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Public Roads

A JOURNAL OF HIGHWAY RESEARCH

U.S. DEPARTMENT OF TRANSPORTATION
FEDERAL HIGHWAY ADMINISTRATION

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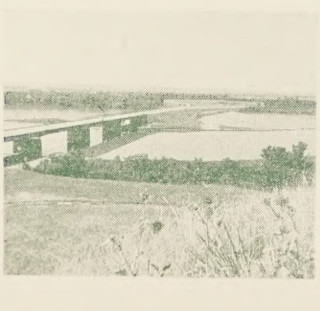

COVER

Grant Marsh Bridge over the Missouri River on Interstate Highway 94 between Bismarck and Mandan in North Dakota. (Photo courtesy of North Dakota State Highway Department.)

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U.S. DEPARTMENT OF TRANSPORTATION
Washington, D.C. 20591

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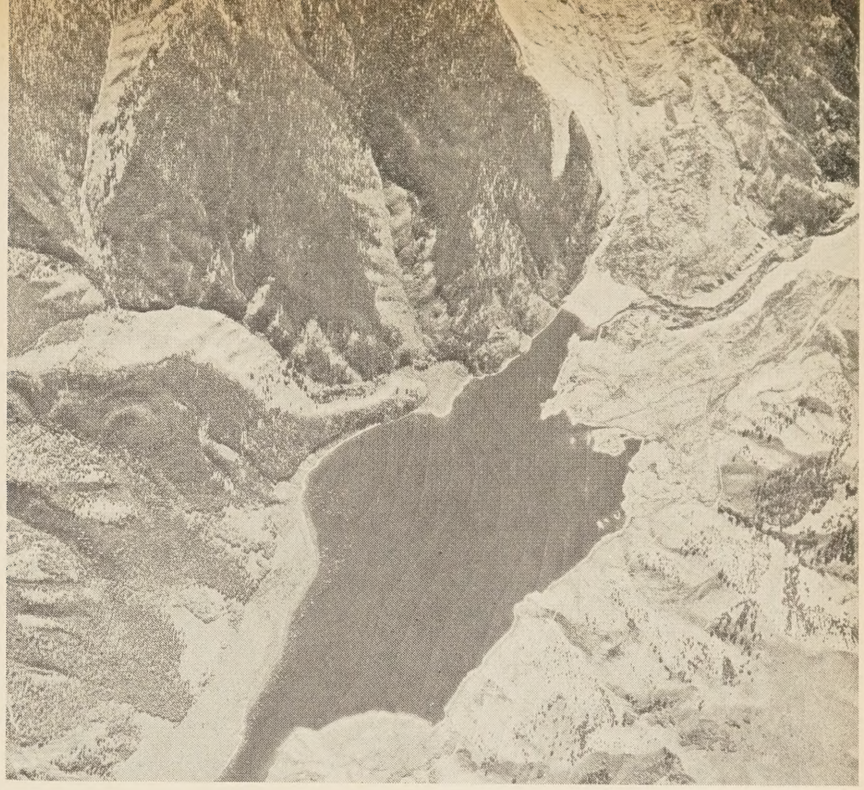

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Ohio Photogrammetric Research Solves Highway Foundation and Right-of-Way Problems

)igested from the third interim report on the research
project "Measure and Depict Trouble Areas in Stereo Models,"
by WAYLAND F. NORELL, Aerial Engineering Section,
OHIO DEPARTMENT OF HIGHWAYS

THE Ohio Department of Highways, under the Federal-aid highway program, has taken progressive steps in researching and developing aerial photographic interpretation techniques to investigate foundation safety and assist right-of-way purchase. Successful implementation of research and development in this field has been highlighted by the following practical results:

- On a proposed highway location, new photogrammetric analysis revealed an unstable slope and potential landslide condition that was previously unknown. A potentially dangerous and costly failure was treated by special design.
- Photogrammetric analysis of another proposed location revealed a mined-out landform. A potential foundation problem was thus spotted and knowledge of the condition will assist right-of-way appraisal if the location is selected.
- In yet another landslide investigation, additional right-of-way was purchased where the effect of highway cuts would affect property outside the normal right-of-way requirement. This application is considered less costly than subsequent renegotiation and damage.
- Photo patterns showed a highway location to be safely separated from an old mine; whereas, a 1904 map incorrectly indicated a serious problem by a one-half mile error in the mine's location.
- States attorneys, in another instance, reported that without mining data produced through photo interpretation applications they would have been handicapped in negotiations with the owners of subsurface mining rights on land beneath a highway.

Research in Progress

Principally, the research emphasizes the detection of photo patterns from stereomodels in photogrammetric plotting instruments that illustrate specific foundation problems in areas where highways may be located (*1*).[1]

Stereomodels (stereopairs), as used in aerial surveying and map making, consist of two aerial photographs of the same area taken from two different camera positions. When properly viewed stereopairs create a mental impression of three dimensions.

Manuals incorporating annotated stereopairs are presently being used by the Ohio Department of Highways to train stereocompilers to recognize visual clues that forewarn of probable foundation problems (*2*).

Originating in 1964, research and development in this field is being conducted in southeastern Ohio, where highways are often unavoidably routed through terrain rated as major for landslide severity. In addition, the area is dotted with coal mines, both active and

[1]Italic numbers in parentheses identify the references listed on page 179.

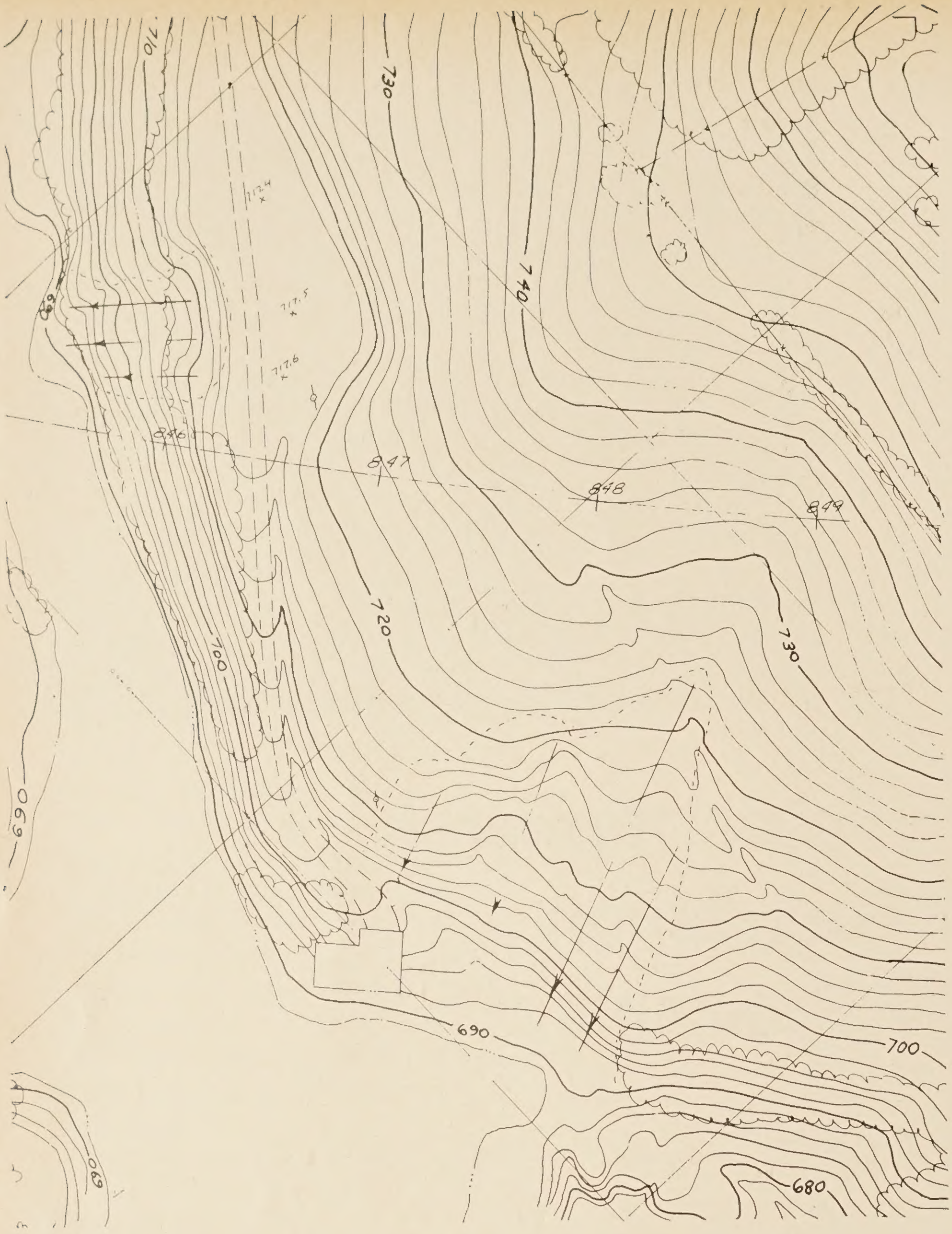

Figure 1.—Photogrammetrically produced highway design map showing delineated and symbolized landslide areas.

mining. A manual documenting subsurfac mining photo patterns has been compiled (3) Potential foundation problem areas resultin from mining, and thus areas where foundatio investigation should be concentrated, ar incorporated in the manual. Also included i a section on coal outcrop mapping for right of-way appraisal and acquisition.

