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U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

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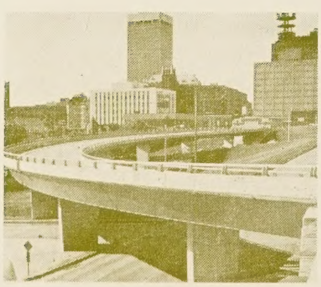

COVER

I–480 exit ramp, Omaha, Nebr., the first trapezoidal unsymmetrical steel box girder bridge completed in the United States. (Photo courtesy of Nebraska State Highway Department.)

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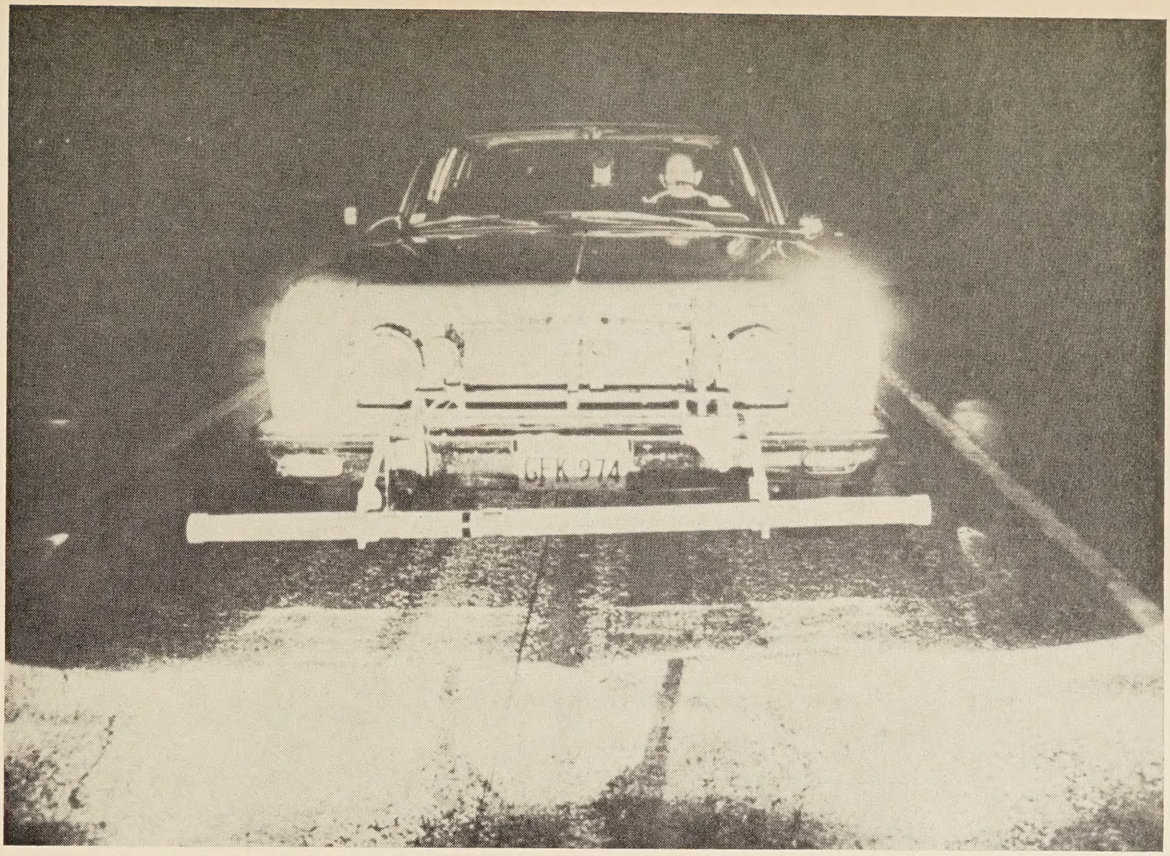

Test vehicle used in research.

Improvement of Visibility for Night Driving

BY THE
OFFICE OF RESEARCH

Reported by[1] RICHARD N. SCHWAB, Electrical Engineer, Federal Highway Administration; and ROGER H. HEMION, Manager, Transportation Research Section, Southwest Research Institute

Various highway vision targets and the effect of various roadway lighting configurations on driver performance are summarized in this article. It appears that a polarized headlighting system is the most promising system likely to solve night visibility problems. This system, technically and economically feasible on today's vehicle population, offers improved visibility with less glare for motorists. Moreover, using this system should result in increased vehicular control, safety, and comfort, and probably improved traffic flow at night.

Introduction

FOR many years the Federal Highway Administration has been studying ways to improve visibility for night driving (*1, 2, 3, 4,* and *5*).[2] Two methods—fixed source lighting and automotive headlamps—are currently used for roadway illumination. Fixed source illumination systems are comparatively expensive; therefore, they are usually considered applicable to only the higher volume, urban areas. At the present time only a small fraction of the total surfaced road and street mileage has some form of fixed illumination.

Although urban driving accounts for approximately half of all travel and is still increasing, the absolute magnitude of travel on the other parts of the highway network is also increasing. One estimate suggests that the amount of travel on the 2-lane rural highway system in the United States will increase about 50 percent within the next 20 years (*6*). Obviously, headlamps must still be relied on for night driving.

The purpose of this article is to summarize 15 recent research and development studies having two broad objectives: (1) to show the effects of glare and its consequent restriction of visibility during darkness upon traffic dynamics, especially on 2-lane rural highways; and (2) to recommend a system of vehicular illumination which could provide an improved visual environment for night driving.

The remainder of this article will first detail the nature of the night visibility problem and then describe a series of studies designed to meet the objectives first stated. As a result of these studies, it appears that a polarized headlighting system might be a promising solution. Therefore, past objections to the polarized system are outlined and means for overcoming them suggested.

[1] Presented at the 50th Annual Meeting of the Highway Research Board, Washington, D.C., January 1971, and received *HRB Award* (for outstanding merit). Award presented to authors at 51st annual meeting, January 1972.

[2] Italic numbers in parentheses identify the references listed on page 267.

Nature of the Problem

Headlight design is currently based on a compromise between the need for adequate road illumination and the need to avoid dazzling the eyes of oncoming drivers with glare light. During the past half century a number of modifications have been made in the control of headlamp beam configuration. These changes in intensity, beam pattern, and aiming have improved the visual environment for night driving. Changes in beam configuration alone, however, cannot possibly provide adequate lighting to enable the driver to operate his vehicle safely under many driving situations. This is especially true on 2-lane highways where there is an extremely small angular separation between approaching vehicles and your own lane. This condition and the variation in possible roadway geometry make it necessary to radically limit the luminous output in the upper left quadrant of the headlamp so that glare does not impair the visibility of oncoming motorists. But some luminance must be aimed in this direction so drivers can see to the left and can negotiate lefthand curves. Also, the designer is limited by physical considerations, such as filament size, in determining how sharp this cutoff can be. Improving visibility by changing beam configuration is therefore extremely difficult.

Reducing headlight glare improves night visibility, and this can be expected to help reduce accident frequency. Although this is perhaps the major benefit, many other aspects of nighttime driving, interrelated to accidents, should benefit from better road visibility. Increased time for driver decision making, a result of increased distances at which road obstacles can be detected, is one example. Additionally, it should be possible to safely maintain somewhat higher driving speeds. This would allow for increased traffic flow and greater use of highways during off-peak nighttime hours. Although an improved headlighting system might produce an increase in night speed, this appears unlikely considering the small and directionally variable shifts in speeds encountered after installing fixed illumination systems. Eliminating glare would reduce stress, reduce the sensation of tunnel driving with its pressures for more exact control of lateral position, and improve the night driving capability of older drivers whose glare adaption responses have deteriorated.

Beam Usage Study

To provide background giving the extent of the problem and to provide data for benefit-cost analyses, a nationwide survey of headlight beam usage practices was conducted (7) at the 17 test sites in 15 States shown in figure 1. Each site was 1,200 feet long and located in most instances on rural, 2-lane unlighted highways of moderate traffic volumes. Three sites, however, differed. Site 15 was a 4-lane freeway with a median 50 feet wide, site 16 was a 2-lane high volume rural road carrying a large proportion of recreational traffic, and site 17 was a suburban, 2-lane road with overhead lighting. One of the rural, 2-lane sites (site 7) was observed under two climate conditions: snowy winter and clear spring weather.

Observers at each end of the test site recorded the type of vehicle and headlight configuration on an event recorder chart with the vehicle position information. The instrumentation package automatically recorded time/position of the passage of individual vehicles in normal traffic on the same recorder chart.

The observations showed that during normal nighttime clear weather conditions over 75 percent of the drivers were using their headlights improperly; that is, using low beams when high beams should have been used because they were neither meeting another vehicle nor following closely behind one.

To obtain sufficient data within a reasonable study time only sites having an average daily traffic volume of 3,000 to 5,000 vehicles were selected. This resulted in minimum nighttime volumes of approximately 20 vehicles per hour. Consequently, observed beam usage patterns must be projected to

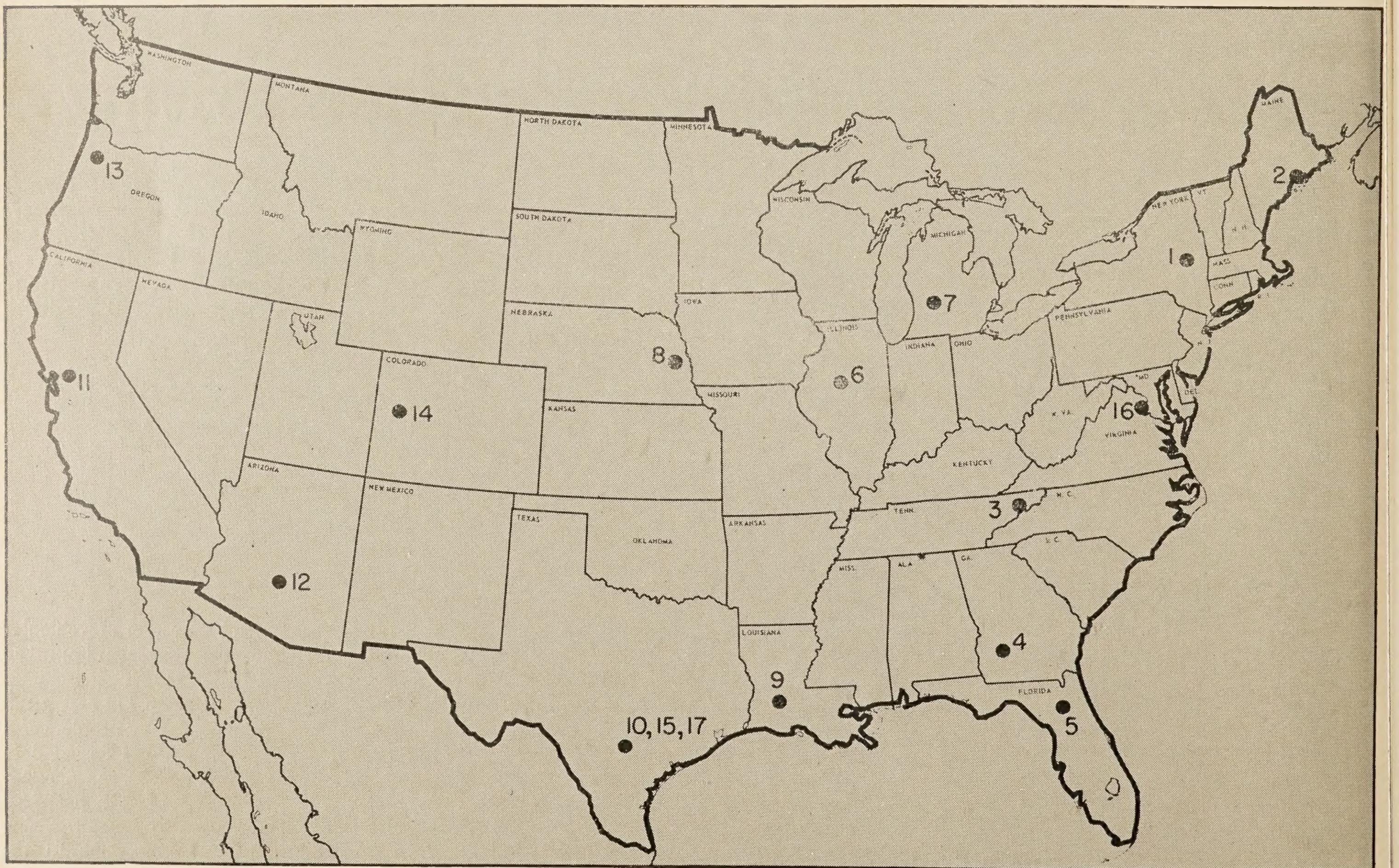

Figure 1.—Site locations for public-beam-usage studies.

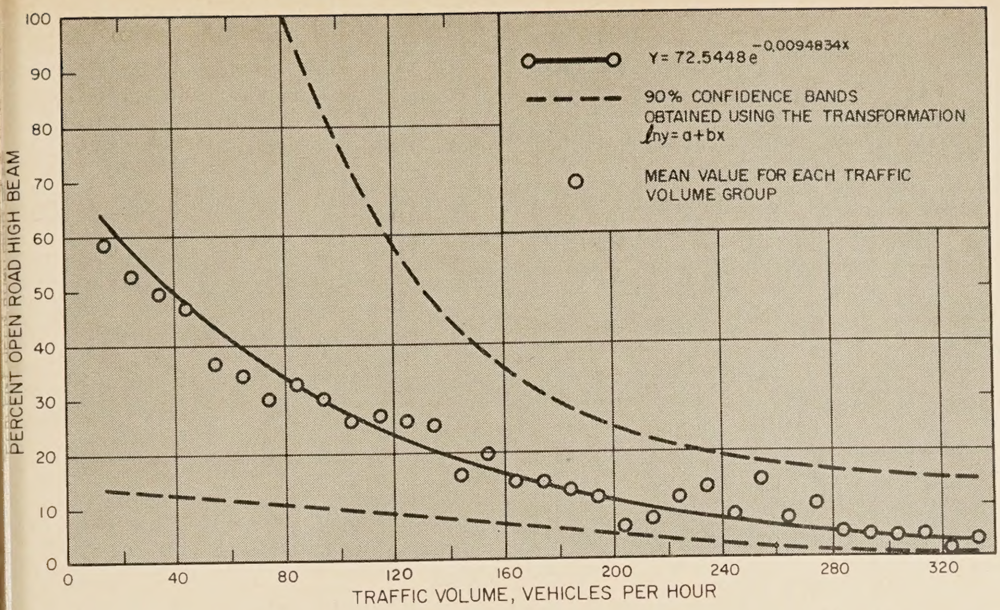

Figure 2.—High-beam usage related to traffic volume.

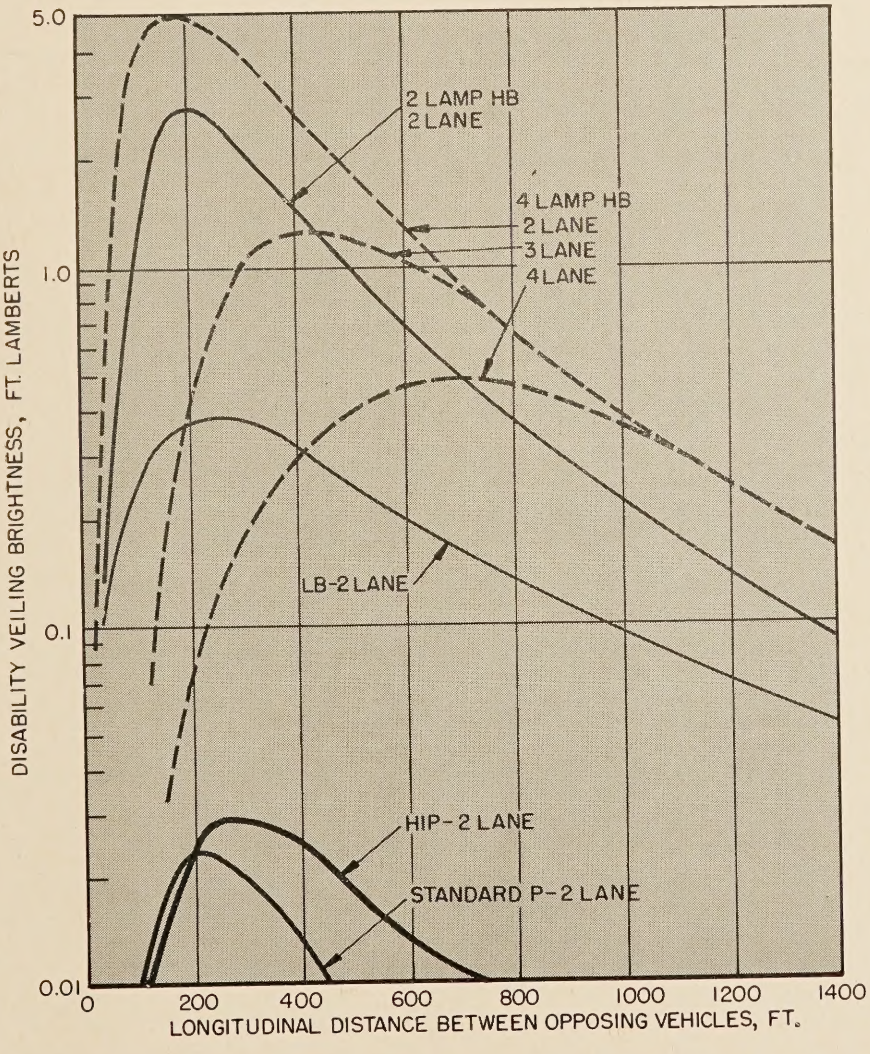

Figure 3.—Headlight disability veiling brightness on 2-, 3-, and 4-lane highways.

apply to lower volume roads not studied and prorated among all roads in order to estimate the beam usage pattern on all rural highways.

