. (FCP No. 44A2274)
Objective: Accumulate data predicting moisture-induced damage to asphaltic concrete—field evaluation phase. A 1,000-ft (305 m) section of 1975 AC pavement will be evaluated. Data includes precipitation, temperature, traffic, soil layer properties, and condition. Periodic cores over 5 years will be tested for tensile split and resilient mod.
Performing Organization: Montana Department of Highways, Helena, Mont. 59601
Expected Completion Date: June 1981
Estimated Cost: $93,000 (HP&R)

Title: Optimization of Design of Asphaltic Paving Mixtures. (FCP No. 44A2284)
Objective: Optimize mixtures design formulation for stability, durability, rutting, fatigue life, and overall field performance; evaluate mixture moduli characteristics, using the modulus of resilience approach for implementation of pavement design and management techniques; and obtain field verification of results and incorporate the results into a pavement management system.
Performing Organization: Ohio State University, Columbus, Ohio 43210
Funding Agency: Ohio Department of Highways
Expected Completion Date: September 1978
Estimated Cost: $84,000 (HP&R)

FCP Project 4C: Use of Waste as Material for Highways

Title: Recycling of Asphalt Concrete. (FCP No. 44C3043)
Objective: Evaluate procedures to recycle asphalt concrete for surface restoration, as hot mix and as base material, both treated and untreated; develop laboratory design procedures; investigate additives to improve mixtures; construct test sections; and perform long term evaluation of performance.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: June 1981
Estimated Cost: $164,000 (HP&R)

FCP Project 4F: Develop More Significant and Rapid Test Procedures for Quality Assurance

Title: In Situ Determination of Permeability of Base and Subbase Courses. (FCP No. 34F1273)
Objective: Develop apparatus and procedures for measuring the permeability of granular materials in place so that base and subbase specifications can be established to include permeability requirements.
Performing Organization: West Virginia University, Morgantown, W. Va. 26506
Expected Completion Date: May 1978
Estimated Cost: $152,000 (FHWA Administrative Contract)

FCP Project 4G: Substitute and Improved Materials to Reduce the Effects of Energy Problems in Highway Costs

Title: Materials and Techniques for Improving the Engineering Properties of Sulfur. (FCP No. 34G1043)
Objective: Investigate and develop sources of materials and suitable processes that can be used to improve the engineering properties of sulfur to provide sulfur cements as alternates for asphalt and portland cements.
Performing Organization: Southwest Research Institute, San Antonio, Tex. 78284
Expected Completion Date: December 1978
Estimated Cost: $233,000 (FHWA Administrative Contract)

Title: Evaluation of Wood Lignins as Substitutes for Asphalt. (FCP No. 34G1053)
Objective: Develop binders from wood lignins or wood bark for use as substitutes or extenders for asphalt cement in flexible paving mixtures. Work includes study of technical and economic feasibility, development of methods to convert the materials to insoluble binders, evaluation of binder properties, and development of thermoplastic concrete mixture design procedures using such binders.
Performing Organization: University of Washington, Seattle, Wash. 98195
Expected Completion Date: December 1978
Estimated Cost: $238,000 (FHWA Administrative Contract)

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

FCP Project 5C: New Methodology for Flexible Pavement Design

Title: Pavement System Evaluation of Alaskan Highways. (FCP No. 45C3372)
Objective: Evaluate the performance of roadways with respect to their ability to carry the maximum allowable axle loadings at all times of the year; determine the required thickness and properties of each layer in the system for climatic regions of Alaska.
Performing Organization: Alaska Department of Highways, Juneau, Alaska 99801
Expected Completion Date: June 1979
Estimated Cost: $130,000 (HP&R)

FCP Project 5D: Structural Rehabilitation of Pavement Systems

Title: Road Roughness Performance of Pavements. (FCP No. 45D4072)
Objective: Identify factors contributing to roughness, measure pavement profiles through service life, determine seasonal and temperature effects, and evaluate improved data reduction techniques.
Performing Organization: Kentucky Department of Transportation, Lexington, Ky. 40508
Expected Completion Date: June 1980
Estimated Cost: $179,000 (HP&R)

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

Title: Development of Methods to Control Pavement Faulting. (FCP No. 45E2022)
Objective: Establish guidelines for modification of the structural design and/or construction procedures to minimize faulting, develop methods to provide protection against the recurrence of faulting after grinding existing pavements, and explore new methods of correcting faulting.
Performing Organization: California Department of Transportation, Sacramento, Calif. 95814
Expected Completion Date: June 1981
Estimated Cost: $190,000 (HP&R)

FCP Project 5F: Structural Integrity and Life Expectancy of Bridges

Title: Retrofitting Procedures for Fatigue-Damaged Full-Scale Welded Bridge Beams. (FCP No. 55F2022)
Objective: Undertake further work on peening the weld toe and applying a gas tungsten arc remelt process; examine the fatigue strength of beams with cracks that have subsequently been retrofitted by peening and by drilling holes at the crack tips.
Performing Organization: Lehigh University, Bethlehem, Pa. 18015
Expected Completion Date: August 1978
Estimated Cost: $150,000 (NCHRP)

Title: Steel Bridge Design Criteria to Help Minimize the Probability of Fracture. (FCP No. 45F2292)
Objective: Establish better and more complete welding requirements for bridge steels of yield strengths ranging from 36 to 100 ksi (248 to 689 MPa). Define limits of acceptability for various types of geometrical discontinuities and allowable design stress; define future research needs.
Performing Organization: University of Illinois, Urbana, Ill. 61801
Funding Agency: Illinois Division of Highways
Expected Completion Date: June 1979
Estimated Cost: $68,000 (HP&R)

Title: Polymer Impregnation of Prefabricated Prestressed Structural Deck Members. (FCP No. 35F3033)
Objective: Use existing research data to design and construct a bridge using precast polymer impregnated concrete deck members. Evaluate alternate designs for both full and partial impregnation and select the most economical design.
Performing Organization: Federal Highway Administration, Region 10. Portland, Oreg. 97204
Expected Completion Date: November 1977
Estimated Cost: $75,000 (NCHRP)

Title: Design Criteria for Post-Tensioned Anchorage Zone Bursting Stresses. (FCP No. 45F3722)
Objective: Develop a state-of-the-art report; conduct analytical and laboratory studies; develop reinforcing concepts; and develop recommendations for specific design criteria for post-tensioned anchorage zone tensile stresses.
Performing Organization: University of Texas, Austin, Tex. 78712
Funding Agency: Texas Highway Department
Expected Completion Date: August 1979
Estimated Cost: $88,000 (HP&R)

Title: Weigh-in-Motion Instrumentation. (FCP No. 35F4012)
Objective: Design and build an automated instrumentation system which uses a highway bridge for continuously monitoring truck type, arrival time, headway distributions, vehicle velocities, lane occupancy, vehicle gross weight, individual axle weights, axle spacing, and dynamic loads for vehicles over 10,000 lb (4,536 kg).
Performing Organization: Case Western University, Cleveland, Ohio 44106
Expected Completion Date: August 1977
Estimated Cost: $109,000 (FHWA Administrative Contract)

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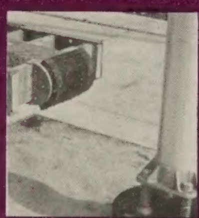

The Development of Acceptance Test Criteria for Frangible Sign and Luminaire Supports

Cement Concrete Roads in Belgium

public roads

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