Accurately predicting the position of co seams and the outcrop in relation to th proposed highway centerline is made possibl by combining photo interpretation and photo grammetry. The map which is produce expedites right-of-way appraisal and acquisi tion and permits an equitable settlemen regarding the affected coal seams. Personn of the Aerial Engineering Section have pr pared this type of map for a number of hig way projects (4).

One such request involved a researcher i problems that tested his knowledge of geolog photogrammetry, and photo interpretatio He had to interpret and compile an 8-mi extension to a previously compiled coal ou crop map. His investigation revealed a co seam that, following the regional dip, d scended more than 200 feet through th landforms. Because of this dip, progressivel younger stratigraphic members, includin three coal seams, outcropped and we mapped. The coal outcrop map was then use for right-of-way appraisal and acquisition i the vicinity of an Interstate highway.

Figure 2 illustrates the application of pho patterns. The multiplicity of roof collaps around the perimeter of the landform indicat the lateral extent of the mine workings. A highway cut would reduce the amount of ro over the mine. One solution to a shallow mi is to set the highway grade below the mi floor, thereby removing the problem. T knowledge that the landform is mined out w also assist an appraiser during right-of-w acquisition.

Today the landform is traversed by feasibility alinement under consideration fo State highway relocation. If this landform crossed, the information about its associat problems will be of greater value than, f example, the cost of producing the previous mentioned photo interpretation manual.

abandoned, with many old mines unrecorded on coal maps.

Topographic maps that portray the land contours and patterns of an area in sharp, vivid detail are studied for possible contour anomalies. Figure 1 is a portion of a photogrammetrically compiled highway design map with a symbolized landslide area. Contours shown in the figure reveal an intercepter ditch that diverts up-slope water away from the slide area. This unstable slope was detected solely by photo interpretation.

Although the landslide manual (2) was limited in its scope to unglaciated landforms, the instability in figure 1 occurred in an area where glaciation had encroached. Therefore, the photo patterns developed can also be applied to glaciated terrain.

Obviously, an unstable slope condition, if undetected, would result in embankment failure involving considerable expense to repair. In fact, the money saved by knowing of and being able to avoid or design for terrain problems has been estimated to be considerably greater than the cost of this research.

Another application for landslide detection has been utilized. On one highway relocation, slide-prone slopes were detected. Cuts would provoke sliding beyond normal right-of-way requirements; therefore, additional property was taken to include the remainder of the slides. It should be noted that the land that the relocation traverses is of limited value when contrasted with the cost of renegotiations.

Related Research

A second phase of research involves documenting aerial photo patterns of subsurface

Practical Applications

Interstate and State roads, in terms right-of-way, have been appraised for acq sition by using knowledge gained fr interpreting aerial photographs. Several sea in various proposed highway areas also ha been analyzed by the same method. T following two paragraphs illustrate how inf mation delineated on maps derived from aer photographs has been beneficial in securi safe rights-of-way for highways.

A 2-mile stretch of land being consider for a proposed highway contained coal o crops, drift mine entries, strip mines, and shafts within the right-of-way. These proble conditions, which were detected and sy bolized, showed subsurface mining below t proposed highway. Small voids were found t cuts during construction, but photo interpretation had detected and predicted their location.

A mislocated mine caused a unique problem. The original (1904) map of the abandoned mine showed the mine workings beneath a proposed highway. The map contained a gross error. If the mine were actually located as indicated, voids would have been 10 feet below the proposed grade, and some roof collapse would have occurred during the intervening years. By utilizing photo patterns developed for the mining manual, the mine was actually found to be located one-half mile west of where the 1904 map had it erroneously placed. At the actual site the mine is well below the proposed grade of the road.

Figure 2.—Application of photo patterns showing mine workings.

Occasionally people seek help from the Ohio Department of Highways regarding a specific problem. In one instance an abandoned coal tipple or coal-screening plant site was involved in a right-of-way dispute. By merging the mine map with an air-photo map of the area and using photo interpretation, it was determined that two coal seams were embedded in the landform. The seam of greater value was virtually mined out, and thus the highway appraiser had some valid information for negotiating with the land owner.

Engineer-In-Training Program

Since 1966 engineering graduates entering the Ohio Department of Highways have been given a 1-week Air Photo Interpretation Short Course. This course, part of a more comprehensive program, introduces all engineering functions related to the Department. To date, 140 Engineers-In-Training have taken the course.

The course is specifically designed to teach the fundamentals of photo interpretation. Selected stereopairs are used as models to teach various applications of photo interpretation to specific foundation problems.

Manuals containing aerial photographs of landslide prone areas and subsurface mining are used as the basic teaching materials. These are used in conjunction with other appropriate stereopairs, which have been selected during years of photo evaluation.

The importance and immediate value of this research becomes clear when one realizes that terrain problems that affect highways are not always discernible from the ground, even while trudging the very area where they exist. This can be attributed to vegetation obscuring the terrain. One must know where to look before finding troublesome areas from ground observation. If one has seen aerial photographs of an area and is trained in terrain interpretation, it is then relatively simple to find existing or potential problem areas.

Implementing aerial photo interpreting techniques has more than proven economically feasible. Substantial expense is saved by knowing precisely where to locate highways, thus avoiding costly foundation problems or minimizing them by designing the highway to treat the foundation problem.

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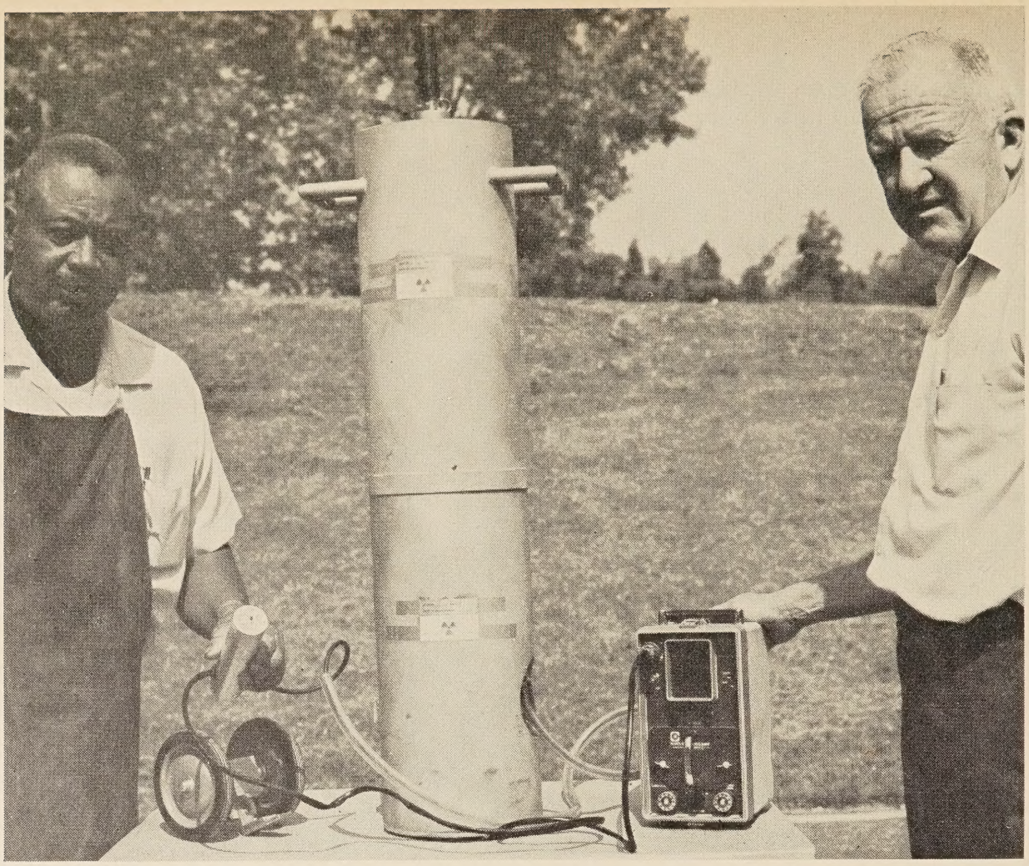
(*3*) *Air Photo Patterns of Subsurface Mining in Ohio*, by Wayland F. Norell, Ohio Department of Highways, Report No. 1606, 1968.