High beam usage

Theoretically, the average driver is not conscious of traffic volume as such but is aware of the time intervals between meetings with opposing vehicles. When this time interval is consistently too short, the driver will not switch back to high beam between meetings and will drive continuously on low beam. To evaluate this theory and determine the effect of traffic volume on beam usage when no other vehicle was in sight, traffic volumes were recorded for each 15-minute interval of the study. The intervals were classified by volume in increments of 10 vehicles per hour and the percentage of high beam usage plotted (fig. 2). As can be seen, 50 percent high beam usage did not develop until the traffic volume had dropped to less than 30 vehicles per hour; that is the driver sees an opposing vehicle at greater than 2-minute intervals.

If a driver in traffic meets 30 vehicles per hour, the traffic volume counted at a fixed position along the roadway will be 15 vehicles per hour in one direction, provided that all traffic is uniformly distributed and moving at the same average speed. If traffic volumes in each direction are equal, the two-way traffic volume will be 30 vehicles per hour or the same volume as met by the moving observer. Thus if the above assumptions are met, the traffic volume in figure 2 may be interpreted in two ways: (1) The number of opposing vehicles that a moving driver meets, or (2) the two-way traffic volume counted at a fixed station.

The confidence bands bounding the regression line (fig. 2) indicate a wide variation in beam usage from one observation to another at any specific traffic volume. However, figure 2 does provide a reasonable method for making a projection because the mean values for all observations at any given volume grouping fall closely about the regression line. For benefit-cost and similar purposes, the mean value is of principal interest rather than beam usage for any individual observation. The fraction of high beam usage for each volume grouping was multiplied by the fraction of nighttime travel occuring in that volume category. The result was a corrected estimate that approximately 39 percent of all unopposed driving at night in rural areas is done on high beams and the remaining 61 percent on low beams.

Visibility Studies

Because the greater share of rural night driving is done on low beams, it is important to determine the effect of both low and high beam headlamps on the ability of drivers to detect typical highway visual targets. A simulated roadway was established on a 5,000-foot asphalt runway of an inactive airfield (*8*). Two vehicles were instrumented to provide a continuous record of their position both longitudinally and laterally on the simulated road. The instruments also provided data on target detection by the driver and glare intensity faced by the driver. From the position data, the distance between opposing vehicles could be determined together with the distance of the driver from a target at the point when he detected the target.

Variation of disability veiling brightness (DVB) with longitudinal distance between opposing vehicles with conventional headlights on 2-, 3-, and 4-lane roads is shown in the upper curves of figure 3. DVB is a photometric measure of the glare effect produced by all the luminances in the field of view. DVB is an expression of the equivalent veiling or uniform luminance which could be superimposed over some visual target to produce the same loss of visibility as that resulting from all the glare sources in the field. Two characteristic effects of increased lateral separation will be noted in figure 3 by comparing the dashed curves for 4 lamp, high beam on 2-, 3-, and 4-lane highways. First is the reduction in intensity resulting from lateral displacement of the observer from the center *hot spot* of the beam. Second is the movement of peak intensity to greater longitudinal distances between opposing vehicles.

Polarized headlighting system

Several alternative methods have been suggested for reducing headlight glare. Many of the systems involve some type of screening or planting between vehicles moving in opposing directions. But these systems are limited to divided highways and cannot be employed on the very roads where they are critically needed—2-lane rural highways. Another possibility is to use controlled headlamp beam distributions that optically project the beam straight ahead of the vehicle without impinging on the opposing lane. Unfortunately, highways are not always straight and unless expensive feedback control systems are employed, it is impossible to keep from blinding opposing drivers at some point in a curving highway situation. Of course, as was suggested earlier, a continuous fixed source illumination system could be used, but this alternative is expensive.

One of the most promising systems involves linear polarizers combined with high wattage light sources (*9*, *10*, *11*, and *12*). Such a method could theoretically provide an illumination system with greatly increased road visibility without causing direct glare to oncoming motorists. This is indicated in figure 3 by the much lower level of disability veiling brightness for the polarized systems.

The polarized system (fig. 4) relies on the principle that two linear polarizers with their planes of polarization perpendicular to each other permit only a negligible amount of light flux to pass the second polarizer. Polarizers are attached to the headlamps having their transmission axes oriented at 45 degrees with the vertical. Another polarizer, called the analyzer, with the same transmission axes—parallel to those over the headlamps—is installed in front of the driver's eyes in a position similar to that of the sunvisor but intersecting the line of sight toward the opposing vehicle. This analyzer is constructed so that it can be moved out of the way when not needed. Because the transmission axes of the headlamp polarizer and the analyzer are parallel, light from the driver's headlamps, reflected from pedestrians, signs, pavement markings, and other parts of the roadside environment, is transmitted through the analyzer to the driver's eye. However, when another vehicle equipped in the same manner approaches, the transmission axes of the polarizers over its headlamps is perpendicular to that of the original vehicle's analyzer. Thus only negligible light is transmitted and neither driver is blinded by glare from the approaching headlamps.

The main portion of the visibility studies involved determining the distance ahead at which each of three targets first became visible. Four specific headlighting systems were used in this phase of the study. The studies included situations with no opposing vehicle (no other vehicle in sight), and meetings of one opposing vehicle that employed the same headlighting system. In preliminary investigations, with both one and three opposing vehicles, it was determined that the increase in intensity and duration of glare caused by multiple opposing vehicles reduced detection distances. However, the number of cars in the platoon caused no differential effects among targets. So the main experiment, employing 20 randomly selected drivers, studied only the *no opposing* and *one opposing* vehicle situations.

Because they were typical of the range of driver detection tasks at night, the following three targets were selected for this study:

Sign target.—A 96-percent diffuse reflective white 2¼-foot square with one quarter missing was seen against a 9-percent diffuse reflective black panel. The bottom of the panel was 60 inches above the ground and the left edge of the sign 6 feet from the edge line of the driving lane. The panel could be rotated and the missing portion of the square used as a target identification task.

Pedestrian target.—A three-dimensional 6-foot tall manikin, its exposed features painted with 17-percent diffuse, reflective grey paint and wearing a 17-percent reflective grey topcoat, was positioned with its right arm 2 feet from the edge line of the driving lane.

Line target.—A portable, reflectorized yellow, 4-inch wide, *no passing* line 100 feet long was positioned in the normal location along the right side of the centerline. The targets are illustrated on the right side of figure 5.

Results of these studies are summarized in figure 5. The left curves indicate little difference in detection distance between conventional high-beam and low-beam headlighting with an opposing glare car that employed the same headlighting mode, except for the *sign* target. When high beam was opposed by high beam, detection distances for the sign were approximately 125 feet further away than when low beam was opposed by low beam. As expected, when no opposing vehicle was present, detection distances were greater with high beam compared to low beam regardless of the target used. When unopposed, low beam detection distances were as good if not better than the meeting situation with high beam. However, the visibility with either high or low beams was generally unsatisfactory and severely limited the time available for taking evasive action. A system allowing visibility equivalent

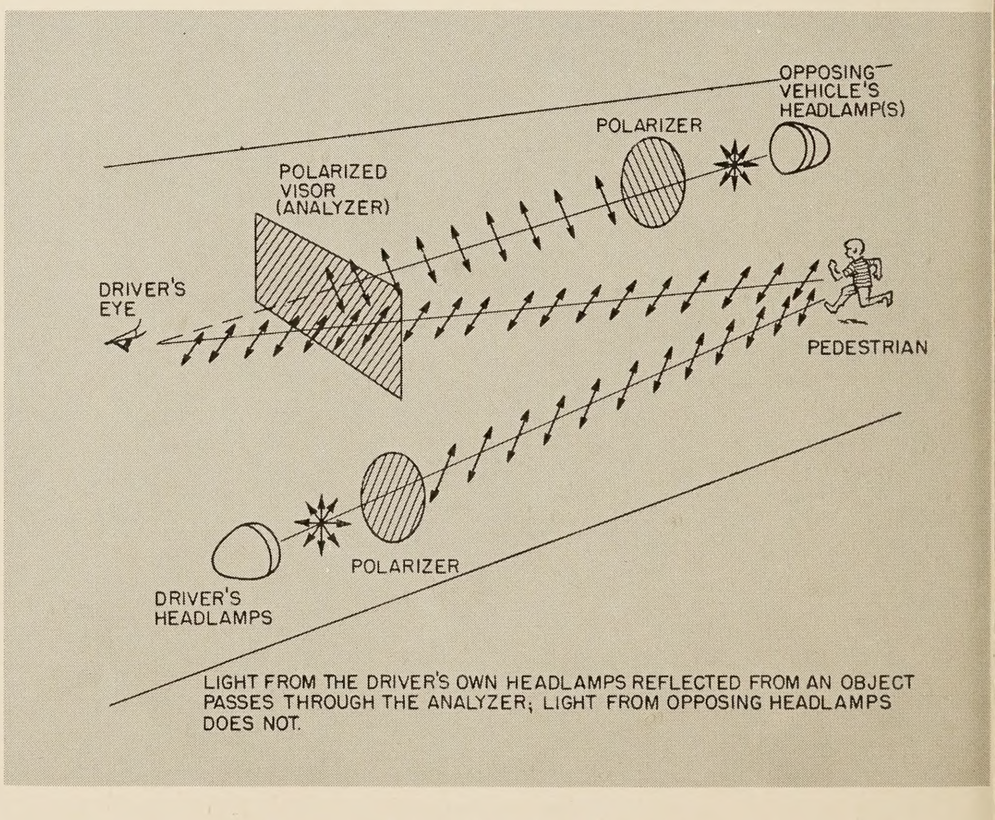

Figure 4.—Polarized headlighting system.

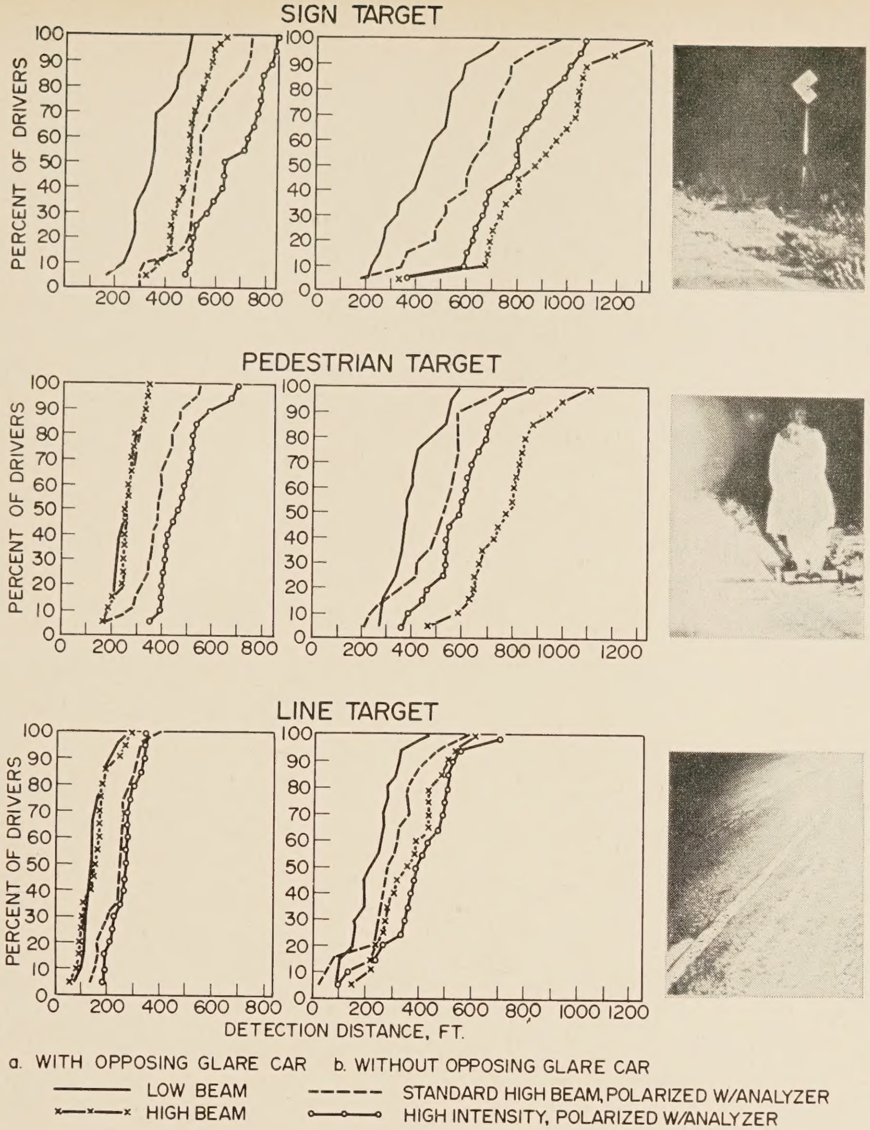

Figure 5.—Distribution of detection distances for all drivers and headlight configurations.

to that of unopposed high beam during the meeting situation was clearly needed.

Increased detection distances with polarized system

When polarization radically reduced glare, the difference in detection distances between opposed and unopposed driving modes was greatly reduced (fig. 5). Using polarization in the opposed mode increased detection distances over those obtained in the unpolarized mode, regardless of target or intensity of the lamp used with the polarization. In the unopposed mode, increased detection distance occurs even if the analyzer is not used. It should not be used when there are no opposing headlights, and if the polarized illumination is equal to or greater than the unpolarized standard headlamps (criteria used in the design of the High Intensity Polarized [HIP] system), then visibility without the analyzer and with no opposing vehicle would be equivalent to that of high beam for most target types. However, for the unopposed mode, detection distances (fig. 5) for the polarized headlight cases included the analyzer because randomly presenting test factors to the subjects precluded the omission of the analyzer for unopposed trials. Also, this constitutes the worst case which can result if the driver becomes lazy and fails to move the analyzer from his line of sight when no opposing vehicles are present.

Substantially higher safe driving speed

How does a polarized system affect the safety of the average motorist? By using the data on visibility of a pedestrian target at the road edge for each headlight system (fig. 5), it is possible to calculate (*13*) a safe speed from which the normal (median) driver would be able to stop before striking the pedestrian. The results of this calculation are given in table 1.

The average speed of over 50,000 vehicles observed during the beam usage study was 60.5 m.p.h. on dry pavements and 55.4 m.p.h. on wet pavements. Thus the great majority of the drivers of these vehicles, approaching 98 percent under some road conditions, exceeded a speed from which they could safely stop if a pedestrian stepped out onto the road. Pedestrians account for approximately 10 percent of the fatalities on rural highways. In the absence of an opposing glare car, there is a minor reduction in the safe speed in going from high beam to high intensity polarized headlights with analyzer. However, this reduction can be compensated for by not using the analyzer when there is no opposing vehicle. The major effect is observed in the case of opposing traffic where the elimination of glare (disability veiling) inherent in the polarized system results in a marked increase in safe speed levels.

The foregoing discussion is based on using high intensity polarized lamps. Using existing high beam lamps with polarizers results in safe speed levels about halfway between that of conventional headlamps and the high intensity lamps when an opposing vehicle is present. Without an opposing vehicle, the safe speed would be less than conventional high beam but well above that of low beam. Overall, using existing high-beam lamps with polarizers (no increase in lamp power) improves the night driving environment over that of conventional headlamps in all cases, except for unopposed driving where high-beam lamps are now used—39 percent of the unopposed driving. However, the improvement would not be sufficient to raise the safe speed for meeting situations to the level required to see and stop for a pedestrian at current highway speeds.

Table 1.—Safe speed for stopping before striking a pedestrian when first detected

Opposing vehicle	Pavement	Conventional headlamps		High intensity polarized headlamps	
		Low beam	High beam	With analyzer	Without analyzer
		(m.p.h.)	(m.p.h.)	(m.p.h.)	(m.p.h.)
Yes	Dry	40	40	63	(2)
Yes	Wet	35	35	50	(2)
No	Dry	50	80	[1] 73	80
No	Wet	40	65	[1] 58	68

[1] With analyzer in use—if the driver does not use analyzer when there is no opposing vehicle, safe speed would be as indicated in *without analyzer* column.

[2] Not applicable, polarized system requires use of analyzer when meeting opposing vehicle. If not used it is the same as high beam.

Dirty windshields and polarizer misalinements

Detection capability for drivers using polarized lights is reduced by a dirty windshield not only because of reduced light transmission but also because the dirt particles are illuminated by the opposing headlights and interpose a more or less bright screen which effectively reduces the contrast between the target and its background. No appreciable depolarization was noted from the dirty windshield (*8*). Table 2 shows that the visibility loss with a dirty windshield, employing the high intensity polarized lamps, was about 6 percent compared to 10 percent with conventional headlights. This result does not imply that driving with a dirty windshield should be ignored. This study only involved the effect of the dirt on detection capability, and the problems related to the distraction and confusion caused by dirt on the windshield were not considered.

Some misalinement of the polarizers between the two approaching vehicles is bound to exist because of road crown, superelevated curves, and other causes of vehicular roll movement, including rough pavements and pot holes. Because of the nature of polarization, misalinement increases the light transmission. Dynamic studies of vehicular roll have shown that a practical maximum of 7 degrees occurs in moving vehicles on normal paved highways. Figure 6 shows that a misalinement of more than twice this value, or 15 degrees, has an insignificant effect on the driver's detection capabilities (*14*). It is therefore concluded that superelevated curves and other causes of vehicular roll will not adversely affect a polarized headlighting system.

Comfort

The discomfort caused by glaring headlights is essentially subjective and not necessarily accompanied by a corresponding visual disability. Visual discomfort was rated on a 6-point scale by asking the subjects, at the completion of each test, for a judgment of comparative discomfort caused by opposing headlamps when meeting another vehicle head-on (*8*). Figure 7 indicates that there was a definite correlation between subjective ratings of glare and measured disability glare levels. Both polarized systems produced only about one-third the discomfort (2 points less) than high beam and about half (1 point less) than low beam.

Table 2.—**Detection distances as affected by dirt on windshield**

Windshield	High beam	High intensity polarized
	Feet	*Feet*
Clean	590	660
Dirty	530	620

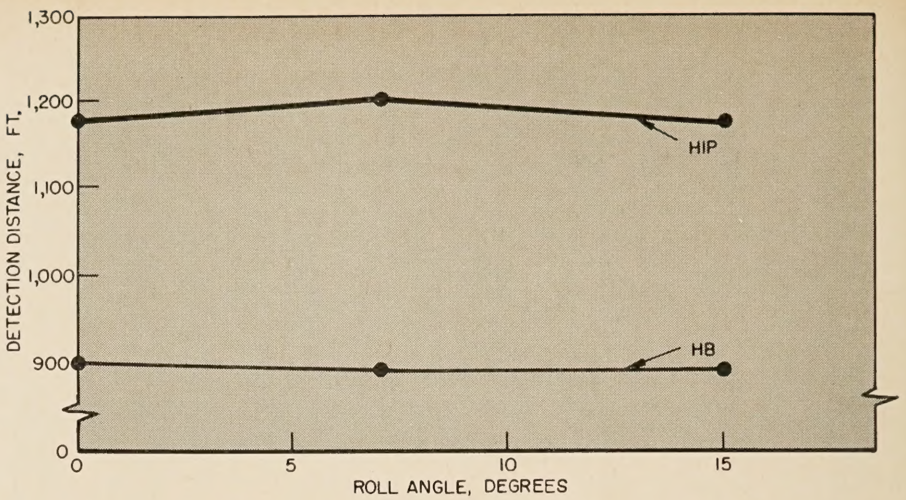

Figure 6.—Mean target detection distances as affected by vehicular roll.

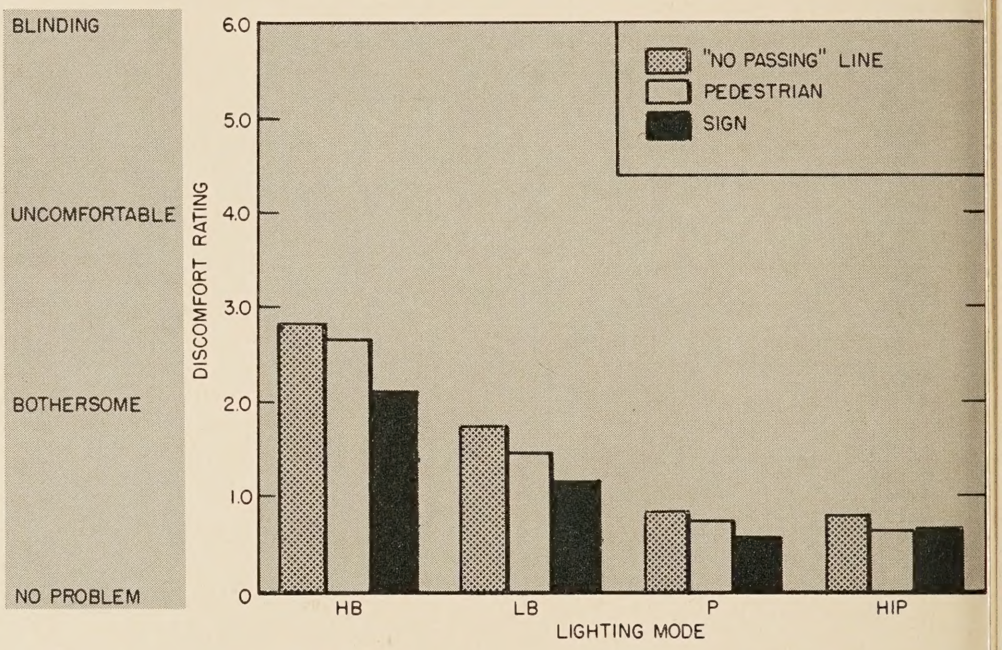

Figure 7.—Relative discomfort from opposing headlight modes.

Fatigue Studies

A reduction in driving tension and physiological and psychological irritation of meeting headlights of approaching traffic was subjectively apparent to all subjects. It was postulated that driving a vehicle for extended periods, when all opposing traffic was controlled with regard to headlight modes, would induce fatigue at differential rates, depending on the stressful characteristics of the specific lighting modes. It was further expected, from previous research (*15*), that sufficiently sensitive indicators of such fatigue propagation were available to discriminate among the comparative fatigue or stress inducing potentials of the various headlighting modes.

Initially, the course used for the visibility studies was extended by utilizing an adjacent parallel runway and an intersecting runway and taxiway to provide a loop course approximately 1¾ miles long. Real and simulated vehicles were used to provide opposing traffic (headlights) for the subject drivers. Fatigue development in the observers was evaluated in terms of their performance of several physiological and psycho-physiological tasks presented at the beginning and end of each test session or periodically during the session.

Simulated driving as well as road simulation was employed in the second part of the fatigue study. For the most part laboratory instrumentation was used to evaluate subject psychophysiological, physiological and vigilance responses. The subject, in the driving seat of a stationary automobile, was given a tracking task in the form of a meter *nulling* response to random input steering offset error

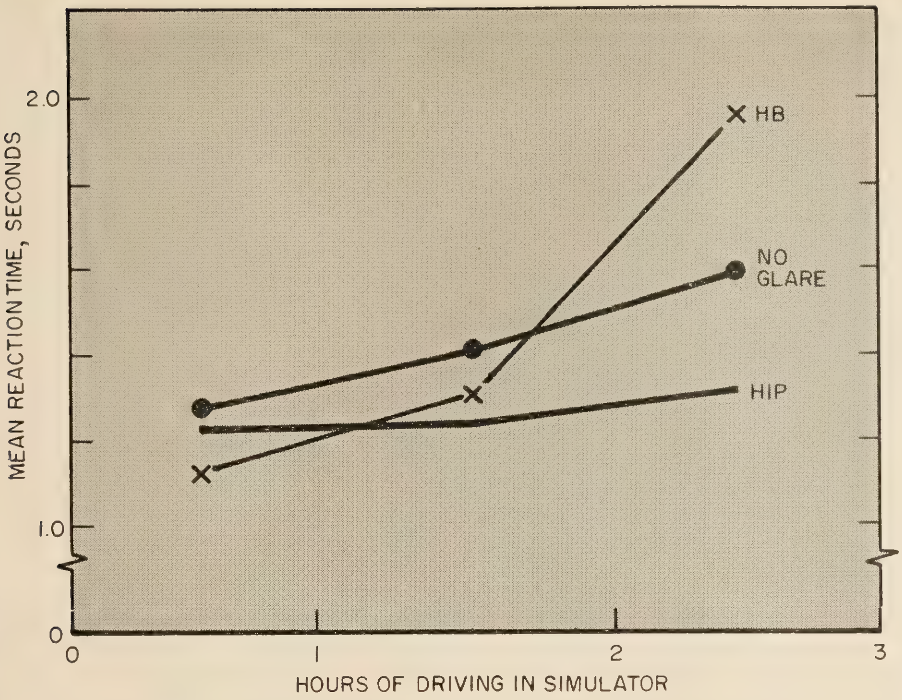

Figure 8.—Effect of lighting conditions on reaction time.

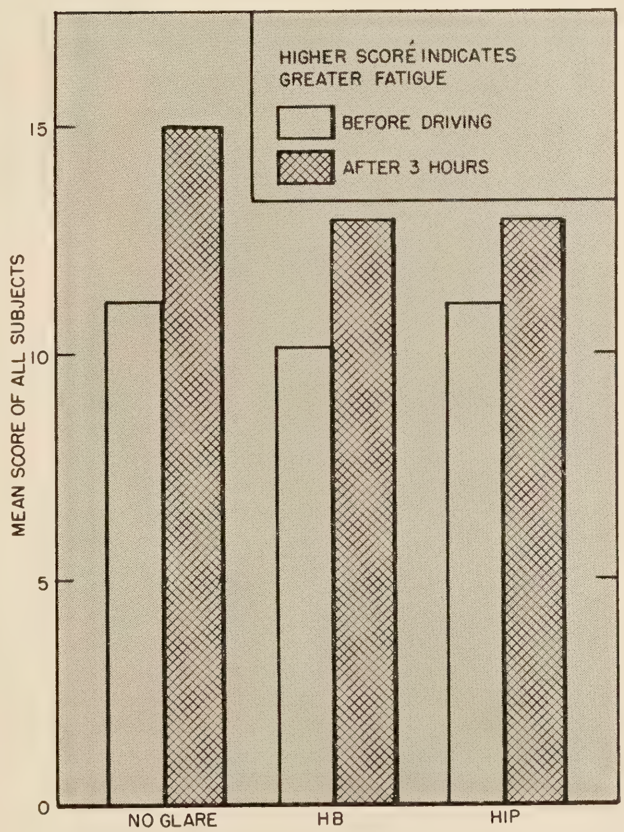

Figure 9.—Subjective fatigue mean score, before and after simulating driving at each lighting condition.

signals. Random visual flicker and audio flutter presentations required the driver to lift his right foot off the accelerator and then apply the brake. These tasks were conducted during repeated exposures of the headlight modes being studied, and each driver's performance was measured.

These experiments, in both the real and simulated driving phases, neither supported nor refuted the hypothesis that fatigue affected driver performance differently with respect to the lighting system involved. Although some of the measures and techniques selected to indicate stress or fatigue development showed changes during the tests, in general these changes were not consistent between drivers nor within a single driver's performance on successive replications (*15*). Brake reaction time in response to an unanticipated signal, however, did show some consistency. Figure 8 shows that no appreciable changes in performance occurred for the first 2 hours of the simulated driving session. In the third hour, differences were apparently related not so much to glare but perhaps to discomfort and stimulation. The polarized system apparently developed an optimum stimulus to alert the driver between the *overloading* effect of the glaring high-beam lights and the soporific effect of no opposing traffic.

Using a scaling technique, subjective discrimination of fatigue development related to lighting modes was achieved (fig. 9). The greatest increase in fatigue was obtained in the absence of glare and the least with the polarized system, although the differences were not statistically significant. Thus only an indication of correlation with performance data was obtained.

Driver Behavior

The effect on the lateral positioning of the subject's vehicle in its lane when approaching the target and the meeting point with the opposing glare car(s) was also investigated. A consistent tendency to steer *toward* the oncoming glare car and a distinct reduction in steering deviation was observed with most drivers at an intercar separation distance of about 800 feet (5 seconds from meeting point). This can be attributed to an anticipatory steadying or tightening of the steering control before the approaching encounter, and a sort of staking out of one's territory. Prior to this point, the approaching vehicle does not present a hazard; that is, sufficient time is available to maneuver in the event something unexpected occurs. However, the presence of the opposing vehicle forces the driver to recognize that he is approaching a potentially hazardous situation; therefore, he makes corrections to his vehicle's path by steering the vehicle *away* from the oncoming vehicle to obtain greater lateral separation.

It was hypothesized that the greater stress of opposing high-beam lights causes proportionately greater lateral displacement than reduced glare from low-beam or polarized headlights. Some difference was noted with headlight modes in opposed trials (fig. 10). The opposing vehicle itself appeared to be the principal motivating influence, irrespective of its lighting, that caused the driver to move to the right as the vehicles approached their meeting point. Only in the unopposed trials did the target appear to exert any influence, but with essentially no distinction between the targets.

Judgments of speed, distance, angular relationship, and the relative position of potentially hazardous situations are important to drivers. The ability to make these judgments may be influenced directly by the lighting system used. When it is safe to leave a side road and proceed across a highway is a particularly critical judgement. Studies were made of right angle gap acceptance behavior, using both conventional headlamps and the polarized system (*16*). A recent reanalysis of the study results is shown in figure 11 and in table 3.

Table 3.—Effect of headlighting system on gap acceptance

Headlamp beam	Mean gap accepted	Standard deviation
	Feet	*Feet*
Low	225	94
Polarized, with glasses	245	99
Polarized, with visor	265	87
High	277	99

Headlighting systems, which in the right angle situation produced the most glare, required somewhat longer gaps for crossing. Low-beam and polarized beam with glasses were the low glare situations requiring somewhat shorter gaps for crossing. Polarized beam with visor and high beam were high glare situations and required a larger gap. The variance in the minimum gap sizes accepted appeared to be about the same in all cases. That the polarized system, using a fixed visor mounted in place of the sunvisor, should be characterized as high glare is reasonable in the right angle situation because the driver is not protected against vehicles approaching from the side. An analyzer in the form of glasses did provide protection to the side. However, because the head tips somewhat as it is turned to the side, the protection is not complete and the end result is quite similar to that of low beam.

High glare appears to make drivers behave more conservatively, by judging the brighter sources to be somewhat closer than they are and thereby allowing a somewhat greater margin of safety. This, of course, could have an adverse effect on traffic flow, particularly when a high volume exists on the main road. Fortunately nighttime traffic is seldom near capacity on most unlighted roads where the glare problem is acute. The gap acceptance studies clearly show that if the polarized headlighting system is introduced, care should be taken in the design of the analyzer to protect drivers during encounters with vehicles from the side.

The design of the analyzer may be particularly important for older drivers because the glare tolerance of individuals is increasingly reduced with age.

Fixed Lighting

The detection distance results from low-beam headlighting alone (*17*) are compared with that obtained with the addition of fixed, overhead lighting in figure 12. The major effect of fixed lighting was confined largely to the two vertical targets. The reflectorized *no passing* line was relatively unaffected by the additional illumination because the brightness of the line was largely controlled by the illumination coming from the headlamps. Because of the geometric relationship of line and driver, additional illumination from the fixed lighting sources did not increase the contrast between the line and pavement. In the case of the two vertical targets—the nonreflectorized sign and pedestrian—adding fixed lighting to low-beam headlighting more than doubled the detection distance. This appears to be primarily because of illumination falling directly on the two vertical targets (this will be discussed later).

Apparently little difference exists in the effectiveness of the two illumination levels used—0.6 and 2.0 footcandles average horizontal illumination (fig. 12). The detection distance was also largely independent of the type of vehicular lighting used (table 4). Glare from an opposing vehicle's headlamps caused a reduction in detection distances, particularly when high beams were employed with or without a fixed lighting system.

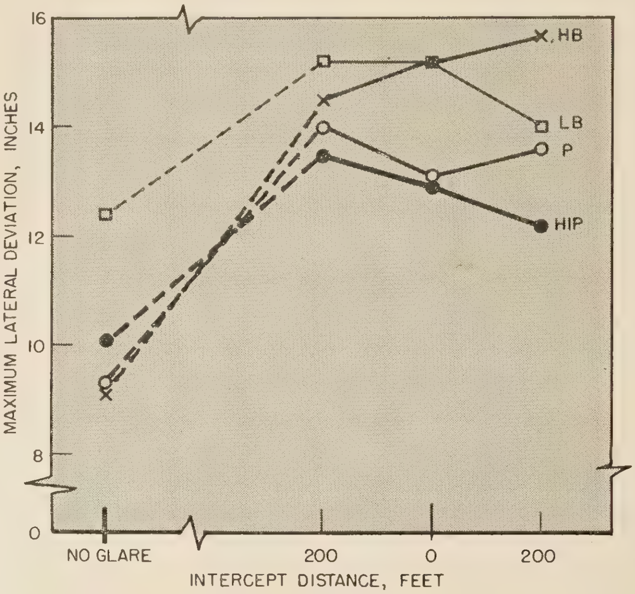

Figure 10.—Maximum lateral deviation as affected by headlamp-beam type.

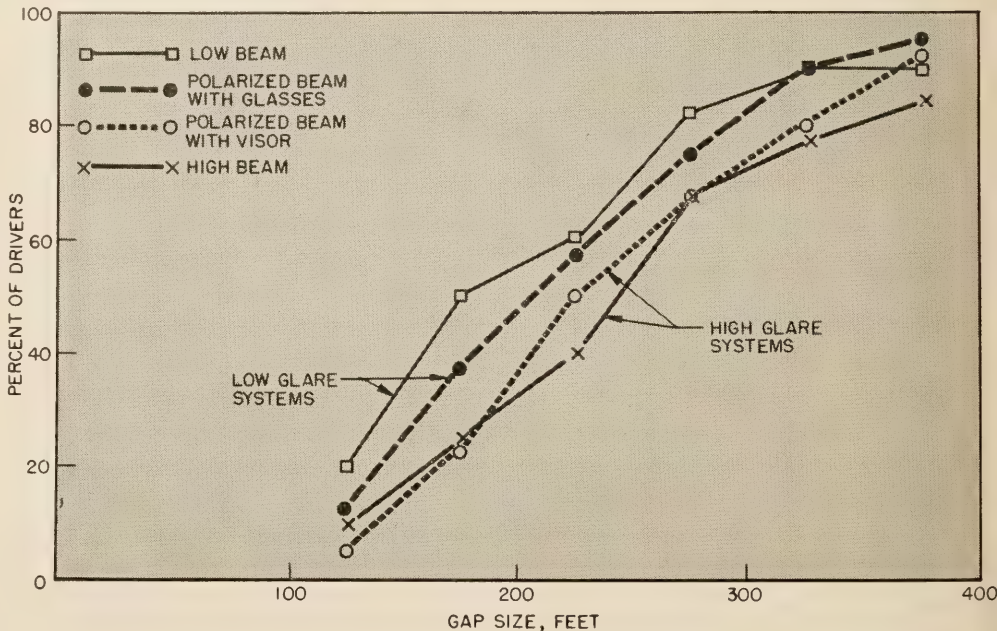

Figure 11.—Distribution of gap acceptance for each lighting mode.