(*4*) *Coal Outcrop and Overburden Mapping with Kelsh Plotter*, by Wayland F. Norell, Highway Research Record 109, 1966, pp. 39–48.



Pretesting Concrete Quality By Gamma-Ray Backscattering

Innovative Nuclear Gage Provides Quick, Accurate Results

A PROTOTYPE portable nuclear gage with the potential for conducting rapid, accurate, and nondestructive field tests to determine the cement content of wet (plastic) portland cement concrete has been developed through research sponsored by the Federal Highway Administration and Atomic Energy Commission (*1*).[1] The research was conducted by the Texas Nuclear Division, Nuclear-Chicago Corp., Austin, Texas.

Although no suitably rapid method for making this analysis is available for all aggregate types, this novel device has proven applicable where the aggregate is essentially siliceous in nature. Accuracy declines rapidly, however, with aggregate mixes containing limestone. Final implementation of the instrument is subject to more extensive laboratory tests and field evaluation.

The portable battery operated gage has three basic parts: probe assembly, electronic digital analyzer, and sample presentation units. Enlcosed in aluminum and hermetically sealed, the probe assembly (fig. 1) contains the radioisotope sources and radiation detector. The electronic digital analyzer (fig. 2) provides power for the probe, pulse amplification, and output display. Either of two sample presentation units, one hand-held and the other small-volume, house the probe assembly, depending on the type of test to be made. The hand-held unit (fig. 3) is inserted in freshly placed concrete on th roadway to conduct in-place tests. Wet con crete samples are packed into the s al volume unit (fig. 4) to sample concrete b for it is paved.

Because cement content of concrete is a important factor in determining struc ural strength and because measurements sh ul be made on-site and within the short tim available between delivery and placin t avoid allowing unsatisfactory concret t harden, this portable, rugged and simp t operate gage should prove valuable to gh way builders and inspectors.

In the application of this method, advan ag is taken of portland cement's essen all

[1] Italic numbers in parentheses identify the references listed on page 183.

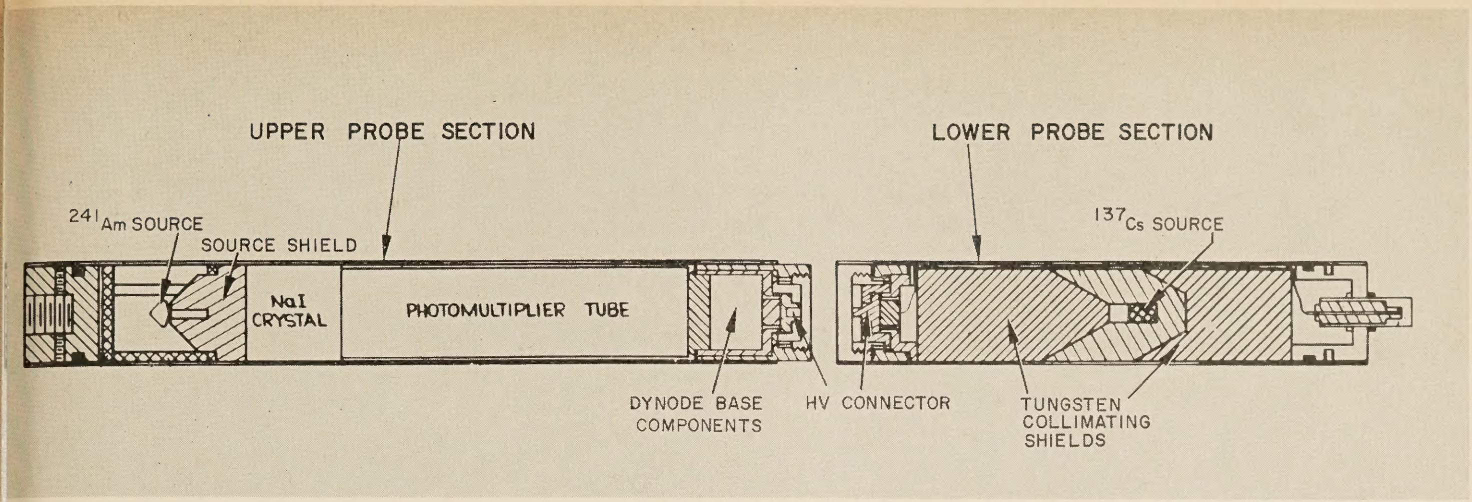


Figure 1.—Probe assembly.

istant composition. Its major element, cium, has a high atomic number and thus a h absorption coefficient for gamma rays npared to other elements usually found in concrete matrix. In a mixture of water, ceous aggregate, and cement, the last plays most prominent role in determining the rall absorption coefficient for low-energy nma radiation; thus, variations in cement tent can be sensitively detected.

The device employs an innovative gamma- backscatter technique in a submersible be.[2] Two radioisotope sources, americium n^{241}) and cesium (Cs^{137}), are used with a ntillation counter device. Americium emits ow-energy radiation and cesium emits a h-energy radiation. The electronic unit ultaneously registers the two backscatter ensities, and cement content is interpreted m a calibration curve.

From an analysis of gamma-ray reflections l the calibration curve, researchers have n able to determine the quantity of cement a plastic concrete mix. Measurements are igned to tell within minutes whether the c contains the proper amount of cement to duce cured concrete of acceptable quality, s preventing a bad batch from being ced in the highway.

Testing Procedure

X-ray fluorescence and low- and high-energy nma radiation scattering techniques were d to study the influence of variables such aggregate type, particle size, water content, k density, and air entrainment resin. Four ic geometrical arrangements of source and ector, three backscatter and one direct nsmission, were the techniques employed to estigate these variables.

Comparative tests on two backscatter angements of probe and sample, surface

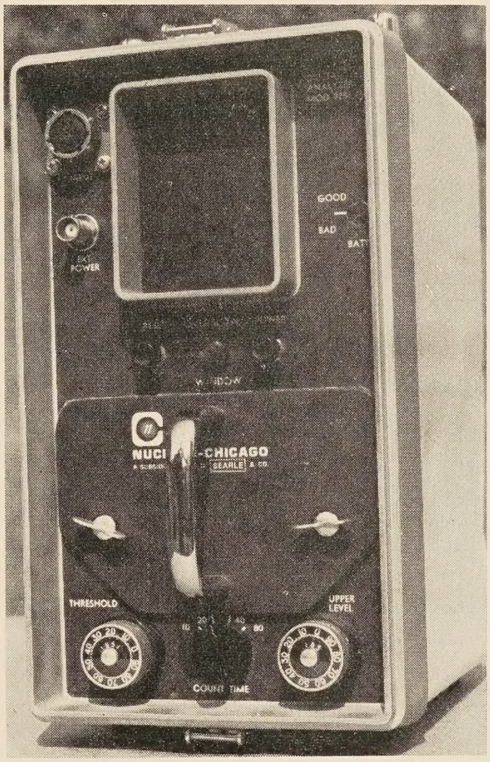


Figure 2.—Electronic digital analyzer.

and subsurface types, showed the latter to be superior. With regard to the effect of variable aggregate particle sizes and for similar source-detector spacings, the subsurface gage was twice as sensitive to variations in cement content.

There was a reduced effect of particle size shown by the subsurface probe. This was attributed mainly to the smaller shadow effect of the large particles because the source was more remotely located from the sample.

By comparison, transmission geometry was sensitive to particle size and sample heterogeneity or different ingredients in the composition.

The range of intensity variation from sample to sample for the transmission gage is considerably greater than for the backscatter gage, and the variation from point to point is even greater (and in some samples exceeds the total change over the entire cement factor range). But if the average radiation intensities of each cement factor group, normalized to the value of the lowest cement factor, are plotted, the sensitivities of the two geometries are very similar. Thus the transmission arrangement appears to offer no advantage in sensitivity and is considerably more affected by sample heterogeneity, in comparison to the subsurface backscatter arrangement.