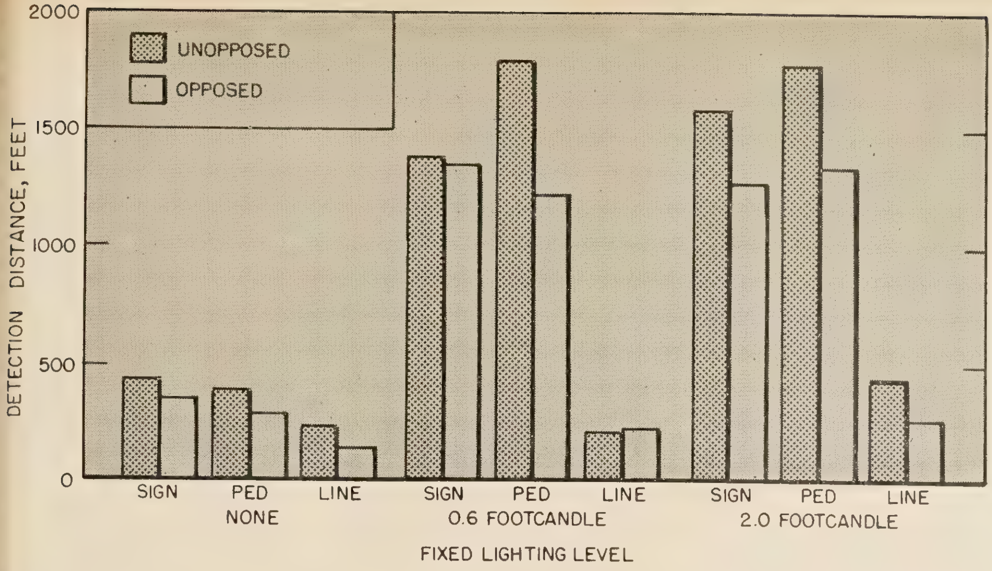

Figure 12.—Detection distance on dry pavement, low-beam headlights and different fixed-lighting levels.

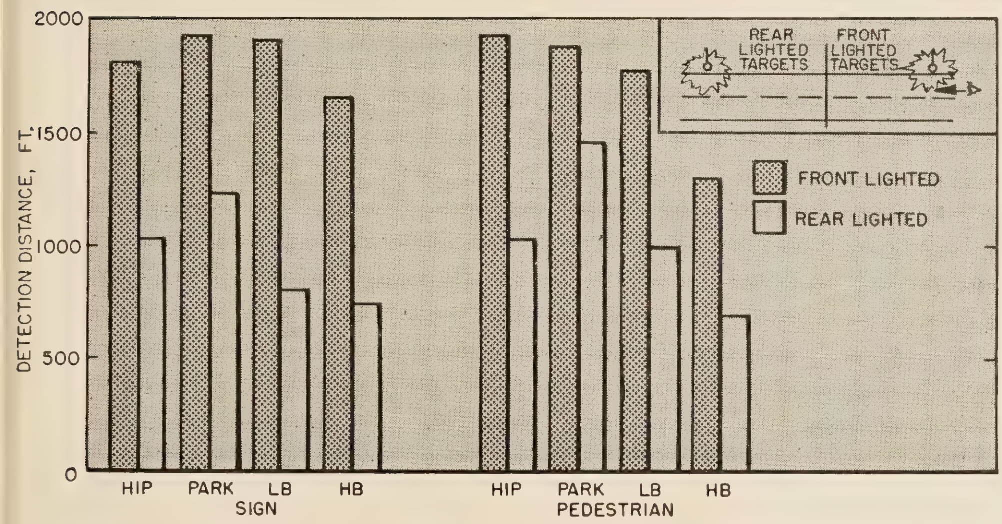

Figure 13.—Effect of target position on detection under fixed overhead lighting.

A much greater effect was observed when the target was forelighted or backlighted by overhead luminaires. Figure 13 shows that front lighting increased detection distance for the two vertical targets (the line target being unaffected by fixed lighting is not shown). The increase varied between 30 and 140 percent with an overall average of 80 percent. The differences were somewhat greater with the two-dimensional sign target than the three-dimensional pedestrian, where the possibility of edge lighting was present. The location of the luminaire is important and should be considered by highway lighting designers when locating luminaires with respect to pedestrian crosswalks. In many instances, the two-way nature of traffic and other considerations make front lighting impossible. However, on one-way streets and/or at mid-block pedestrian crossings, front lighting may be feasible.

Luminaire location had the least effect when the driver used parking lamps. This is reasonable because the target in rear lighting is seen by silhouette, and any illumination reaching the target from the vehicular lighting system only reduces the contrast. This indicates that with adequate overhead lighting levels, motorists may operate safely with only marker lights (perhaps of somewhat large size and intensity) on their vehicles. This would reduce glare to a minimum. However, further investigation relating to the ability of pedestrians and drivers in detecting vehicles employing only marker lamps is needed. Some research in this area is currently under way at Franklin Institute Research Laboratories in Philadelphia.

Eliminating glare by polarization, expanding overhead fixed lighting, or by any other practicable means would greatly improve the ability of drivers to see roadside obstacles, traffic markers, signs, and other objects in the highway scene. The best way to improve visibility appears to be adequate levels of fixed, overhead lighting.

In 1967 there were 1.1 million miles of unlighted, paved, rural highways. The estimated cost for installing and operating fixed lighting at a 2-footcandle average level for the paved road network for a 20-year replacement cycle

Table 4.—Effect of various fixed and vehicular lighting system combinations on mean detection distance

Fixed lighting	None				0.6 ft.-c.				2.0 ft.-c.			
Vehicular lighting	Parking	Low beam	High beam	HIP	Parking	Low beam	High beam	HIP	Parking	Low beam	High beam	HIP
Dry pavement:												
Unopposed:		*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*	*Feet*
Sign	------	420	865	800	1,467	1,360	1,710	1,367	2,092	1,563	1,960	1,783
Pedestrian	------	375	780	590	1,317	1,789	1,564	1,785	1,662	1,762	1,722	1,919
Line	------	215	360	380	177	175	249	290	336	411	582	497
Opposed:												
Sign	------	345	490	630	1,417	1,330	1,191	1,122	1,536	1,260	1,005	1,377
Pedestrian	------	245	245	460	1,743	1,206	860	1,359	1,618	1,353	845	1,205
Line	------	130	135	260	158	197	153	238	340	267	184	424
Wet pavement:												
Unopposed:												
Sign	------	------	[1] 680	[1] 653	------	------	------	------	2,221	1,886	2,220	1,657
Pedestrian	------	------	------	------	------	------	------	------	2,298	1,695	2,008	1,280
Line	------	------	------	------	------	------	------	------	178	209	236	128
Opposed:												
Sign	------	------	------	[1] 563	------	------	------	------	1,671	1,387	1,160	1,315
Pedestrian	------	------	------	------	------	------	------	------	1,722	1,560	991	1,245
Line	------	------	------	------	------	------	------	------	182	160	102	172

[1] Partial series only.

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is $87 billion (*18*). The total cost for fixed lighting would, therefore, approximate the total cost of the Interstate Highway System.

It is clear that this solution is not economically feasible for the entire highway system, although it is justifiable on certain highways. Fixed lighting appears to be warranted where the driver is faced with unpredictable situations in which the added visibility from fixed lighting is of considerable aid. Some factors which may lead to this lack of predictability are heavy concentrations of pedestrians, unusual geometry, and high traffic volumes.

Normal headlighting on unlighted, rural roads still causes glare, and such roads constitute nearly all of the Nation's highway system and carry the largest portion of the traffic. A considerable increase in overhead, fixed highway lighting will not eliminate the need for modification of vehicle headlighting if the objective is improved illumination for operation on the highway at night. Increased headlamp intensity to provide greater visibility while eliminating glare from opposing headlamps remains essential for the bulk of highway operations. The polarized headlighting system, therefore, is essential. It provides substantial improvement in visibility, coupled with a radical reduction in glare on all roads, instead of the partial solution provided by fixed lighting employed only on certain roads.

Objections to a Polarized System

Evolutionary changes in the motor vehicle have made objections to the polarized headlighting system invalid. Laminated glass for windshields is an advancement that eliminates distortions caused by tempered glass. Another improvement is the higher capacity of modern electrical systems, a condition which makes higher powered lamps feasible. Some existing limitations must still be studied and overcome. Chief among these are the reduction of heat generated in the polarizers from the lamp and the transition to a polarized system from the present conventional lamps.

Heat problems

Commercially available polarizing materials (and agents used for bonding them to the lamp) can withstand approximately 150 degrees C. without detrimental effects which reduce light transmission. The maximum power consumption for filaments that fit a standard 5¾-inch lamp is about 125 watts. To obtain this limit, a dichroic reflector must be employed to dissipate the heat. Such reflectors are comparatively expensive. The lamp intensities required to obtain the increased detection distances shown in figure 5 could be provided by four 50- to 75-watt lamps. Standard aluminum on glass reflectors could be used on such a lamp. The major problem of heat dissipation, therefore, would not arise unless longer detection distances than those shown in figure 5 were required for very high-speed operation. Because of the constraints imposed by vehicle power generation systems, it is not likely that heat dissipation will be a problem for any retro-fit system. On new vehicles, if the heat problem can be solved, increased detection distances from very high-intensity lamps might be considered.

Transition to the polarized system

Converting to a polarized system is difficult and requires considerable analysis of alternative methods. With more than 100 million vehicles on the road, it is important to develop smooth, expeditious plans for a transition. It is obviously not possible to equip all vehicles on the road overnight. Therefore,

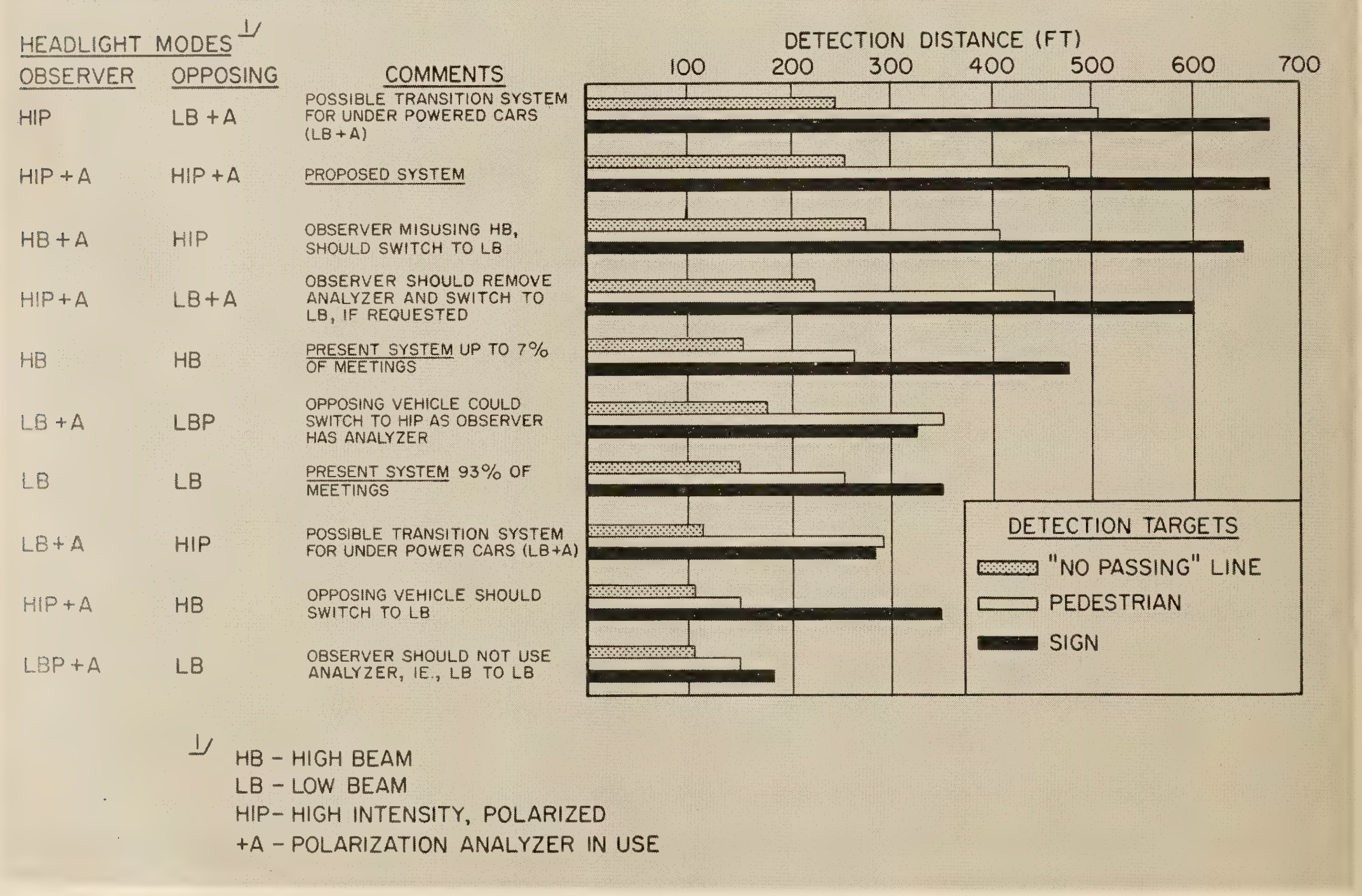

Figure 14.—Mean target detection distances during transition period.

during the transition period, both polarized and unpolarized headlights will be present in traffic. A series of studies was made to determine what effect on visibility such mixed meetings would have (*19*).

Figure 14 shows that in only two instances would visibility conditions in a meeting between a modified and unmodified vehicle be significantly worse than those presently encountered with low beams—93 percent of all meetings occur with low beam. In both situations, the driver with polarized equipment would be able to quickly recognize that because he is seeing white lights—the sign of conventional unpolarized lamps—he is at a disadvantage. He could then simply move the analyzer out of his viewing field and revert to a conventional meeting situation with the same visibility; that is, no worse than the present low beam to low beam meeting.

Several conversion schemes for the transition are possible, and only by additional research and field testing will it be possible to develop the necessary information for wisely choosing among them. A scheme which is feasible, but may not be the optimum solution, is as follows:

- During the first 3 years of the conversion, all new vehicles would be equipped with a polarized headlighting system. The conventional system would continue to be used for all meetings between vehicles which are not equipped; but where both vehicles are equipped, the polarized system can be used. Therefore, if one vehicle in a meeting did not have the polarized system, the driver would flash his lights and the meeting would occur on low beam as at present. Unopposed driving would be either with high intensity polarized beams for new vehicles or high beam for used vehicles.
- At the end of the third year, all drivers would be required to install analyzers in their vehicles. Used vehicles would be required to be equipped with the full system including the polarized lamps on title change. All new vehicles would continue to be equipped. Vehicles equipped with the polarized system would use it for meeting situations. All other vehicles would use low beam plus their analyzer.
- At the end of the sixth year, conversion of all remaining vehicles would be required.

This means that for the program's first 3 years, owners of new vehicles would have the system and obtain some limited benefit when meeting another new vehicle. By the end of the third year, approximately one-third of all vehicles would be equipped (*20*). Because newer vehicles tend to be used more than older vehicles, equipped vehicles would account for perhaps 45 percent of all oncoming traffic. Fitting all vehicles with the analyzer at the end of the third year would increase the benefit to those with fully equipped vehicles and speed the voluntary conversions. By requiring retrofitting of used vehicles on resale for the next 3 years, conversion of the remaining vehicles is speeded up, and by the end of the sixth year more than three-quarters of all vehicles would be fully equipped. These vehicles will account for 85 to 90 percent of all meetings.

Equipment for new vehicles would cost the owner approximately $30 (*19*). To retro-fit used vehicles would cost between $26 and $45 per vehicle, depending on the type of headlighting system presently on the vehicle. Modification of existing vehicles is possible in most cases by any driver who is capable of making simple repairs.

Because of their low electrical power capacity, a few vehicles probably could not be converted at these costs. Such vehicles could be equipped with an analyzer only and allowed to operate on low beam. Visibility would be approximately the same (fig. 14).

Conclusions

The research described here has demonstrated that: Polarization appears to be the only practicable method by which adequate vehicle headlighting can be provided without causing disturbing levels of glare to motorists coming in the opposite direction. It is technically and economically feasible, and is advantageous in terms of improved visibility. The result will improve vehicular control, safety, and comfort, and probably traffic flow and use of highways at night.

Recommendations

Although it appears that polarization of headlights provides the most practicable approach to achieving adequate visibility at night, many aspects of the conversion and use of polarized headlamps require further consideration and evaluation. A public test and evaluation program should be undertaken to examine problems and develop precise data on the costs and benefits of such a system. The data would evaluate public response to the system through interviews and operational studies of traffic flow and accident characteristics. Also, problems of equipment operation and maintainability would be studied. To obtain reliable measurements, the test should be conducted in isolation from traffic equipped with conventional headlights, for a sufficient period of time. A test is currently being planned on an island, with access from the mainland limited to car ferry, ocean-going shipping, and air transport (*21*).

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Travel by Motor Vehicles in 1970

BY THE OFFICE OF
HIGHWAY PLANNING

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MOTOR vehicle travel in the United States in 1970 totaled 1,121 billion vehicle miles. This is equivalent to an average daily traffic (ADT) of 820 vehicles on each mile of the 3.7 million miles of roads and streets in the Nation. To accumulate this total travel in 1970, 89 million passenger cars, 3 million motorcycles, 379 thousand buses, and 19 million trucks traveled an average of 10,076 miles and consumed 92 billion gallons of gasoline and diesel fuel at a rate of 830 gallons per vehicle. Total travel for 1971, based on information for the first 6 months of the year, is estimated at 1,170 billion vehicle-miles, a 4.4 percent increase over 1970.

National travel by motor-vehicle type and related data have been reported in this journal for many years. For 1969 these data appeared in PUBLIC ROADS, vol. 36, No. 6, February 1971.

Definitions

The term vehicle-miles and other technical terms used in this article are defined in the following statements:

Vehicle-miles.—The term vehicle-miles refers to the amount of travel by one motor vehicle traveling 1 mile and includes travel on all highways and streets in the United States.

Trailer combinations.—A trailer combination is a truck or truck tractor pulling one or more trailers and/or a semitrailer.

Vehicles registered.—Vehicles registered refers to the total number of vehicles registered in a State in a calendar year or in a registration year if the registration year does not differ from the calendar year by more than 1 month.

Motor fuel consumption.—Motor fuel consumption is the total consumption of motor fuel by highway vehicles for the year. The total amounts are obtained from State records and adjusted to remove fuel consumed for farm and other nonhighway uses.

Motor fuel consumption rate.—Motor fuel consumption rate is the average rate of motor fuel usage in miles per gallon (m.p.g.).

Annual miles per vehicle.—Annual miles per vehicle is an average figure obtained by dividing the total travel in annual vehicle-miles by the total number of vehicles registered.

Gallons per vehicle.—Gallons per vehicle is a figure obtained by dividing the fuel consumed by the vehicles registered.

Interstate System traveled-way.—The traveled-way of the Interstate System consists of completed sections plus those roads and streets now carrying traffic that will be served by the Interstate System when it is completed.

Travel

The travel and related data for 1970 are shown in table 1 by road system and vehicle type. Travel estimates by State and highway system, prepared by the State highway departments, are shown in table 2.

Ten States reported 1970 travel in excess of 30 billion annual vehicle-miles, accounting for almost 53 percent of all the travel in the Nation. California with more than 10 percent of the total led the way with 117.0 billion vehicle-miles. Following California, in order, were New York, 68.6 billion; Texas, 68.0 billion; Pennsylvania, 56.7 billion; Ohio, 56.0 billion; Illinois, 55.3 billion; Michigan, 53.1 billion; Florida, 41.8 billion; New Jersey, 39.9 billion; and Indiana, 32.6 billion.

Twenty States, including the 10 just listed, reported travel exceeding 20 billion annual vehicle-miles. These States accounted for approximately three-fourths of the Nation's travel.

Main rural roads, comprising 17 percent of the Nation's total of 3.7 million miles of roads and streets, carried 36.8 percent of the 1970 travel. Urban streets accounted for 51.5 percent of the total travel, although they represented only 15 percent of the total mileage. Local rural roads accounted for 11.7 percent of the travel on approximately 68 percent of the mileage.

The Interstate System traveled-way accounted for about 1 percent of the total mileage of roads and streets and carried 18.7 percent of the travel.

The Federal-aid primary system, including Interstate, represented about 7 percent of the mileage and carried 48.5 percent of the travel. All Federal-aid systems combined, which includes 24 percent of the mileage, carried 66 percent of the travel.

Passenger cars represented 80 percent of the vehicles and accounted for over 79 percent of the travel; motorcycles, 2.5 percent of all vehicles and about 1 percent of all travel; and trucks and truck combinations, 17 percent of all vehicles and 19 percent of all travel. Similar figures for buses were less than one-half of 1 percent.

In the area of vehicle performance, annual miles per vehicle rose from 9,969 in 1969 to 10,076 in 1970, a sharp rise when compared to the trend. Gallons of fuel consumed per vehicle continued to rise—from 821 in 1969 to 830 in 1970. Miles traveled per gallon of fuel consumed, which began dropping in 1967 after several years of relative stability, dropped only slightly—from 12.15 in 1969 to 12.14 in 1970.

Table. 1.—Estimated motor-vehicle travel in the United States and related data—1970 [1]

[From table VM-1, Highway Statistics, 1970]

Vehicle type	Mortor-vehicle travel: Main rural roads	Mortor-vehicle travel: Local rural roads	Mortor-vehicle travel: All rural roads	Mortor-vehicle travel: Urban streets	Mortor-vehicle travel: Total	Number of vehicles registered	Average travel per vehicle	Motor-fuel consumption: Total	Motor-fuel consumption: Average per vehicle	Average travel per gallon of fuel consumed
	Million vehicle-miles	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Million vehicle-miles*	*Thousand*	*Miles*	*Million gallons*	*Gallons*	*Miles gallon*
Personal passenger vehicles:										
Passenger cars [2]	-----	-----	-----	-----	890, 844	89, 280	9, 978	65, 649	735	13. 57
Motorcycles [2]	-----	-----	-----	-----	10, 148	2, 815	3, 605	135	48	75. 00
All personal passenger vehicles	307, 047	99, 402	406, 449	494, 543	900, 992	92, 095	9, 783	65, 784	714	13. 70
Buses:										
Commercial	939	194	1, 133	1, 810	2, 943	90. 3	32, 591	644	7, 132	4. 57
School	784	902	1, 686	414	2, 100	288. 7	7, 274	300	1, 039	7. 00
All buses	1, 723	1, 096	2, 819	2, 224	5, 043	379. 0	13, 306	944	2, 491	5. 34
All passenger vehicles	308, 770	100, 498	409, 268	496, 767	906, 035	92, 474	9, 798	66, 728	722	13. 58
Cargo vehicles:										
Single-unit trucks	76, 949	28, 671	105, 620	68, 823	174, 443	17, 788	9, 807	17, 237	969	10. 12
Trailer combinations	26, 874	1, 570	28, 444	11, 783	40, 227	960	41, 903	8, 363	8, 711	4. 81
All trucks	103, 823	30, 241	134, 064	80, 606	214, 670	18, 748	11, 450	25, 600	1, 365	8. 39
All motor vehicles	412, 593	130, 739	543, 332	577, 373	1, 120, 705	111, 222	10, 076	92, 328	830	12. 14

[1] For the 50 States and District of Columbia.

[2] Separate estimates of passenger car and motorcycle travel are not available by highway category.

[Table VM-2 from Highway Statistics, 1970]

(Millions)

Division	State	Federal-aid highway system: Interstate rural: Final 01	Interstate rural: Traveled-way 1/ 31	Interstate rural: Total rural	Interstate urban: Final 02	Interstate urban: Traveled-way 1/ 32	Interstate urban: Total urban	Sub-total Inter-state	Other primary: Rural 03	Other primary: Urban 04	Other primary: Total	Secondary: State rural 05	Secondary: State urban 06	Secondary: Local rural 07	Secondary: Local urban 08	Secondary: Total	Total Federal-aid rural	Total Federal-aid urban	Total Federal-aid	Not on Federal-aid systems: Other State rural 09	Other State urban and munici-pal 10	Local rural 11	Local urban and munici-pal 12	Sub-total rural	Sub-total urban and munici-pal	Total
New England	Connecticut	665	250	915	2,236	461	2,697	3,612	1,202	1,832	3,034	960	869	6	47	1,882	3,083	5,445	8,528	277	1,986	283	5,372	3,643	12,803	16,446
	Maine	498	181	679	72	77	149	828	1,499	487	1,986	942	152	-	-	1,094	3,120	788	3,908	997	339	404	460	4,521	1,587	6,108
	Massachusetts	1,332	134	1,466	1,800	1,194	2,994	4,460	3,075	3,244	6,319	645	626	1,257	1,667	4,195	6,443	8,531	14,974	156	1,488	568	8,886	7,167	18,905	26,072
	New Hampshire	531	46	577	138	58	196	773	1,240	336	1,576	836	157	4	2	999	2,657	691	3,348	171	220	139	558	2,967	1,469	4,436
	Rhode Island	134	18	152	718	62	780	932	214	1,231	1,445	217	435	16	203	871	599	2,649	3,248	69	107	88	1,090	756	3,846	4,602
	Vermont	366	92	458	44	39	83	541	840	183	1,023	436	11	147	10	604	1,881	287	2,168	68	3	211	237	2,160	527	2,687
	Total	3,526	721	4,247	5,008	1,891	6,899	11,146	8,070	7,313	15,383	4,036	2,250	1,430	1,929	9,645	17,783	18,391	36,174	1,738	4,143	1,693	16,603	21,214	39,137	60,351
Middle Atlantic	New Jersey	603	251	854	1,835	1,520	3,355	4,209	3,207	6,032	9,239	40	108	1,759	2,956	4,863	5,860	12,451	18,311	1,819	2,437	4,508	12,817	12,187	27,705	39,892
	New York	3,073	148	3,221	7,781	771	8,552	11,773	8,171	15,217	23,388	1,711	1,039	2,966	1,404	7,120	16,069	26,212	42,281	21	31	7,617	18,642	23,707	44,885	68,592
	Pennsylvania	5,473	307	5,780	2,483	714	3,197	8,977	8,560	6,453	15,013	5,856	3,776	49	107	9,788	20,245	13,533	33,778	3,415	4,590	4,054	10,840	27,714	28,963	56,677
	Total	9,149	706	9,855	12,099	3,005	15,104	24,959	19,938	27,702	47,640	7,607	4,923	4,774	4,467	21,771	42,174	52,196	94,370	5,255	7,058	16,179	42,299	63,608	101,553	165,161
South Atlantic (North)	Delaware	81	-	81	388	65	453	534	1,043	679	1,722	318	197	-	-	515	1,442	1,329	2,771	-	-	92	95	1,534	1,424	2,958
	Dist. of Col.	-	-	-	231	169	400	400	-	1,066	1,066	-	-	-	506	506	-	1,972	1,972	-	-	-	849	-	2,821	2,821
	Maryland	1,082	40	1,122	2,539	611	3,150	4,272	2,880	2,844	5,724	1,623	1,368	481	463	3,935	6,106	7,825	13,931	655	93	3,255	2,558	10,016	10,476	20,492
	Virginia	3,476	674	4,150	1,028	416	1,444	5,594	5,425	3,359	8,784	3,528	868	2,242	575	7,213	15,345	6,246	21,591	123	560	2,157	3,988	17,625	10,794	28,419
	West Virginia	611	430	1,041	115	192	307	1,348	1,708	754	2,462	1,387	218	1,085	39	2,729	5,221	1,318	6,539	8	30	376	1,448	5,605	2,796	8,401
	Total	5,250	1,144	6,394	4,301	1,453	5,754	12,148	11,056	8,702	19,758	6,856	2,651	3,808	1,583	14,898	28,114	18,690	46,804	786	683	5,880	8,938	34,780	28,311	63,091
South Atlantic (South)	Florida	2,409	1,287	3,696	1,879	1,081	2,960	6,656	5,352	4,238	9,590	4,994	3,387	383	102	8,866	14,425	10,687	25,112	1,092	1,800	2,918	10,859	18,435	23,346	41,781
	Georgia	2,678	1,004	3,682	2,157	193	2,350	6,032	6,558	2,406	8,964	2,870	581	1,415	521	5,387	14,525	5,858	20,383	148	345	1,893	6,761	16,566	12,964	29,530
	North Carolina	1,835	1,088	2,923	583	436	1,019	3,942	5,022	2,369	7,391	10,213	2,740	4	212	13,169	18,162	6,340	24,502	1,884	623	33	2,582	20,079	9,545	29,624
	South Carolina	1,195	541	1,736	605	146	751	2,487	5,005	2,120	7,125	3,062	1,120	173	9	4,364	9,976	4,000	13,976	383	1,280	455	465	10,814	5,745	16,559
	Total	8,117	3,920	12,037	5,224	1,856	7,080	19,117	21,937	11,133	33,070	21,139	7,828	1,975	844	31,786	57,088	26,885	83,973	3,507	4,048	5,299	20,667	65,894	51,600	117,494
East North Central	Illinois	3,026	1,206	4,232	4,994	580	5,574	9,806	8,043	9,141	17,184	1,018	807	1,982	817	4,624	15,275	16,339	31,614	1,393	3,563	2,862	15,881	19,530	35,783	55,313
	Indiana	2,673	1,162	3,835	1,800	319	2,119	5,954	8,902	2,606	11,508	3,753	683	1,592	215	6,243	18,082	5,623	23,705	196	173	1,483	7,021	19,761	12,817	32,578
	Michigan	3,598	298	3,896	4,188	1,754	5,942	9,838	7,329	7,797	15,126	1,860	590	7,228	1,589	11,267	20,313	15,918	36,231	21	69	2,854	13,936	23,188	29,923	53,111
	Ohio	5,526	55	5,581	6,095	976	7,071	12,652	8,395	6,522	14,917	4,393	1,906	2,307	2,392	10,998	20,676	17,891	38,567	109	412	4,754	12,172	25,539	30,475	56,014
	Wisconsin	1,916	141	2,057	847	234	1,081	3,138	5,938	2,863	8,801	2,502	692	1,753	1,079	6,026	12,250	5,715	17,965	135	73	1,780	4,610	14,165	10,398	24,563
	Total	16,739	2,862	19,601	17,924	3,863	21,787	41,388	38,607	28,929	67,536	13,526	4,678	14,862	6,092	39,158	86,596	61,486	148,082	1,854	4,290	13,733	53,620	102,183	119,396	221,579
West North Central	Iowa	2,104	205	2,309	470	41	511	2,820	6,254	1,795	8,049	-	-	2,310	325	2,635	10,873	2,631	13,504	112	74	1,130	3,407	12,115	6,112	18,227
	Kansas	1,145	151	1,296	542	60	602	1,898	3,864	1,262	5,126	578	27	1,428	469	2,502	7,166	2,360	9,526	73	69	1,139	2,559	8,378	4,988	13,366
	Minnesota	764	426	1,190	1,676	503	2,179	3,369	4,850	2,945	7,795	1,095	88	3,849	188	5,220	10,984	5,400	16,384	36	93	1,650	4,217	12,670	9,710	22,380
	Missouri	1,676	748	2,424	2,418	182	2,600	5,024	5,958	2,406	8,364	2,643	726	20	46	3,435	11,045	5,778	16,823	181	665	1,848	6,268	13,074	12,711	25,785
	Nebraska	978	99	1,077	233	9	242	1,319	3,122	781	3,903	675	33	669	101	1,478	5,543	1,157	6,700	4	-	914	1,982	6,461	3,139	9,600
	North Dakota	472	79	551	16	4	20	571	1,190	220	1,410	220	8	465	16	709	2,426	264	2,690	1	2	687	470	3,114	736	3,850
	South Dakota	622	172	794	25	14	39	833	1,656	246	1,902	248	38	448	34	768	3,146	357	3,503	22	5	589	570	3,757	932	4,689
	Total	7,761	1,880	9,641	5,380	813	6,193	15,834	26,894	9,655	36,549	5,459	920	9,189	1,179	16,747	51,183	17,947	69,130	429	908	7,957	19,473	59,569	38,328	97,897
East South Central	Alabama	1,095	961	2,056	348	940	1,288	3,344	4,726	2,100	6,826	1,533	308	961	177	2,979	9,276	3,873	13,149	48	19	905	3,450	10,229	7,342	17,571
	Kentucky	2,180	270	2,450	855	219	1,074	3,524	4,617	1,718	6,335	3,850	561	235	63	4,709	11,152	3,416	14,568	1,275	770	1,140	2,207	13,567	6,393	19,960
	Mississippi	1,038	235	1,273	299	378	677	1,950	4,065	778	4,843	924	126	1,215	203	2,468	7,477	1,784	9,261	13	20	645	1,688	8,135	3,492	11,627
	Tennessee	2,164	836	3,000	1,349	551	1,900	4,900	4,925	2,687	7,612	1,015	242	684	103	2,044	9,624	4,932	14,556	53	7	1,870	4,233	11,547	9,172	20,719
	Total	6,477	2,302	8,779	2,851	2,088	4,939	13,718	18,333	7,283	25,616	7,322	1,237	3,095	546	12,200	37,529	14,005	51,534	1,389	816	4,560	11,578	43,478	26,399	69,877
West South Central	Arkansas	1,133	119	1,252	360	11	371	1,623	2,943	1,166	4,109	2,376	400	284	76	3,136	6,855	2,013	8,868	53	83	813	1,468	7,721	3,564	11,285
	Louisiana	849	976	1,825	762	297	1,059	2,884	2,823	1,662	4,485	3,668	672	13	1	4,354	8,329	3,394	11,723	802	477	1,153	2,683	10,284	6,554	16,838
	Oklahoma	1,513	76	1,589	1,136	-	1,136	2,725	5,028	1,737	6,765	1,202	430	885	453	2,970	8,704	3,756	12,460	330	114	1,000	4,167	10,034	8,037	18,071
	Texas	4,477	1,618	6,095	7,445	1,386	8,831	14,926	12,096	8,232	20,328	8,640	3,005	-	-	11,645	26,831	20,068	46,899	1,796	1,374	2,741	15,221	31,368	36,663	68,031
	Total	7,972	2,789	10,761	9,703	1,694	11,397	22,158	22,890	12,797	35,687	15,886	4,507	1,182	530	22,105	50,719	29,231	79,950	2,981	2,048	5,707	23,539	59,407	54,818	114,225
Mountain	Arizona	1,549	1,164	2,713	538	166	704	3,417	1,482	394	1,876	670	68	665	2,305	3,708	5,530	3,471	9,001	295	13	732	2,083	6,557	5,567	12,124
	Colorado	1,351	584	1,935	766	121	887	2,822	2,075	1,738	3,813	1,554	210	-	-	1,764	5,564	2,835	8,399	22	44	1,479	3,254	7,065	6,133	13,198
	Idaho	657	262	919	62	15	77	996	1,363	305	1,668	404	26	342	146	918	3,028	554	3,582	4	-	554	601	3,586	1,155	4,741
	Montana	384	521	905	37	16	53	958	1,560	282	1,842	477	67	-	-	544	2,942	402	3,344	-	-	923	600	3,865	1,002	4,867
	Nevada	588	146	734	49	67	116	850	671	319	990	316	64	82	218	680	1,803	717	2,520	25	37	155	723	1,983	1,477	3,460
	New Mexico	1,274	407	1,681	255	166	421	2,102	1,591	748	2,339	960	116	6	-	1,082	4,238	1,285	5,523	106	514	470	850	4,814	2,649	7,463
	Utah	643	644	1,287	648	48	696	1,983	922	595	1,517	535	437	138	192	1,302	2,882	1,920	4,802	30	72	446	758	3,358	2,750	6,108
	Wyoming	694	217	911	22	23	45	956	929	151	1,080	261	21	17	9	308	2,118	226	2,344	2	3	355	341	2,475	570	3,045
	Total	7,140	3,945	11,085	2,377	622	2,999	14,084	10,593	4,532	15,125	5,177	1,009	1,250	2,870	10,306	28,105	11,410	39,515	484	683	5,114	9,210	33,703	21,303	55,006
Pacific	California	5,226	4,173	9,399	12,912	4,394	17,306	26,705	11,740	16,302	28,042	2,561	931	4,531	6,121	14,144	28,231	40,660	68,891	414	983	12,664	34,040	41,309	75,683	116,992
	Oregon	1,656	81	1,737	924	32	956	2,693	2,442	1,606	4,048	1,097	414	863	556	2,930	6,139	3,532	9,671	39	29	1,591	2,195	7,769	5,756	13,525
	Washington	714	850	1,564	2,528	337	2,865	4,429	2,549	2,014	4,563	1,185	774	1,363	947	4,269	6,661	6,600	13,261	26	105	1,533	5,786	8,220	12,491	20,711
	Total	7,596	5,104	12,700	16,364	4,763	21,127	33,827	16,731	19,922	36,653	4,843	2,119	6,757	7,624	21,343	41,031	50,792	91,823	479	1,117	15,788	42,021	57,298	93,930	151,228
Total - All Divisions		79,727	25,373	105,100	81,231	22,048	103,279	208,379	195,049	137,968	333,017	91,851	32,122	48,322	27,664	199,959	440,322	301,033	741,355	18,902	25,794	81,910	247,948	541,134	574,775	1,115,909
	Alaska	-	-	-	-	-	-	-	446	138	584	166	52	-	-	218	612	190	802	-	-	198	387	810	577	1,387
	Hawaii	67	129	196	292	209	501	697	702	477	1,179	172	48	50	38	308	1,120	1,064	2,184	9	-	259	957	1,388	2,021	3,409
United States Total		79,794	25,502	105,296	81,523	22,257	103,780	209,076	196,197	138,583	334,780	92,189	32,222	48,372	27,702	200,485	442,054	302,287	744,341	18,911	25,794	82,367	249,292	543,332	577,373	1,120,705