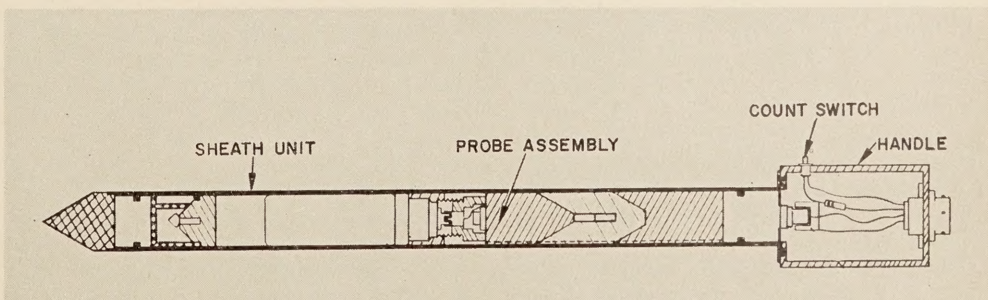


Figure 3.—Hand-held sample-presentation unit, including probe assembly.

Gamma-ray backscatter is simply the reflection of un- rbed gamma rays in a direction approximately opposite of the normal radiation given off by the emitting radio- e substance. Reflections from the concrete particles of bsorbed gamma rays are measured electronically.

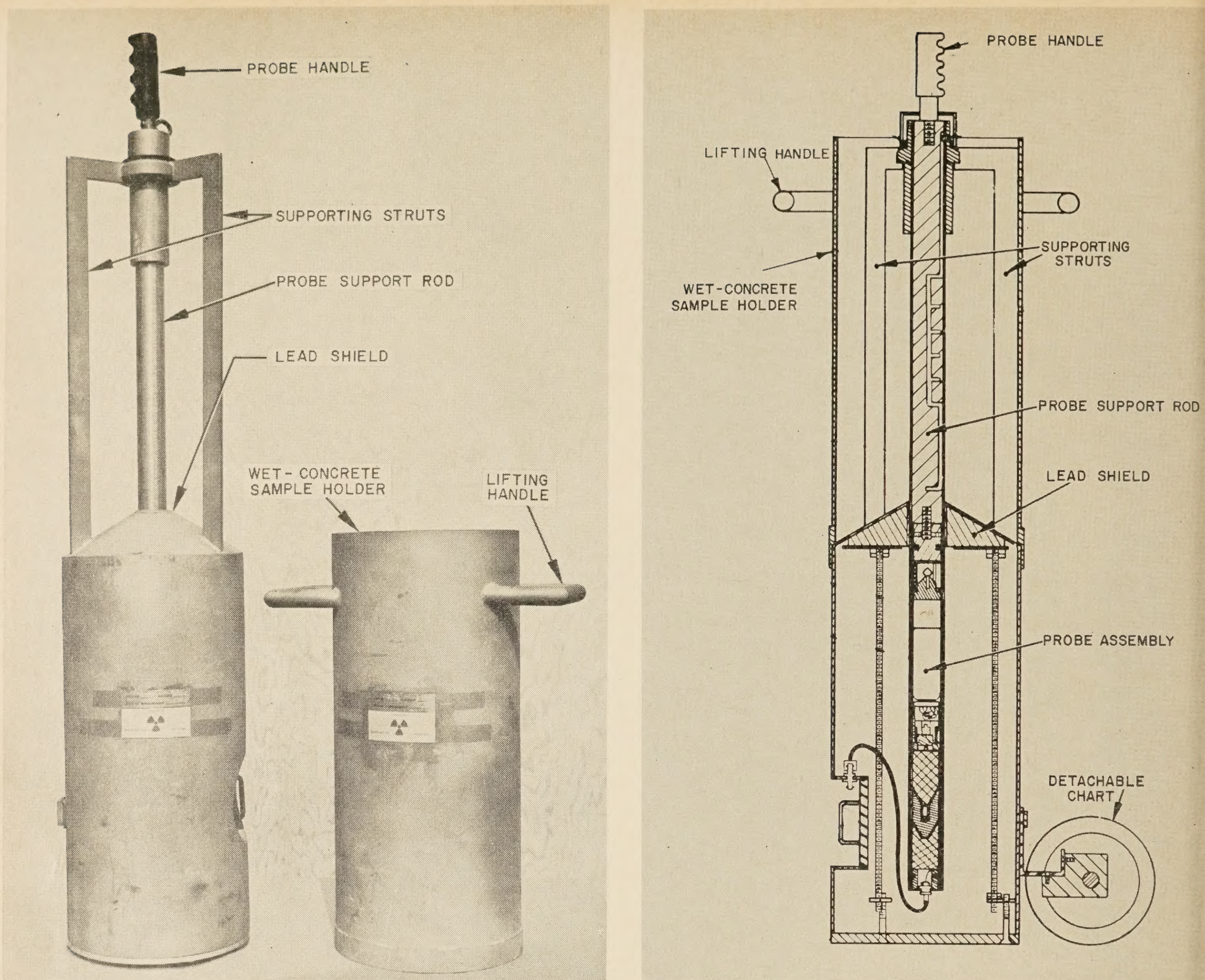


Figure 4.—Small-volume sample presentation unit with probe assembly in standard position.

Effects of Aggregate Type

It is clear that the Am^{241} backscatter intensity is strongly dependent on the type of aggregate used in the mix. This is not unexpected because the chemical composition from one aggregate to another is highly variable.

In a previous report (*2*), experimental results on four siliceous and one calcareous aggregate showed that by normalizing each intensity to that obtained on the aggregate alone, the four siliceous curves fell on a single calibration curve. The calcareous aggregate gave a different response, but still with sufficient sensitivity to permit a reasonably accurate measurement.

Only small aggregates were used in the early studies and differences in composition of the siliceous aggregates were fairly small. This was obviously not the case for the coarse aggregate siliceous mixes used in current studies. Here the effect of the limestone content of the coarse fraction is very marked. Limestone, in fact, creates problems and has not been accurately measured by this system.

In the present study, Am^{241} backscatter measurements were also made on dry samples of aggregates to investigate the possibility of normalizing the response. Measurements were made separately on fine and coarse material and also on a three parts coarse to two parts fine mix. For fine material the intensity variation between remixes was less than 1 percent and within acceptable statistics. For coarse and coarse-fine mix, the relative standard deviation in the average intensity was about 5 percent.

The intensity obtained for the three parts coarse to two parts fine mix provided a good normalization point, although the fit was not perfect. The intensity on the raw aggregate was very sensitive to sample packing density, which was the reason the coarse-fine mi gave a lower reading than either the coa fine material alone.

One encouraging aspect of this investig was the relatively small change in the slo the calibration curve in contrast to the change in intensity between aggregates. suggests that by more careful measurem the aggregates and with more extensive e ence, it may be possible to predetermin response curve of a mix containing a c aggregate to an acceptable degree of accu

Effects of Water Content and Density

Water content per se in the range of 8.3 percent introduces a negligible error i interpreted cement factor. Errors result from the accompanying change in density.

Bulk density effects are significant, and typically a change of ±1 lb./cu. ft. produces an error equivalent to ±0.3 sack/cu. yd. in the determined value of cement content.

The method of bulk density measurement employed in these experiments was not always accurate, and errors led to imperfect correlations. It was noticed, for example, that the density gage reading was somewhat sensitive to the surface water layer and, in general, was affected by sample compaction at the surface; whereas, the subsurface Am^{241} backscatter gage was not.

Reliability Study

Additional measurements on the hardened concrete standards and further investigation of the Am^{241} subsurface backscatter gage design well demonstrated its potential for determining cement quantity in concrete mixes of known aggregate type.

Despite the long source-detector separation of the Cs^{137} gage, the correlation between the scattered intensity and that of the Am^{241} gage appears good.

For aggregate mixes containing a high proportion of limestone, the accuracy required of the density gage is more stringent, and it is unlikely that the desired accuracy goal of ±0.3 sack/cu. yd. will be reached. Other difficulties with aggregates, e.g., chemical heterogeneity from sample batch to sample batch, are also evident. Further measurements are needed.

Theoretical Drawbacks of Other Nuclear Approach: Activation Analyzer

Investigations of activation analysis on bulk material have so far been inconclusive. Spectrum interference is a major problem unless high resolution detectors are used, and these require continuous liquid nitrogen cooling. Half-life discrimination can be applied, but this adds to the analysis time and also introduces more sophistication in technique. Furthermore, high intensity neutron sources are required. Machine generator sources are unattractive for field use and radioisotope neutron sources of sufficient output require substantial shielding, thereby sacrificing portability.

Despite its drawbacks, neutron activation probably has the best future potential in the analysis of cement in concrete. However, neutron activation would require a radioisotope source such as Californium-252 and relatively sophisticated electronic apparatus.