1/ Traveled-way includes travel on highways in the Interstate System status of completion group 2. This group consists of mileage which is adequate for present traffic but will need further construction and improvement to bring it to full Interstate standards. In some instances these are divided highways with full control of access and in others they are two-, three- or four-lane undivided highways with no access control.

Digest of Recent Research and Development Results

Reported by the Implementation Division, Office of Development

The items reported here have been condensed from highway research and development reports, predominantly of Federally aided studies. Not necessarily endorsed or approved by the Federal Highway Administration, the items have been selected both for their relevancy to highway problems and for their potential for early effective application.

Each item is followed by source or reference information. Reports with an "NTIS" reference number are available in microfiche (microfilm) at 95 cents each or in paper facsimile at $3 each from the National Technical Information Service (NTIS), 5285 Port Royal Road, Springfield, Va. 22151.

HYDRAULIC FLOW RESISTANCE FACTORS

Design engineers concerned with hydraulic resistance of corrugated metal conduits commonly used as drainage structures will find useful information in a recently published report. It discusses the parameters influencing flow resistance of corrugated conduits, including relative roughness, Reynolds number, corrugation form, and method of manufacture. While the report does not cover the entire spectrum of culvert and storm drain design, it presents resistance factors and aids in tabular and graphic form for dealing with the geometry of various shapes and sizes of corrugated conduits. Of significance too is the evident magnitude of the errors inherent in applying a single resistance factor to corrugated pipes of all sizes, shapes, flow rates, and corrugation forms. The resistance factors are presented in terms of both Darcy *f* and Manning *n* values, permitting incorporation into most design procedures.

Hydraulic Flow Resistance Factors for Corrugated Metal Conduits, Federal Highway Administration, U.S. Department of Transportation. Available from Superintendent of Documents, Government Printing Office, Washington, D.C. 20402, for 55 cents.

EFFECTS OF PROPOSED HIGHWAY IMPROVEMENTS ON PROPERTY VALUES

Apparently, highway agency authority to acquire excess land would benefit both the State and the land owner in connection with right-of-way acquisition for proposed highway improvements. Moreover, advance acquisition would reduce the causes of enhancement or diminution in the right-of-way, and to a limited degree in areas contiguous to it. These views are among a number set forth in a recent NCHRP study of the effects of proposed highway improvements on property values. The author further states that revolutionary thinking and new concepts should be considered in right-of-way alinement and acquisition. He says that apparently statutory authority is generally (but not always) needed to eliminate the concept of *use* before highway agencies can proceed effectively under a revised concept of *public purpose.*

One of the objectives of the study was to develop and objectively set forth alternative valuation and legal methods, and to state the pros and cons of each. Appraisers, legal practitioners, right-of-way engineers and agents, and other public works officials will find this document of practical use. It presents in condensed form many ramifications of and suggested solutions related to a vexatious problem.

Effects of Proposed Highway Improvements on Property Values, Available from the Highway Research Board, National Academy of Sciences, 2101 Constitution Ave., NW., Washington, D.C. 20418. Price $2.60.

HIGHWAY SOILS MAPPING BY AERIAL COLOR PHOTOGRAPHY

Aerial color photography is the best single type of remote sensing technique for extracting highway-oriented information on soils and terrain conditions, according to a recent evaluation. Color mosaics of surface areas of low relief (300 feet or less), properly annotated, make excellent master engineering soil plans that are more complete than black-and-white with respect to environmental conditions. In areas of high relief differences, some perspective distortions occur, but for most engineering site selection and design studies, the total view of the terrain and the soil environment shown on annotated color aerial photographs is more important than the geometric accuracy lost. The summary report discusses a number of remote sensing techniques and is rated an excellent reference source on types of soil information obtainable from the various sensing techniques, such as infrared, radar, photography, etc.

Remote Sensing and Development of Annotated Aerial Photographs as Master Soil Plans for Proposed Highways, Summary report, Indiana State Highway Commission, NTIS No. PB 199422.

NEW SPEED AND ACCURACY IN HIGHWAY DESIGN

Simplicity and greater speed and accuracy in highway engineering location and design are the benefits of a newly integrated single computer program for use in aerial triangulation. This new program combines and refines three earlier individual processes and offers such improvements as (1) minimizing of chance of errors from manual handling, (2) elimination of intermediate steps into and out of the computer, (3) reduced computer usage and easier detection of errors, and (4) convertibility to an integrated set of programs suitable for use on smaller systems. In addition, broader programs for roadway design are now nearing completion, into which the triangulation program can be integrated.

Electronic Computer Program for Analytical Strip Triangulation and Adjustment (Program No. R 0300), U.S. Department of Transportation, Federal Highway Administration, Office of Development, Implementation Division, Washington, D.C. 20590.

FREEWAY TRAFFIC MERGING CONTROL

Recent advancements in the development of freeway and ramp control systems have significantly contributed to freeway traffic control technology, and offer a rational approach to establishing control policy consistent with the demand-capacity philosophy. This approach can mean more efficient use of freeway sections operating near capacity. An investigation by the Texas Transportation Institute (TTI) documents the conduct and findings of research performed mainly on the Gulf Freeway in Houston. The work also covers development of various hardware and software aspects of freeway control systems using centralized digital computers, including system design requirements. Results can be directly applied to design and implementation of ramp and freeway control systems.

Gap Acceptance and Traffic Interaction in the Freeway Merging Process—Phase II, By the Texas Transportation Institute, NTIS No. PB 193901.

Highway Bridge Field Tests in the United States, 1948–70

BY THE OFFICE OF RESEARCH

Reported by CONRAD P. HEINS, JR., Associate Professor, University of Maryland, and CHARLES F. GALAMBOS, Structural Research Engineer, Federal Highway Administration

THE following tabulation of highway bridges provides a reference to bridge types and parameters on which loading performance data are available for the use of highway bridge design and research engineers. The tabulation was compiled from a survey conducted by Subcommittee No. 4, Static Field Tests, of Committee A2C05, Dynamics and Field Testing of Bridges, of the Highway Research Board.

The bridge descriptions are presented in alphabetical order, by State. The format is designed to permit a quick examination of the bridge characteristics, such as girder size and spacing, type and thickness of deck, length of spans, and type and orientation of supports. The kind of test loading to which the bridge was subjected is also indicated. The numbers in the last column pertain to reports listed at the end of the tabulation. The best source of any published reports would be the various highway departments in which the bridges are located.

This report includes the data collected by Varney and Galambos, published in Highway Research Record No. 76, as well as an unpublished tabulation of bridge tests by W. W. Sanders. Their contribution, and that of those who replied to the survey, is gratefully acknowledged.

Definitions

AC —Asphaltic concrete
C_L —Centerline
con. —Continuous
CP —Cover plates
ext. —Exterior
int. —Interior
LL —Live load
o.c. —On center
PC —Prestressed concrete
p.c.f.—Pounds per cubic foot
PMS—Plant mix surface
RC —Reinforced concrete
rdwy—Roadway
rpt. —Report

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
Alabama									
Auburn University, Auburn.	2	2′9″ deep, 4′7½″ at haunch.	8′	6¼″ RC 16′ wide	3-span con., 44′, 55′, 44′.	0° skew, 2 rollers, 2 plates.	Monolithic RC.	Static, dynamic, crawl.	39.
California									
Sacramento Road at Bryte Bend.	3	9′2″ deep, box is trapezoidal, 26′ wide at bottom.	17′	10″ RC	One simple span, 145′.	0° skew, pinned.	Composite steel box girder and slab.	Dynamic, crawl.	No rpt.
Route 680/580 separation (old 107/5), Dublin.	2	4.5′ deep, 15′ floor beam spacing.	23′	⅜″ and 7/16″ steel deck plate, ¼″ and 2″ epoxy and PMS; closed trapezoidal ribs, 5.5″ overhangs.	4-span con., 75′, 85′, 85′, 75′.	0° skew, pinned.	Orthotropic steel deck plate.	Static (concentrated multiple axle and superposition loads).	9, 10, 13, 14.
Harrison Street, Oakland.	5	4′10″ deep	7′3″	6⅜″ RC	One simple span, 80′.	0° skew, pinned.	RC box girder bridge with and without diaphragms.	Dynamic crawl, 57 kip Euclid.	15,16.
Webber Creek near Placerville.	4	8′9″ deep	9′4″	7¼″ RC	Four simple spans.	0° skew, pinned.	Composite	Static and dynamic, crawl.	7.
San Leandro Creek Bridge.	3	36WF230	11′	8″ RC 33′ wide	23-span con., 63′ each 3d span hinged.	0° skew	Noncomposite.	Dynamic	48, 49.
Vacaville		Deck plate only 34′ wide.		7/16″ deck plate, T-stiffeners at 18″ o.c. with plate 15″ by ¼″ butt flange 10″ by ½″ skew, transverse bar stiffeners 5″ by 5/16″ at 1′ intervals.	Five spans, 26′ each.	30° skew, pinned.	Orthotropic, steel deck.	Static, dynamic.	8.
Webber Creek Bridge.	4	Top flange 14″ by 1¼″, web 94″ by ⅜″, bottom flange 18″ by 1¾″.	9′4″	7¼″ RC	Two simple spans, 135′, 135.′	0° skew, pinned.	Composite	Crawl	41.

Connecticut									
South Road over I-84, Farmington.	3	7′ to 12′ deep at pier.	19.25′	9½″ min. RC haunched 11¼″ at girders, 48.5′ wide, 5.5′ sidewalk, 40′ rdwy.	2-span con., 175′, 175′.	Radial supports spherical bridges at pier, rockers at abutments.	Welded noncomposite curved girder, radius=1,040′.	Dead load, static LL.	74.
District of Columbia									
Bridge No. 516 ramp from 11th St., S.W. onto O St., S.W.		7′ by 4′9″		7″ to 8″ lightweight RC	2-span con., 53′, 72′, C_L S to N.	19° roller, 8° fixed, 47° roller, S to N.	Curved RC box beams.	Static 15-ton truck.	43.
Bridge No. 1205, Theodore Roosevelt Bridge approach structure.	4	Web 35″ by ⅝″, flange 18″ by ¾″ to 2⅝″.	7′2″	7″ RC 2½″ bituminous surface, 26′ wide.	3-span con., 41′, 100′, 95′ (inside girder) W to E.	25° roller, 54° roller, 46° fixed, 12° roller, W to E.	Steel curved plate girder.	Static	21.
Bridge No. 1206 Theodore Roosevelt Bridge approach structure.	4	Web 35″ by ⅝″, flange 18″ by ¾″ to 2⅝″.	8′1″	7″ RC, 2½″ bituminous surface, 27′ wide.	2-span con., 81′, 59′ (inside girder) E to W.	24° roller, 7° fixed, 22° roller, E to W.	Steel curved plate girder.	Static 15-ton truck.	21.
Florida									
U.S. 19 Suwanee River.	6	AASHO type IV	5′2½″	7″ RC with 2″ AC surfacing, 28′ wide.	2-span con., 120′, 120′.	0° skew sliding, raker-type bearings.	120 p.c.f. lightweight concrete deck and girders.	HS20 dynamic and static.	67, 73.
Idaho									
U.S. 95 over Skookumchuck Creek near White Bird.	5	Folded plate girders 5′ deep.	7′11¾″	6½″ RC 39′8″ wide	One simple span, 70′.	15° skew, elastomeric bearing pads.	Cast-in-place deck monolithic with precast girders; resultant girders are trapezoidal, spaced box girders.	Static	No rpt.

453-025—72——3

Test site and date	Girders: Number	Girders: Size	Girders: Spacing	Deck or slab	Spans	Supports	Remarks	Test loadings	Reference
				Bridge description					
				Illinois					
AASHO Road Test Bridges (18 bridges tested).		All bridges 3 beams only.		All bridges 15′ wide	All bridges simple spans 50′9″.	0° skew simple.		All bridges static, dynamic.	1.
		Steel wide flange beam at 5′ spacing:		Reinforced concrete:					
1A		18WF55 with CP.		6.45″			Noncomposite.		
1B		18WF50		6.55″			do		
2A		18WF55		6.45″			do		
2B		18WF50 with CP.		6.30″			Composite		
3A		21WF62		6.45″			Noncomposite.		
3B		18WF60 with CP.		6. 45″			Composite		
4A, 4B		18WF60 with CP.		6.45″			Noncomposite.		
9A, 9B		18WF96 with CP top and bottom.		6.75″			do		
		Beams at 4′8″ spacing:		Prestressed concrete:					
7A, 7B 8A, 8B		All beams 20″ deep by 11.5″ wide.		6.50″, 6.55″, 6.60″			Monolithic		
		I-section beams:		Prestressed concrete:					
5A, 5B		Post-tensioned		6.60″, 6.70″			Composite		
6A, 6B		Pretensioned		6.55″, 6.60″			do		
I–57, Jefferson County.	5	48″ prestressed I-beams.	6′6″	7″ RC 30′ wide	4-span con., for LL, 46′, 72′, 72′, 46′.	13° skew, RC piers on piles.	Composite	Long-term deformations.	27, 60.
Salt Fork River Bridge, U.S. 150, St. Joseph.	5	36WF170, CP top and bottom 10″ by 1⅛″ by 21′.	7.5′	7″ RC slab 32′4″ (includes an 8″ high by 40″ wide curb on both sides).	3-span con., 75′1″, 96′6″, 75′1″.	0° skew, pinned.	Noncomposite	130 test truck runs (dynamic and crawl) 12 random traffic (dynamic) truck.	75, 76, 77.

C.B.&O. Railroad Bridge, I–80, Rock Island County.	5	36WF150, CP top and bottom 10½″ by ½″ by 14″, CP at center of center span 10″ by ⅜″ by 35′.	8′	7″ RC 36′ wide (9″ high, 3′-wide curb on both sides).	3-span con., 50′6″, 76′3″, 50′6″.	0° skew, pinned.	Noncomposite.	1,497 random traffic (dynamic) truck runs.	76, 77.
Shaffer Creek Bridge, I–74 and I–280, Rock Island County.	9	24WF100	Five at 5.5′ and 4 at 5.33′, with 3.0′ and a separation joint between the groups of 4 and 5 beams.	7″ RC 43′8″ wide (9″-high, 2′10″-wide curb on south side, 9″-high, 1′10″-wide curb on north side).	2-span con., 43′, 43′.	0° skew, pinned.	Noncomposite.	1st test series 77 test truck runs (dynamic and crawl); 302 random traffic (dynamic) (truck) 2d test series: 905 random truck traffic.	76, 77.
Indiana									
Bean Blossom Creek SR No. 37, 3 miles north of Bloomington, Monroe County.		125′ Pony-Truss, 8 panels at 15.6′.	3′9″ stringer spacing.	6¼″ prestressed precast slab	Single span	0° skew, pinned.	Noncomposite experimental deck with prestressed plank.	Special State truck, dynamic and static measurements.	33.
Yellow River Bridge, SR 39, Starke County.	6	33WF130	4 int. at 5′5″, 2 ext. at 5′10″.	7″ RC 30′ wide	3-span con., 62′6″, 75′, 62′6″.	Right angle skew, RC piers and abutments.	Noncomposite vibration study summer 1953.	H–15 truck, H–20 truck, and special State test truck, dynamic and static.	34.
Iowa									
Ashworth Road 1958.	2	30WF108 ext	9′	7¼″ RC, 28′ wide	4-span con. 53′, 67′, 53′.	0° skew	Composite	Crawl, dynamic, static.	50, 51.
	2	30WF130 int	9′						
Clive Road 1958	4	36″ int 34″ ext.	9′6″	8″ slab, 33.5′ wide	4-span con., 41′, 69′, 69′, 41′.	0° skew	Composite aluminum plate girder.	Crawl, dynamic, static.	50, 51.

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
Iowa—Continued									
Des Moines River Bridge 1958.	6	60''	6'	6¼'' RC 30' wide	One simple span, 100'.	0° skew	Prestressed post-tensioned.	Crawl, dynamic, static.	50, 51.
Holcomb Test, Bridge No. 1.	2	24WF76, CP 6'' by ⅜'' by 20'.	9'8¼''	8.63'' RC, 30' wide	Simple span 41.25'		Composite	Static	38.
	2	27WF94, CP 8½'' by 1'' by 29'.	9'8¼''	do	do		do	do	Do.
Holcomb Test, Bridge No. 2.	2	33WF130, CP 10'' by ⅞'' by 44'6''.	9'8¼''	30' by 8.07''	Simple span 71.25'		Composite	Static	38.
	2	36WF194, CP 11'' by 1⅜'' by 47'.	do	do	do		do	do	Do.
Miller's Creek 1956.	2	27WF94 ext.	8'11''	8'' RC 28' wide	3-span con., 55', 70', 55'.		Composite	Crawl, dynamic, static.	58.
	2	30WF116 int.							
Skunk River 1957.	2	33WF141 and 33WF152 ext.	9'4''	8½'' RC 48' wide	3-span con., 73', 94', 73'.	0° skew	Composite	Crawl, dynamic, static.	40.
	4	36WF194 int.							
Wapsipinicon Road 1956.	2	33WF125 ext.	8'5'', 6'3'', 8'5''.	8'' RC 24' wide	5-span con., 73', 94', 94', 94', 73'.	0° skew	Noncomposite.	Crawl, dynamic, static.	58.
	2	36WF160 int.							
Maryland									
I–83N over Bunker Hill Road.	8	33WF161 ext.	6'7''	7'' RC 42' wide	Three simple spans, 27', 47', 22'.	0° skew	Noncomposite test span is 47' span only.	Dynamic, random (24 hours).	36.
		33WF141 int.							
I–83S over Bunker Hill Road.	8	24WF76 with 12'' by 11/16'' by 33' CP.	5'11''	7'' RC 39' wide	Three simple spans, 27', 47', 22'.	0° skew	Composite spiral shear connectors, test span is 47' span only.	Dynamic, random (24 hours).	36.

I–83N over Shawan Road.	9	60″ deep, 44″ by 6″ top flange; 20″ by 6″ botom flange.	4′	6″ to 2″ bituminous wearing surface on top of girder.	One simple span, 100′.	0° skew simple.	Post-tensioned concrete.	Crawl, dynamic.	59.
I–83S over Shawan Road.	7	51″ by 3/8″ web, 18″ by 2 1/8″ bottom flnage, 14″ by 1 1/2″ top flange.	7.3′	7″ RC deck, 42′ wide	One simple span, 100′.	0° skew	Composite, spiral shear connectors.	Crawl, dynamic.	59.
I–495 underpass at U.S. Route 1.	7	27WF84, 7″ by 1 1/2″ by 24′ CP.	7′2″	7″ RC, 39′ wide	Five simple spans, 35′, 83′, 71′, 41′, 38′.	7° skew	Test span is 38′ span only, spiral shear connectors.	Dynamic, random (24 hours).	36, 55.
Maryland Route 4 over Patuxent River.	5	36WF230 with 14″ by 1″ by 52′ CP.	8′1 1/2″	7″ RC, 31′ wide	One simple span, 80′.	0° skew	Composite, stud shear connectors.	Static, dynamic.	55, 63.
Maryland Route 4 over Patuxent River.		200′ through truss, floor beam 36WF230 stringers, 24I70.	25′ floor beams 4.5′ stringers.	9″ RC, 27′ wide	One simple span, 200′.	0° skew, pinned.	Noncomposite.	Static	69.
Maryland Route 32 (Sykesville bypass) over River Road.		Five aluminum triangular modules 5′8″ by 7′.		6″ to 9″ lightweight RC, 30′ wide.	Three simple spans, 105′, 94′, 93′.	0° skew 4.8% grade.	Composite, semi-monocoque.	Crawl, dynamic.	59.
Maryland Route 5 and Maryland Route 301.	8	36WF194 with 10″ by 7/8″ by 43′ CP.	5′3″	7″ RC, 30′ wide	Two simple spans, 76′, 76′.	33° skew	Spiral shear connectors, one span tested only.	Dynamic, random truck traffic (24 hours).	36.
Maryland Route 301 over Western Branch (N).	7	27WF94 int. 27WF102 ext.	5′	7″ RC, 28′ wide	3-span con., 42′, 52′, 42′.	6° skew	Noncomposite.	Dynamic, random truck traffic (1 week).	64.
Maryland Route 301 over Western Branch (S).	7	27WF97 with 6″ by 5/8″ by 25′ CP.	7′	7″ RC, 40′ wide	Three simple spans, 42′, 52′, 42′.	6° skew	Spiral shear connectors, test span N 42′ span.	Dynamic, random truck traffic (1 week).	64.
U.S. Route 1 over Patuxent River at Laurel.	7	30WF116 int. 36WF160 ext., CP over int. supports 1/2″ by 12″ by 21′.	5′4″	7″ RC, 32′ wide	Three simple spans, 65′, 85′, 65′.	20° skew	Composite, spiral shear connectors.	Static	35.

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
Massachusetts									
Conway–Shirkshire 1953.	4	33WF141	4′10″	6½″ RC, 14′ wide	One simple span, 69′.	0° skew	Noncomposite.	Crawl, dynamic, static.	6.
Gilbert–Ware 1953.		102″		7″slab, 24′ wide	One simple span, 114′.	0° skew	Noncomposite.	Crawl, dynamic, static.	6.
Townsend–South 1953.	2	84″		7″ slab, 24′ wide	One simple span, 86′.	17° skew	Noncomposite through plate girder span.	Crawl, dynamic, static.	6.
Ware–Malboeuf 1953.	2	54″ 16WF45 floor beam.		Open grating 2½″, 18′ wide	One simple span, 77′.	0° skew	Through plate girder span.	Crawl, dynamic, static.	6.
Michigan									
B1–11–18–7—1958.		36WF182	4′6″	7″ slab	2-span con., 128′, 68′.	0° skew	Noncomposite.	Crawl, dynamic traffic record.	No rpt.
B1–18–12–2—1956.		36WF160 36WF194	5′ 5′	7″ slab	2-span con., 67′, 69′.	0° skew	Composite	Dynamic, static.	No rpt.
B1–18–12–2—1957.		36WF160 36WF194	5′ 5′	7″ slab	2-span con., 67′, 69′.	0° skew	Composite	Dynamic, static.	56.
B1–34–6–1—1957.		48″ plate girder 36WF160	6′ 6′	7″ slab	2-span con., 74′, 47′.	0° skew	Noncomposite.	Dynamic, static.	56.
B1–34–13–8—1956.		18WF50 stringer on plate girder.		7″ RC	Con. span, 101′.	0° skew	Noncomposite.	Crawl, dynamic, static.	No rpt.
B1–38–11–25		33″ T-beam		8″ slab	3-span con., 42′, 58′, 42′.	0° skew		Crawl, dynamic static.	56.

B1-39-5-8—1957.		36WF150 36WF160	4′10″ 4′10″	7″ slab	2-span con., 57′, 61′.	0° skew	Noncomposite.	Dynamic, static.	56.
B1-56-12-6—1957.	2	36WF150 plate girders 66″ deep.	5′6″	7″ slab	2-span con., 74′, 56′.	0° skew	Composite	Dynamic, static.	56.
B1-62-12-1—1957.	2	Plate girders 81″ deep.		7″ slab, 24′ wide	Con. span, 97′.	0° skew	Noncomposite.	Dynamic, static.	56.
B1-64-10-11—1956.		67″ plate girder		7″ slab	2-span con., 84′, 60′.	0° skew	Noncomposite.	Crawl, dynamic static.	No rpt.
B1-70-7-3—1957.		66″		7½″ RC	Con. span, 69′ end span.	Skewed	Noncomposite.	Crawl, dynamic static.	56.
B1-73-20-3—1957.		36WF182 33WF141 36WF170	5′ 5′	7″ slab	3-span con., 74′, 58′, 75′.	0° skew	Composite	Dynamic, static.	56.
B1-77-20-11—1957.		36WF194 78″ plate girder.	5′4″	7″ slab	100′, 60′, 60′	18° skew	Noncomposite.	Crawl, dynamic, static.	No rpt.
B1-81-1-13—1956.		36WF182 53″ plate girders		7½″ slab 7″ slab	5-span con., 98′, 98′, 66′ 60′, 60′.	0° skew	Composite (60′ spans) noncomposite (98′, 98′, 66′ spans).	Crawl, dynamic, static.	No rpt.
B2-38-1-14—1957.		36WF170	5′2″	7¼″ slab	3-span con., 42′, 80′, 42′.	0° skew	Noncomposite.	Dynamic, static.	56.
B2-39-3-8—1957.		30WF108 30WF124	4′10″	7½″ RC	Two simple spans 45′, 49′.	Skewed	Noncomposite.	Crawl, dynamic, static.	56.
B2-61-3-21—1956.		Two through trusses, 16WF46 floor beams.			Two simple spans, 108′, 110′.	0° skew	Through truss.	Crawl, dynamic, static.	No rpt.
B2-73-20-2—1957.		36WF182 36WF150	5′ 5′	7″ slab	2-span con., 74′, 58′.	0° skew	Composite	Dynamic, static.	56.
B3-38-1-14—1957.		36WF160 do	4′ 7′6″	7″ slab	2-span con., 50′, 65′.	0° skew	Noncomposite.	Dynamic, static.	56.

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
Michigan—Continued									
B5-81-11-8—1957.		28″ T-beam	6′2¼″	8″ slab	4-span con., 39, 53′, 53′, 39′.	0° skew		Crawl, dynamic, static.	56.
B8-58-7-26—1956.	3	36WF150	6′	7″ RC	Three simple spans, 70′, 60′, 72′.	28° skew	Composite	Crawl, dynamic, static.	No rpt.
Fennville	7	36WF182 steel web diaphragms 28″ by ⅜″ with 2L's 3″ by 3″ by ⅜″ top and bottom.	5′2¼″	7⅛″ RC, 33′ wide	Six simple spans, 58′5″, 59′3″, 59′3″, 59′3″, 59′3″, 58′5″.	0° skew	Five spans noncomposite, one span composite.	Static, dynamic.	24, 25.
Jackson Bypass Bridge.	12	Plate girders 50″ deep with CP.	4′2¼″	Two 29′ rdwys	Eight spans:				24.
					72′6″	0° skew	Continuous	Dynamic, creep.	
					92′	do	do		
					74′	do	do		
					84′3″	do	Simple		
					84′3″	1.5° skew	do		
					76′3″	do	do		
					81′9″	do	do		
					76′	do	do		
X3-16-7-26—1956.		36WF170	6′	7″ slab	3-span con., 65′, 53′, 67′.	30° skew	Composite	Crawl, dynamic, static.	No rpt.
	2	36WF230							
X3-33-6-1—1957.	3	36WF170	5′4″	7½″ RC	One simple span 61′.	0° skew	Composite	Crawl, dynamic, static.	56.
Minnesota									
I-35W, Bloomington.	8	30WF108 with 18WF42.7 diaphragms.		6″ RC deck, 38′ wide	3-span con., 38′,61′,38′.	0° skew, pinned.	Composite	Random traffic and controlled static and dynamic.	12.
No. 6440—1952.	8	36WF150		6½″ RC, 27′ wide	6-span con., 85′,100′, 100′,100′, 100′,85′	0° skew	Composite	Dynamic, static.	78.

Nebraska									
Loup River Bridge.	5	36WF, 14″ by ⅜″ CP piers 1 and 6; 18″ by ⅜″ CP piers 2 and 5; spans 1 and 7—36WF150; spans 2 and 6—36WF194; spans 3 and 5—36WF230; span 4—36WF230.	5′11″ centers	7½″ RC deck, 21′ clear rdwy	7-span cantilever type; length: 72′, 105′,105′, 115′, 105′, 105′,72′, includes cantilevered and suspended spans.	No skew simple supports, pinned hangers.	Noncomposite H–12.5 design built in 1934.	Static, dynamic, H–15 H–15–S12.	18.
New York									
Routes 5 and 13, Chittenango.	6	21WF112, CP 20″ by 1⅛″ top, 24″ by 1⅛″ bottom, 10″ by 30″ precompressed concrete in tension flange.	8′8″	7½″ RC deck, 55′4″ wide	One simple span 79′.	29° 11′ skew elastomeric bearings.	Monolithic deck, haunches and web protection *preflex* girders.	Dead load only.	No rpt.
Hulls Falls Road, Keene.	4	60″ deep, size varies.	8′8″	7″ RC deck 28′10″ wide with 4″ wearing surface.	One curved simple span, length = 123′, radius at C_L = 477′.	Radial	Composite deck.	Dead load static LL.	5.
I–540 Ramp CBW, Rensselaer.	4	60″ deep, size varies.	8′	7½″ RC deck, 30′6″ wide	One curved simple span, length at C_L = 95′, radius at C_L = 162′.	Radial	Composite deck.	Dead load static and dynamic LL.	22.
I–540 Ramp C43, Rensselaer.	5	60″ deep, size varies.	7′9″	7½″ RC deck 37′6″	2-span con., radius = 265.5′ length = 2 at 100′.	Radial	Composite deck.	Dead load static and dynamic LL.	23.
I–490, Rochester	10	3′1″ by 10″ RC	6′	7″ RC deck, 59′6″ wide	4-span con., 35′, 60′, 60′, 35′.	14° 35′ skew	Monolithic high strength reinforcing steel.	Dynamic, crawl.	2, 4.

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
New York—Continued									
I–490, Rochester	7	4′11″ by 10″ RC	7′11″	7″ RC deck, 55′2″ wide	Simple span 65′10″.	14° 20′ skew	Monolithic high strength reinforcing steel.	Dynamic, crawl.	2, 4.
Ohio									
Ohio Highway Department Test Bridge.	5	WF beams	6′11″	7.75″ RC, 32′ wide	Con. span	0° skew	Noncomposite.	Static	37.
Oklahoma									
I–40, Canadian County.	4	24WF68, spans 1 and 5.	7′8″	6.5″ RC, 24′ rdwy, two 1′6″ curbs with steel rail.	Two simple spans and 3-span con., 32′, 59′, 59′, 59,′ 32′.	0° skew	Noncomposite.	Dynamic, crawl, design load H15.	No rpt.
	4	24WF84, spans 2 and 4.	7′8″						
U.S. 64, Arkansas River, Pawnee–Osage Counties.	4	84″ by ½″	9′	7″ RC, 27′ rdwy, 1′6″ curb left, 4 median right.	Simple span, 160′.	0° skew	Composite, plate girders.	Dynamic, crawl design load H20.	No rpt.
U.S. 64, over State Highway 97, Tulsa County.	6	48″ by 5/16″	8′	7. 5″ RC, 43′ rdwy, two 1′ curbs and rails.	2-span con., 84′3″ and 79′3″.	0° skew	Composite, plate girders.	Dynamic, crawl, design H20–S16.	No. rpt.
I–244 Mingo Creek, Tulsa County.	9	42″ by 5/16″	9′	7. 5″ RC, 73′ rdwy	Three spans, 64′, 80′, 64′.	77° skew	Composite, plate girders.	Dynamic, crawl H20–S16 PPM 20–4.	No rpt.
Oregon									
North Dillard Bridge, Pacific Highway, Bridge No. 2555, Douglas County.	2	5.9′ to 9.2′ deep, 20′ floor beam spacing.	26.5′	6½″ concrete deck, 30′ rdwy 3.5′ walk each side.	3-span con., 122′, 160′, 122′.	0° skew, pinned.	Composite for positive LL moment.	H20–S16–44 dynamic.	20.