Evaluation of Gamma-Ray Scattering

The method of low-energy gamma-ray scattering using the Am^{241} source in submersible backscattering geometry appears to offer a practical means of cement content determination in concrete mixes comprising aggregates that are predominately siliceous in character. Although applicable to aggregate mixes which contain some limestone, the accuracy degenerates rapidly with increasing limestone content as a result of the reduction in the intrinsic sensitivity to cement factor and the increase in the bulk density influence.

The submersible gage design allows a representative sample measurement and does not appear to be affected by aggregate particle size and water content per se.

For siliceous type aggregates, the necessary compensation for bulk density variation can be accomplished by using a gamma-ray backscatter gage employing Cs^{137}. Cesium can be accommodated conveniently in the same probe envelope and used as a practical method of carrying out the two radiation measurements from which the minimum of interference of one on the other was developed. The measurement technique also minimizes the effect of electronic drift, ambient temperature change, and other factors which would deny operational reliability.

Further laboratory evaluation is clearly needed to investigate the major technical problems of aggregate heterogeneity of batch samples and bulk density determination.

Aggregate composition is the most outstanding and obvious source of difficulty and, although it appears possible to classify the aggregate and hence the calibration curve by radiometric determination, it will be necessary to accumulate more data with a larger variety of aggregates to establish a workable set of curves.

For bulk density measurement, the long-spacing Cs^{137} source arrangement appears adequate for predominantly siliceous aggregates where an accuracy equivalent to 1 lb./cu. ft. in density is required to achieve an accuracy equivalent to 0.3 sack/cu. yd. in cement factor. To obtain the higher accuracy in the bulk density (~0.3 lb./cu. ft.), which is obviously required in the calcareous aggregate mixes, redesign will be necessary.

Future Design Improvements

The basic design is flexible and could accommodate future improvement and modification, such as:

1. Improved means of detector gain stabilization to permit more extensive use of the probe for, example, in the hand-held accessory, with less frequent standardization.
2. Investigation of alternate scintillation crystals and the employment of a steel filter to improve the accuracy of the bulk density measurement on matrices of variable chemical composition.
3. Possible relocation of the high- and low-energy backscatter components to simplify operation in the wet sample holder and in the sheath unit.

Although it appears that this device has an excellent chance of eventually being implemented, it is not ready for immediate field application. Its use, this far, is severely limited to siliceous aggregate mixes. It does not accurately measure the cement content of calcareous aggregate mixes, and most concrete mixes contain calcareous aggregates.

Additional laboratory research and evaluation at FHWA's Fairbank Highway Research Station is in progress—its purpose: to evaluate the instrument and determine whether a basis exists for supporting the research necessary for refinement of the device so that in the field it will accurately determine the cement content of any concrete mix.

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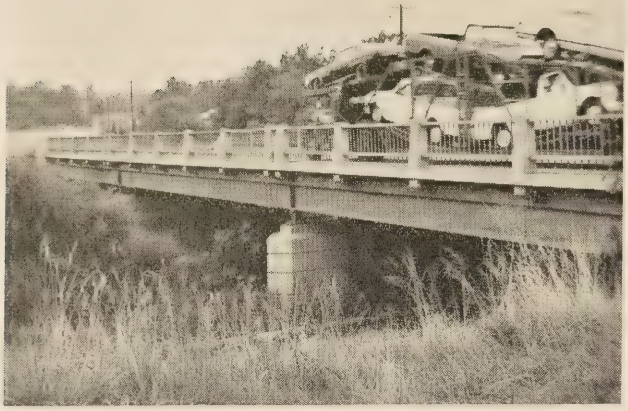

Loading History of a Highway Bridge— Comparison of Stress Range Histograms

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

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Results of a loading history test—measuring truck stress waves—on a rural bridge in Maryland are presented here. Two data gathering techniques, both employing histograms, are discussed and their application described. Only one stress response per truck crossing is recorded by one technique, although more than one is recorded by the other procedure. Composition and weight of truck traffic were taken into consideration during the test, as were occurrences of multiple crossings, which produced higher average stress ranges than single crossings. Several methods of estimating bridge fatigue life are also presented.

Introduction

FOR several years extensive field tests have been performed to determine the loading history of the main load carrying members of highway bridges. Promoted on a national scale by the Federal Highway Administration (FHWA), this project has been guided by committees from the American Society of Civil Engineers and the Highway Research Board.

Actual field testing and data gathering are being done by various agencies; consequently, some differences occur in the final data presentation. A series of stress range histograms, in which the magnitude of the stress range is plotted as the abscissa and the percentile of the total number of stress ranges as the ordinate, is normally the primary product of each study. Sizable variations in percentages based on the same number of truck passages may occur, depending on how many vibrations caused by a single truck are recorded.

As it is desirable to draw some common conclusions from these tests, and because the field test results are being adapted to laboratory fatigue tests, some standardization of recording, data reduction, and presentation is important.

Differences that may result in the shape of a stress range histogram, as well as the resulting importance of these differences, are described herein. In addition, a common approach to data presentation is suggested, and several methods relating field test results to an estimation of the structure's fatigue life are explored.

Field Test

The comparison of results is based on a cooperative field test conducted on a bridge in Maryland in July 1969. A crew from the University of Maryland's Civil Engineering Department prepared the bridge site for testing and attached strain gages. Separate but simultaneous data recordings were made, from eight sets of two adjacently placed gages, by the University of Maryland crew and by a crew from the Structures and Applied Mechanics Division of the FHWA.

The bridge, a three-span continuous structure built in 1949 is located on U.S. 301 near Maryland Route 4. Constructed of steel WF sections with a 7-inch thick concrete deck, the bridge carries southbound traffic across a small stream. Views of the bridge are shown

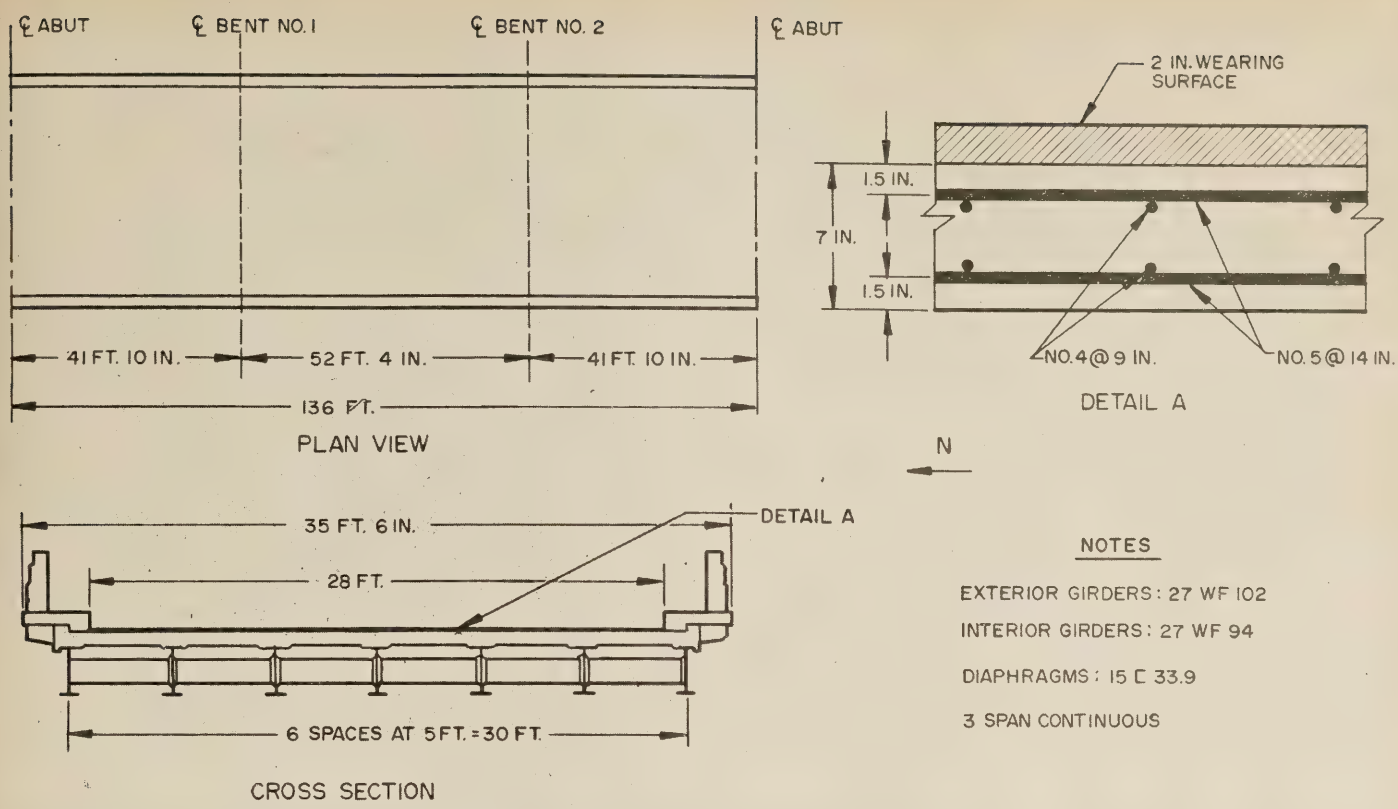

Figure 1.—Bridge structure—U.S. 301, Marlboro bypass, southbound.

n the photographs, and construction details re given in figure 1.