Battle Creek Bridge, Pacific Highway E., Bridge No. 2245A, Marion County.	2	2.3′ deep, 7.5′ floor spacing (open rib system), floor beam at bottom plate midspan (closed rib system).	14′ (each half rdwy).	7/16″ deck plate, half closed rib, half open rib 44′ total rdwy.	3-span con., 22.5′, 30′, 22.5′.	0° skew, fixed.	Orthotropic steel deck.	HS20–44, dynamic.	No rpt.
Marquam Pacific Highway Bridge, Bridge No. 8328.	6	Steel arches	8′	7½″ deck, 57′ rdwy, 2.3′ walk each side.	Three spans, 301′6″, 440′, 301′6″.	30° skew, pinned.	Steel arches instrumented with elec. strain gages.	Static HS20–S16.	No rpt.
	2	do	5′4″						
	2	do	6′						
Oneatha Gorge, Old Col. River Highway 1952.	4	16½″ by 51″ plate girders.	7′½″	6½″ concrete deck, 26′ rdwy, 3.5′ walk each side.	One simple span, 48′.	0° skew, rockers.	Steel instrumented with elec. strain gages.	Static H20–5–16.	57.
Vista Ridge Tunnel, Sunset Highway, Portland.		Ring girders (tunnel).	Varies				Ring girders instrumented with elec. strip gages.	Static	No rpt.
				Pennsylvania					
Bartonsville	5	AASHO type III PC I-beams.		7½″ RC deck, slab 37′6″ wide, 32′ wide rdwy.	Simple span, 68′6″.	0° skew		Crawl, speed runs with FHWA test vehicle.	No rpt.
Berwick	4	4′ by 39″ PC box beams.	8′9⅜″	7½″ RC deck, slab 33′6″ wide, rdwy 28′ wide.	Simple span, 65′3″.	0° skew		Crawl, speed runs with FHWA test vehicle.	31, 71.
Brookville	4	4′ by 36″ PC box beams.	8′10″	7½″ RC deck, slab 33′6″ wide, rdwy 28′ wide.	Simple span, 64′10½″.	45° skew		Crawl, speed runs with FHWA test vehicle.	66.
Centerport Creek 1954.	9	21″ by 36″, 12½″ hollow cores.		25′ wide	One simple span, 33′.	0° skew	Pretensioned, prefabricated.	Crawl, dynamic, static.	61.

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
				Pennsylvania—Continued					
Drehersville	5	4′ by 33″ PC box beams.	7′2″	7½″ RC deck, slab 35′6″ wide, rdwy 30′ wide.	One simple span, 61′6″.	0° skew		Crawl, speed runs with FHWA test vehicle.	17, 30, 71.
Fort Loudon 1948.		14′ Warren through truss.		7″ RC, 23′ wide	Simple span, 111′.	0° skew	Warren through truss.	Crawl, dynamic, braking run.	70.
Hazleton	5	4′ by 42″ PC box beams.	9′6″	7½″ RC deck, slab 45′6″ wide, rdwy 40′ wide.	Simple span 69′7″.	1½° skew		Crawl, speed runs with FHWA test vehicle.	No rpt.
Lehighton	6	24″ by 45″ PC I-beams.	6′9″	7½″ RC deck, slab 38′ wide, rdwy 36′ wide.	Simple span, 71′6″.	0° skew	Curb parapet on one side only, other side completely free; tested with and without midspan diaphragms.	Crawl, speed runs with FHWA test vehicle.	No rpt.
Philadelphia	5	4′ by 42″ PC box beams.	9′6″	7½″ RC deck, slab 45′6″ wide, rdwy 40′ wide.	Simple span, 71′9″.	3° skew	Bridge tested with and without midspan diaphragms.	Crawl, speed runs with FHWA test vehicle.	47, 71.
Smithfield Street Bridge, Pittsburgh.		Trusses 25′ o.c., floor beams 9′2½″ o.c.		7/16″ thick aluminum plate longitudinally stiffened with closed ribs, 7″ deep and 16⅛″ o.c., 21½′ wide rdwy (does not participate with floor beams or trusses).	Trusses 360′ (2 spans), floor beams 25′ (simple span), orthotropic deck 9′2½″ (3-span con.).	0° skew, pinned.	Tests involved aluminum orthotropic deck only.	Static and moving load from 20-ton tractor-trailer truck.	62, 68.

White Haven	4	3′ by 42″ PC box beams.	9′	7½″ RC deck, slab 33′6″ wide, rdwy 28′ wide.	Simple span, 64′8″.	8° skew		Crawl, speed runs with FHWA test vehicle.	32, 71.
Rhode Island									
Seekonk River, Providence.	3	Welded girders 72′ by ½″ web, flange varies.	11.5′	9″ RC slab with 3″ asphalt wearing, 23′3″ rdwy.	Two con. spans, 123′1″, 119′9″.	16.4° skew on one abutment, others 0° pinned, fixed, pinned.	Composite C_L radius =497′4″ diaphragms at 23″.	Dead load, static and dynamic LL.	No rpt.
Seekonk River, Providence.	4	34″ by ½″ web, flange varies.	7′8″	9″ RC slab with 3″ asphalt wearing, 23′3″ rdwy.	Simple span=80′.	0° skew, pinned.	Composite C_L radius =248.5′, diaphragms at 19′8″.	Dead load, static and dynamic LL.	No rpt.
South Dakota									
James River 1955.	4	30WF108		6″ RC, 24′ wide	Simple span, 55′.	0° skew	Composite	Crawl, dynamic, static.	19.
James River 1955.	4	30WF108		6″ slab, 24′ wide	3-span con., 70′, 90′, 70′.	0° skew	Composite	Dynamic, static.	19.
Tennessee									
State Route 111 over I–40, Cookeville.	4	Left lane 48″ by ⅜″ web plate.	9′	7½″ RC deck 40′ wide, two 3′ overhang.	3-span con., 110′6″, 110′6″, 110′6″.	82°55′37″ tangent to curve, pinned.	Composite	Regular traffic, dynamic.	No rpt.
	5	Right lane 48″ by ⅜″ web plate.	9′	40′ wide, two 3′ overhang.					
I–40 over U.S. 70, Crab Orchard.	2	Left lane 54″ by ⅜″ web plate.	13′	7″ RC deck, 30′ wide, two 5′3″ overhang.	5-span con., 86′6″, 107′6″, 79′6″, 95′8⅞″, 77′⅞″.	30°33′ skew, pinned.	Composite	Interstate traffic, dynamic.	No rpt.
	2	Right lane 54″ by ⅜″ web plate.	13′ to varies	30′ varies, two 5′3″ overhang	86′6″, 107′6″ 79′6″, 96′1⅝″, 78′11″.				

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
Tennessee—Continued									
I-40 over State Route 61, Harriman.	3	Left lane 46″ by ⅜″ web plate.	9′	7½″ RC deck, left lane 31′3″ wide, 4′ and 4′9″ overhang.	7 span con., 55′, 72′, 11′, 115′, 110′, 72′, 60′.	44°40′ skew, pinned.	Composite	Construction traffic.	No rpt.
	4	Right lane 46″ by ⅜″ web plate, flange varies from 12″ by ⅝″ to 14″ by 2″.	9′3″	Right lane 30′ wide, 4′6″ and 5′3″ overhang.					
I-40 over Campbell Station Road, Knox County.	4	1′6″ by 4′5″ both lanes.	7′6″	7″ slab, 40′ rdwy, two 4′2″ overhang both lanes.	3-span con., 41′, 60′, 41′ both lanes.	60° skew, selflubricating plates.	Cast-in-place concrete.	Interstate traffic.	No rpt.
I-40 over Everett Road, Knox County.	4	Left lane 3′10″ by 1′6″.	5′2″	7″ slab, 30′ rdwy, two 4′2″ overhang.	3-span con., 47′, 66′ 47′.	75° skew selflubricating plates.	Cast-in-place concrete.	Interstate traffic.	No rpt.
	3	Right lane 1′6″ by 3′.	7′4″	7″ slab, 30′ rdwy, 4′3″ overhang.	58′, 72′, 58′.				
Broadway relocated over Tazewell Pike, Knoxville.	9	36WF135	9′6″	7″ RC deck, two 3′3″ overhang, two 36′ rdwy.	3-span con., 63′2½″, 93′, 39′2½″.	46° skew, pinned.	Composite	Traffic	No rpt.
I-640 over Broadway (present), Knoxville.	5	Left lane 28½″ by ⅜″ web.	Varies	8½″ RC deck, two 3′7½″ overhang.	3-span con., 31′9″, 57′6″, 36′9″.	57°40′40″ skew.	Composite	Construction	No rpt.
	3	Right lane 28½″ by ⅜″ web.	12′3″	42′ rdwy, 3′7″ RR overhang, two 3′7½″ overhang.	29′9″, 57′6″, 35′.				
I-640 over Broadway (relocated) Knoxville.	5	Left lane 42″ by ⅜″ web.	7′8⅜″	7″ RC deck, 42′ rdwy, overhang varies.	2-span con., 93′6″, 85′.	63°8′ skew, pinned.	Composite A36 steel.	Construction traffic.	No rpt.
	6	Right lane 42″ by ⅜″ web.	8′1″	52′ rdwy, overhang varies					

I–40 over Tennessee 95, Loudon County.	4	33WF130, 36WF170 middle span both lanes.	7′	7″ slab, 30′ rdwy, two 4′3″ overhang both lanes.	Simple spans, 42′6″, 71′6″, 42′6″ both lanes.	70° skew, selflubricating plates.	Composite	Interstate traffic.	No rpt.
Ramp O over Bellevue Blvd., Memphis.	5	36WF230 span 2, 36WF135 and 36WF118 spans 1 and 3.	Varies	7″ RC deck, 31′2″ wide	Three simple spans.	62°36′55″, pinned.	Composite, A36 steel.	Construction traffic.	No rpt.
Ramp X over Bellevue Blvd., Memphis.	2	36WF300 to 36WF230.	5′6″	7″ RC deck, 22′2″ wide, overhang 2′6″, 3′.	Three simple spans.	52°59′40″, pinned.	Composite, A36 steel.	Construction traffic.	No rpt.
	3		3′8″						
Ellington Parkway over Douglas Ave., Nashville.	5	Northbound 33WF118.	8′	7″ RC deck, 44′ rdwy, two 3′2″ overhang.	2-span con., 63′6″, 69′6″.	89°27′7″ skew to Douglas Ave., pinned.	Composite, A36 steel.	Traffic	No rpt.
	4	Southbound 33WF118, 33WF130, 33WF118.	8′6″	38′ rdwy, two 3′2″ overhang.					
I–65 over L.&N. Railroad, Nashville.	5	Left lane 36WF182	7′5″	7″ RC deck 41′2″ wide, 4′8½″ overhang.	3-span con., 82′, 88′, 84′.	28°36′ skew, pinned.	Composite, A36, steel.	Interstate traffic, dynamic.	No rpt.
	5	Right lane 36WF182, 36WF135.	7′5″	Overhang varies from 4′8″ to 4′.					
Mansford Road over Elk River.	4	27WF94 with CP in 45′ span.	7′4″	7″ RC deck 24′ wide	3-span con., 45′, 60′, 45′.	90° skew, pinned.	Noncomposite H–15 design.	Rolling loads, vibration loads, static test to failure.	11
	4	27WF102 with CP in 60′ span.	7′4″						
Tennessee Highway 130 over Boiling Fork Creek.	4	AASHO Type III	9′	7″ RC deck, 28′ wide	Four simple spans, each 66′.	68° skew, bearing pads.	Composite HS–20 design.	Rolling loads, vibration loads, static test to failure.	11
Tennessee Highway 130 over Elk River.	4	36WF160 with CP in 70′ spans.	8′4″	7″ RC deck, 28′ wide	4-span con., 70′, 90′, 90′, 70′.	90° skew, rocker.	HS–20 design composite in positive moment region.	Rolling loads, vibration loads, static test to failure.	11.
	4	36WF170 with CP in 90′ spans.	8′4″						

Test site and date	Bridge description								Reference
	Girders			Deck or slab	Spans	Supports	Remarks	Test loadings	
	Number	Size	Spacing						
Tennessee—Continued									
U.S. Highway 41A over Elk River.	4	18″ by 49″ RC T-beams.	5′	8″ RC deck, 24′ wide	Six simple spans, one at 28′ and five at 53′.	60° skew, bearing pads.	Monolithic (T-beam) H-15 design.	Static test to yielding.	11.
Texas									
I-10 over T. & N.O. Railroad, El Paso.		33WF130, 33WF141	8′ centers	6½″ RC 52′ rdwy	5-span con., 50′, 65′, 65′, 65′, 50′.	43°27′ skew	Noncomposite.	Dynamic	29.
U.S. 80 over P.S. & F. Railroad, El Paso.		27WF and 30WF	7.28′ centers	6½″ RC, two 22′ rdwy with 5′ median.	3-span con., 40′, 51′, 40′ and five simple spans 39′, 30′, 37′, 40′, 34′.	Various skew	Noncomposite.	Dynamic	28.
I-35 near Hillsboro.	3	2.375′ wide	8.875′ centers	6½″ RC, 24′ rdwy	4-span con., haunched RC girders, 55′, 88′, 88′, 55′.	30°22′ skew	2′ wide sidewalks.	Dynamic	54.
U.S. 75 over SL-SF Railroad near Sherman.		34″ and 40″ PC beams.	7′3″ centers	3″ prestressed, precast deck panels and 3″ cast-in-place RC top.	Five simple spans 40′, 50′, 50′, 50′, 40′.	19°38′ skew	Composite for LL.	Dynamic	28.
Virginia									
Cedar Creek Bridge, I-81.	5	3′9″ PC AASHO type III I-beams, 4,000 p.s.i.	8′	8″ RC, 30′ rdwy plus 2′7″ and 3′7″ curbs and sidewalks.	Six simple spans, 60′1″, 60′, 85′, 60′, 60′, 60′1″.	10°54′7″ skew, 4% grade.	Composite	Dynamic to crawl.	No rpt.

Hazel River Bridge, Route 729, 10.5 miles north of Culpeper.	4	36WF150 in spans 1 and 3, 36WF160 in span 2 with CP 10″ by ⅜″ in spans 1 and 3, CP 10½″ by ⅝″ in span 3.	7′8″----------	7½″ RC deck, 24′ wide rdwy---	Three simple spans, 62′7″ 67′6″, 62′7″.	0° skew, 0.18% grade, pinned.	Composite----	Dynamic to crawl.	45.
New Market Bridge, Route 793 over I–81 south of New Market.	4	36WF135 CP over center portion 11″ by ¾″ plate on ext. girders, 11″ by ⅞″ plate on int. girders.	8′4″----------	8″ RC deck, 24′ rdwy----------	Six simple spans 63.5′, four at 61′, 48′.	0° skew 5.44% grade, pinned.	Composite----	Dynamic to crawl.	52.
Appomattox River Bridge, Route 36 north of Peterburg.	5	Aluminum (6061–T6) modules of triangular shape 6′9″ wide by 4′10″ high.	--------------	7½″ lightweight concrete, 115 p.c.f., 28′ rdwy.	One simple span, 97′.	0° skew, 1% grade, pinned.	Composite----	Dynamic to crawl.	46.
Dumfries Bridge, Route 95 over relocated Quantico Creek and Route 629.	6	Two 36WF230 ext. with CP 15″ by ¾″, four 36WF194 int. with CP 10½″ by 1⅛″.	8′4″----------	8″ RC, 42′ rdwy plus two 2′8″ sidewalks.	Four simple spans, three at 69′4″, one at 76′.	3° 46′ skew, 3% grade.	Composite----	Dynamic to crawl.	No rpt.
Weyer's Cave Bridge, Route 276 north of Weyer's Cave.	4	36WF160 with CP 10½″ by ⅝″.	7′8″----------	7½″ RC deck, 24′ rdwy.	Six simple spans at 67′7″.	0° skew, very flat vertical curve, pinned.	Composite----	Dynamic to crawl.	44.
				West Virginia					
U.S. 50A over Ohio River, St. Marys.		Truss stiffened suspension bridge with floor beams and stringers (not gaged).	--------------	3″ concrete filled steel grid, 27′ wide.	380′, 700′, 380′.	Eyebar chain suspension bridge.	Duplicate of failed Pt. Pleasant Bridge, design LL, 1,400 lb. per lin. ft., 4,200 lb. concentrated load.	FHWA vehicle loaded to 44,000 lb.; also resonant, harmonic vibration.	72.

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