Eight sets of strain gages were placed on hree of the seven girders at three cross ections (fig. 2). Most comparisons in this eport were based on readings obtained from age position B1, which is on the second nterior girder on the right side facing south nd at the middle of the first span. A more xact description of gage locations is given in he University of Maryland report (*1*) [1].

The two gages at each location were ori- nted in the longitudinal direction on the ottom flange and placed side by side as lose as physically possible. One gage served he University of Maryland's recording equip- nent and the other served FHWA's equip- nent. There should have been no differences n strain readings between the two gages.

Calculated design stresses, based on the 1944 ASHO Bridge Specifications, for a point orresponding to gage position B1 were: ead load, 5.9 k.s.i.; and live load with npact, 12.1 k.s.i.

Data Acquisition and Reduction

University of Maryland Data.—The data cquisition system used by the University of

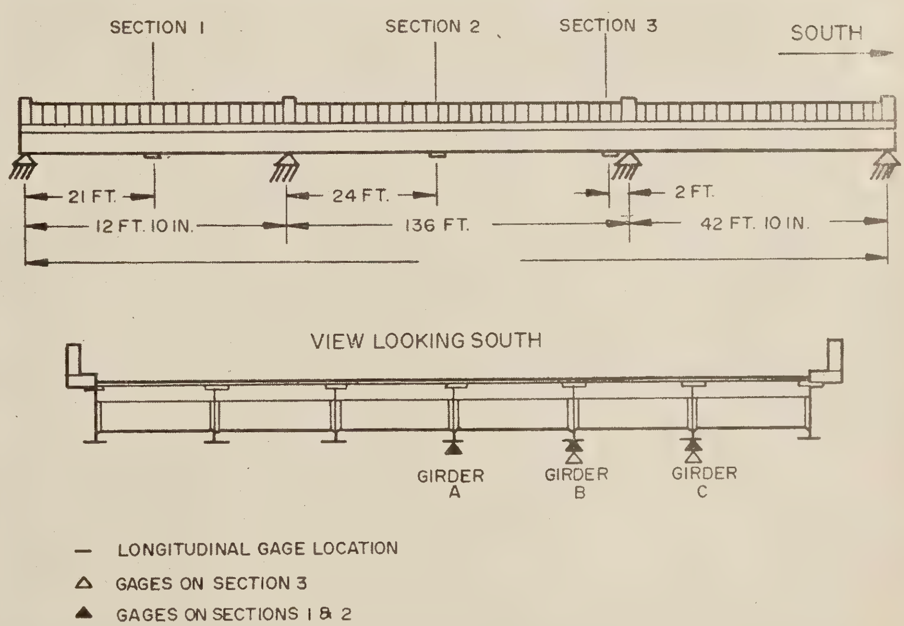

Figure 2.—Strain gage locations.

[1] Numbers in parentheses identify the references listed on ge 191.

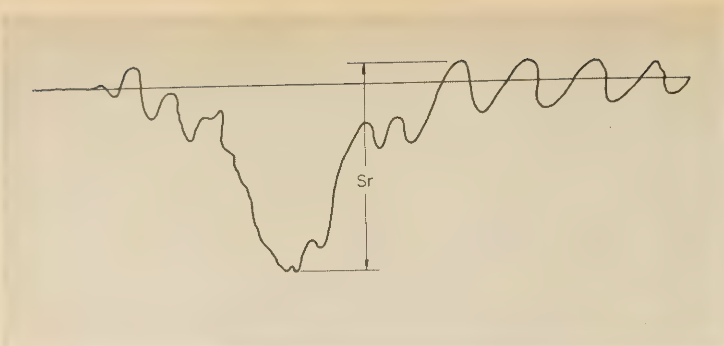

Figure 3.—Typical strain trace caused by 3-axle dump truck.

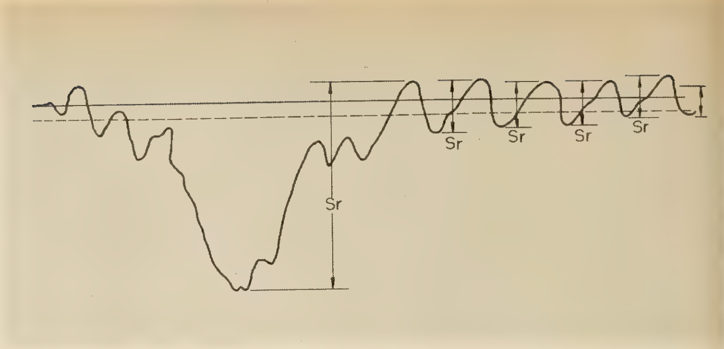

Figure 4.—Definition of stress ranges, S_r.—FHWA equipment

Maryland recorded a strain trace from gage B1 each time a truck crossed the structure. A typical trace or response curve, produced by a three-axle dump truck is shown somewhat idealized in figure 3. Different axle configurations produced slightly different traces.

The University's instrumentation record traces from each gage on oscillograph paper for the passage of 1,275 trucks. The principal data reduction for this test consisted of obtaining the maximum stress range only for each record. Maximum stress range, S_r, is defined in figure 3. Notes made during the test permitted each record to be related to a specific truck. Occasionally two trucks crossed the bridge simultaneously; however, only one record was produced.

Dynamic field data were obtained with a light beam oscillograph and two 4-channel carrier amplifiers. A time-line generator and event marker system permitted evaluation of the axle spacings and vehicle speeds.

The dynamic records were edited and read on a digital data reduction system. The system translated selected points on the record to a digital output, which was punched on punch cards.

Data from the vehicle classification notes were also punched on cards. Several computer programs were subsequently used to further process the data and produce the desired output of strains and vehicle types.

FHWA Data.—This data acquisition system used by FHWA was an automated computer controlled system that takes output analog voltages from the strain gages, digitizes the voltages, and stores and tabulates strain ranges for specified periods of time (*2*). Visual strain traces were not recorded, nor were notes taken to relate individual strain ranges to specific trucks.

The usual period of recording was 1 hour, 4 minutes of which were used for typing the results. No strains were recorded during typeouts. Data between the two recording systems for selected hours were correlated by marking the exact time on the University's notes when sampling began and ended on the FHWA system. Strains were sampled for 63 hours by the FHWA system, sometimes continually for 24 hours.

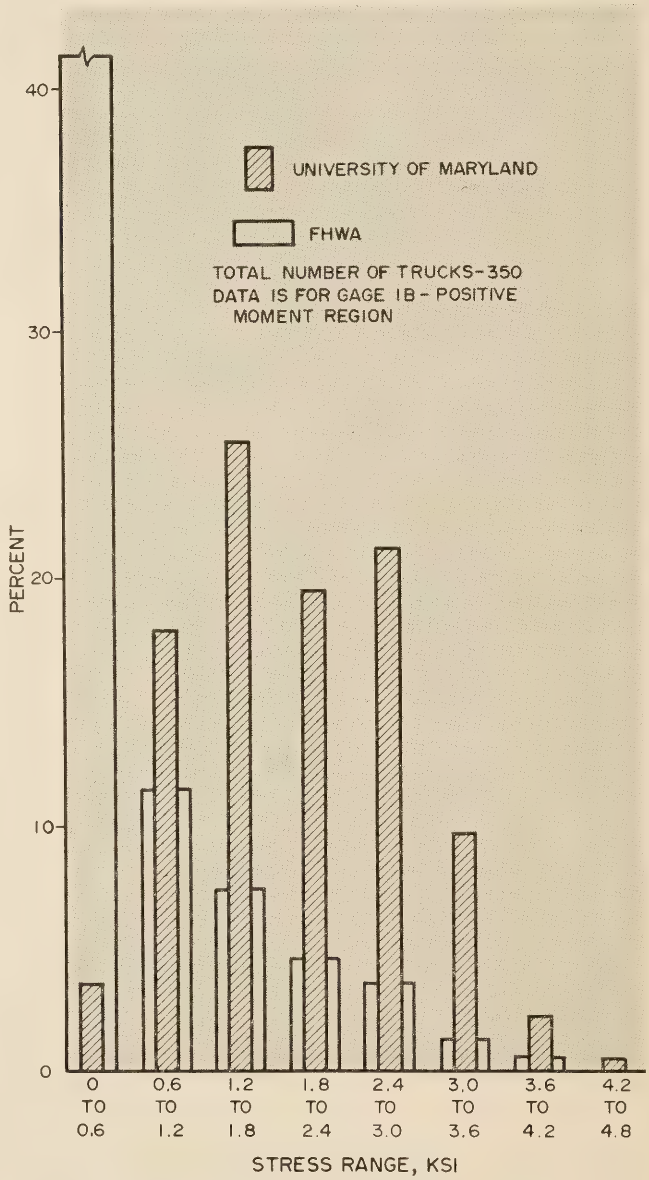

Figure 5.—Comparison of stress range histograms.

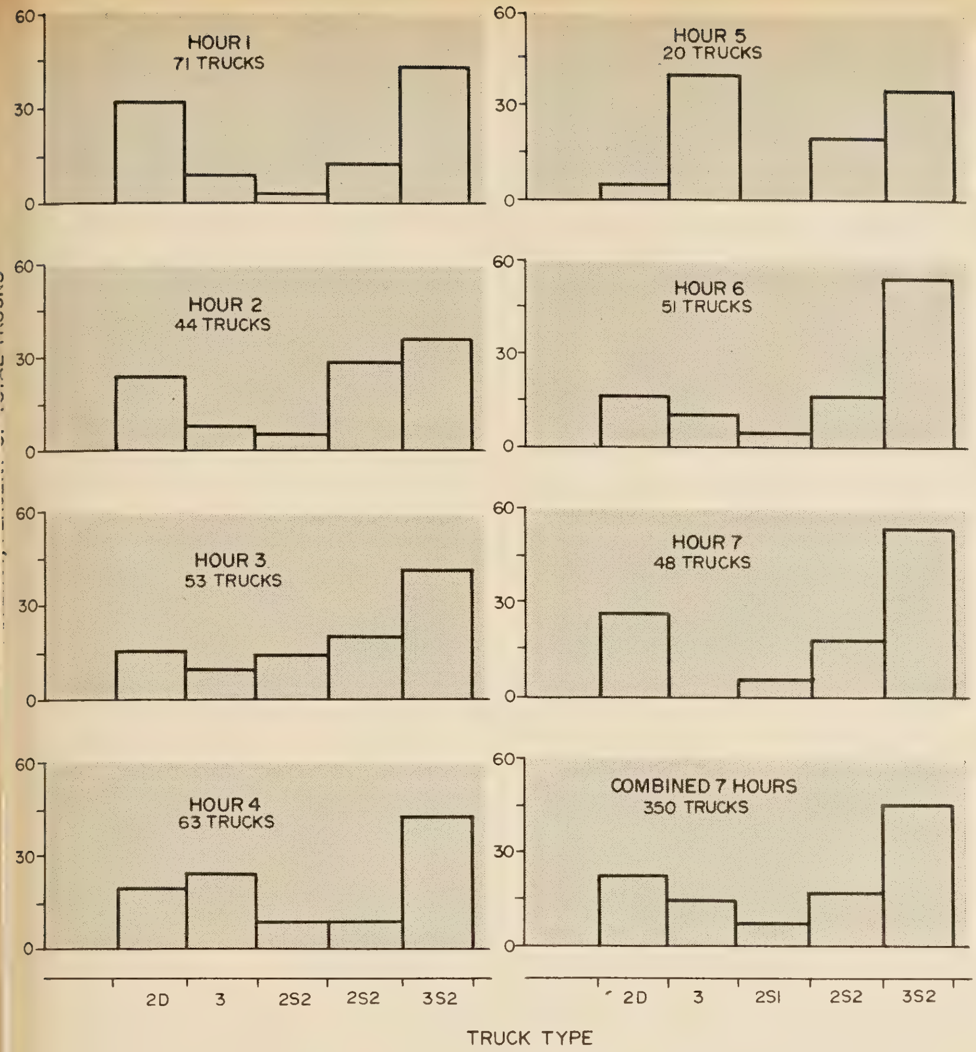

Figure 6.—Distribution of truck types, 7 selected hours.

A definition of a stress range recording in e FHWA system is shown in figure 4. The shed line represents a level of strain below iich no recordings are made. This level, 5 croinches per inch of strain in the subject t, was set to eliminate counting small rations produced by automobiles.

Figure 4 shows that, in addition to the major ess range, several other stress ranges are orded as long as the trace goes beyond dashed line and returns to the zero level.

ble 1.—Number of stress ranges during 7-hour comparison

Stress range	Number of occurrences	
	University of Maryland	FHWA
k.s.i.		
.2–4.8	1	--------
.6–4.2	7	8
.0–3.6	32	28
.4–3.0	71	80
.8–2.4	65	100
.2–1.8	86	163
.6–1.2	60	251
.0–0.6	12	1,605
Total	334	2,235

Thus, it is possible for one truck to produce a number of stress ranges at a point on a bridge. In fact, some secondary stress ranges caused by one truck may be larger than the maximum stress range from another truck.

Traffic Description.—Rural high-speed traffic crosses the bridge from a nearly flat or gently rolling terrain. The speed limit is 55 m.p.h., but vehicle speeds were not measured in the study reported here.

The average daily traffic, based on a 1968 traffic count, was 10,500 vehicles (17 percent trucks). The makeup of the truck traffic is described later.

Comparison of Field Data

Stress Range Histograms.—The following comparisons are based on data obtained from 7 selected sampling hours. Both Maryland University and FHWA data were recorded from the same 350 trucks. Trucks missed by the FHWA system during typeouts were excluded from the Maryland University data. Therefore, any differences apparently were due to the difference in the number of stress ranges only.

The two stress histograms of the 350 trucks are shown in figure 5. A marked difference appears in the two sets of data, which show a very large percentage of small values (600 p.s.i.) for the FHWA data. Note that the stress ranges always originate from zero. For example, the bars above 2.4 to 3.0 k.s.i. represent a stress range from zero to some value between 2.4 and 3.0 k.s.i.

The actual numbers of stress ranges for the two sets of data are given in table 1 where, but for one exception, the FHWA column shows a greater number of stress ranges in each level. In the Maryland University data, only 334 stress ranges were produced from the 350 trucks because there were several multiple crossings that produced only one record.

Truck Traffic Composition.—Composition of truck traffic is shown in figure 6 for each hour of the 7-hour comparison, as well as for the 7-hour total.

No trucks were weighed during the test, but in July of the previous year (1968), all trucks in the adjacent northbound lane were weighed during a 7-day period (*3*). There is no reason to believe that northbound traffic is different from southbound traffic, nor is it probable that the mean gross weight of the trucks changed in 1 year. Therefore, it is assumed here that the mean gross weights of the five truck types were as follows:

2D	10–15 kips
3	45–50 kips
2S1	25–30 kips
2S2	35–40 kips
3S2	50–55 kips

For the 7-hour comparison, the frequency of assumed mean gross weights per truck type is shown in the left half of figure 7.

Stresses above 3 k.s.i.—According to previous laboratory fatigue tests, stress ranges below the material's fatigue limit seem to have no effect on the structure's life expectancy and can therefore be ignored. However, recent tests at Lehigh University (*4*) on the fatigue of weldments tend to show that there may not be a material fatigue limit. There is however a practical fatigue limit, in that it would take almost forever for small stresses to cause detectable damage.

But what are small stresses? If all the stress ranges below 3.0 k.s.i. are dropped from both columns of table 1, very few numbers remain and it becomes difficult to make statistically meaningful data comparisons. Even so, a t-test shows that at the 95 percent confidence level there are no significant differences in the means of the two sets of data above 3.0 k.s.i.

A more meaningful comparison of the Maryland University and FHWA data can be made if all values above 3.0 k.s.i. are compared for the entire test period (table 2). The Maryland University data were recorded only during daytime hours, whereas FHWA data include some nighttime traffic. The previously selected 7-hour data (table 1) are included in table 2. Maryland University data were collected on 1,275 trucks during random hours over 4 consecutive days. FHWA data were collected in 61 hours and involved an estimated 2,500 trucks.

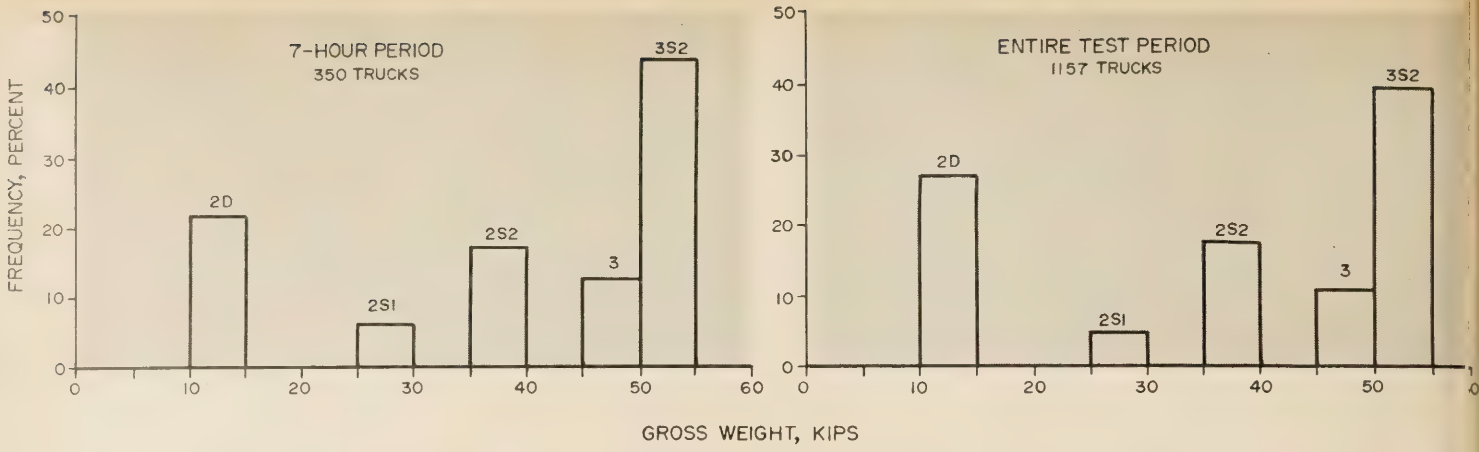

Figure 7.—Frequency of mean gross weight.

No significant difference between the means of the two sets of numbers in table 2 was found at the 95 percent confidence level. When these numbers are converted to percentages, the stress range histograms shown in figure 8 represent the best estimate of the stress ranges above 3.0 k.s.i. to which the bridge is subjected at the maximum positive-moment section in the end spans during present-day traffic. The corresponding truck traffic distribution with assumed mean gross weight is shown in the right half of figure 7.

Multiple Crossings.—The bridge under investigation is a 2-lane structure, and occasionally two or more trucks cross the bridge at the same time. It is also possible to have more than one truck in the same lane at the same time as the three spans total 136 feet.

Multiple crossings were noted and recorded during some of the sampling periods. Table 3 compares the stress ranges produced by 53 multiple crossings with 1,170 single crossings. The stress-range mean value was approximately 2.3 k.s.i. for multiple crossings, and approximately 1.9 k.s.i. for single crossings. The difference is significant at the 95 percent confidence level, although none of the 53 multiple crossings produced a stress range greater than 4.2 k.s.i.

The 53 multiple crossings compared with the 1,170 single crossings should not be taken as a true indication of the frequency of occurrence of multiple crossings. The definition of *multiple crossing* in this test is very loose: it means that there was not time to make two clearly separate strain records when two or more trucks were on the bridge simultaneously. Trucks could have been separated by as much as 150 feet as they crossed the bridge. Moreover, the 53 multiple crossings and the 1,170 single crossings occurred in about 32 daylight hours during a 4-day period.

Probable Fatigue Life

Given certain material behavior characteristics under cyclic loading, and with certain other assumptions, an attempt can be made to determine the probable fatigue life of this structure.

Since there are no stress raisers, as in partial length cover plates, it is assumed that the governing material behavior will pertain to plain rolled beams. The most critical section is the positive-moment section in the side-spans, where B1 is located. The bridge is made continuous by riveted splices, which of course are not in high-moment locations. The negative stresses, both calculated and measured, were not as high as the positive-moment stresses.

There are several different methods of estimating the structure's fatigue life, each somewhat related. These methods and the assumptions associated with them are presented next.

Root Mean Square Method.—Some limited laboratory fatigue data (*4*) indicate that random, variable load stress ranges can be related to constant stress ranges by calculating the root mean square of the variable ranges. This is then assumed to produce the same damage as the constant stress range of the same value. By using the measured stress ranges and their frequencies above 3.0 k.s.i., a root mean square stress of 3.6 k.s.i. can be obtained.

In recently completed extensive constant-cycle fatigue tests at Lehigh and Drexel Universities, the following relationship between constant cycle life and stress range for A–36 plain rolled sections was developed:

$$\text{Log } N = 10.637 - 2.943 \text{ Log } S_r \qquad (1)$$

where,

S_r = stress range

The laboratory investigation at the universities did not go beyond 10 million cycles, but it is assumed that equation (1) holds true beyond this point. By substituting 3.6 k.s.i. for S_r in the equation, a life of 1×10^9 cycles is obtained.

The present average daily traffic (*ADT*) across the bridge is about 10,000 vehicles. If it is assumed that 20 percent of the ADT is truck traffic, about 730,000 trucks will cross the bridge each year, and if it is further assumed that 12.5 percent of these trucks produce 5 percent stress ranges above 3.0 k.s.i., about 182,500 damage producing stress ranges will occur per year. If whatever the traffic lacked in weight and frequency in the 20 years of the bridge's life is made up in the next 20 years, $40 \times 182{,}500 = 7{,}300{,}000$ cycles of damage producing stress will have occurred in the 40-year span. This is still far below the 1×10^9 cycles to failure previously mentioned.

Another procedure is to calculate backwards in equation (1). Substituting 7.3 million cycles for N results in a stress range of 18.7 k.s.i., which is above the combined dead- and live-load allowable stress.

Table 2.—Stress ranges above 3.0 k.s.i.

Stress range	Number of occurrences	
	University of Maryland	FHWA
k.s.i.		
5.4–6.0	1	2
4.8–5.4	1	3
4.2–4.8	7	10
3.6–4.2	36	51
3.0–3.6	107	230
Total	152	296

Table 3.—Stress ranges induced by multiple crossings

Stress range	Number of occurrences	
	Multiple crossings	Single crossings
k.s.i.		
5.4–6.0	----------	1
4.8–5.4	----------	1
4.2–4.8	----------	7
3.6–4.2	4	32
3.0–3.6	7	100
2.4–3.0	12	201
1.8–2.4	15	246
1.2–1.8	10	297
0.6–1.2	5	251
0.0–0.6	----------	34
Total	53	1,170

Miner's Method.—A common procedure for stimating cumulative fatigue damage is to se Miner's hypothesis (*5*), which says that amage is proportional to the number of pplied cycles divided by the total number ecessary to produce failure at a certain stress inge, and that the summation of all the actions at the various stress ranges is equal unity at failure.

This method requires an appropriate S–N irve and the number of cycles at the various ress ranges must be known or properly timated. Instead of an S–N curve, equation) will be used, again with the assumption iat it holds true beyond 10 million cycles.

Based on the field test, it is estimated that e 182,500 yearly stress ranges above 3.0 s.i. are distributed as follows:

k.s.i.	*Stress range*
3.3	127, 750
3.9	36, 500
4.5	9, 125
5.1	5, 475
5.7	1, 825
6.3	1, 825

so, it is again assumed that both the number d the distribution of stress ranges can be ld constant for 40 years.

The summation of the fractions is less than 005, which confirms the previously deter-ned fact that the tested bridge does not ve a fatigue problem.

Extreme Load Method.—It is often argued it, although there is no present danger of igue distress in a highway bridge, the ight and number of trucks using it continue increase, and this must be considered in ure bridge life. This argument is not irely valid because it should be recognized t neither the weight nor the number of cks on any road can increase without its.

Truck volumes are rarely more than 200 per ır on a highway. If the truck percentage reases, not only does the total traffic volume rease, but the average traffic speed also reases and the level of service generally eriorates. However, to illustrate the effect ın extreme load condition, it is attempted e to produce an upper limit of truck traffic acity across this bridge.

The *Highway Capacity Manual* (*6*) indicates t it is possible under ideal conditions to e 420 trucks per hour on a 2-lane, one-way, l highway, in addition to some 1,200 cars, traveling at 35 m.p.h. This adds up to 080 trucks per day or 3,679,200 per year. remaining service life of 30 years more is ımed, the bridge will then supposedly y about 100 million truck crossings.

t will be assumed for this calculation that east half of the trucks are fully loaded to kips—a gross weight limit that is not rely unreasonable. This of course is not stantiated by the present field tests.

n approximate relationship between gross ght and live load stress range was developed n earlier investigation (*7*) for simple spans. Adapting this relationship to the present case, the following expression evolves:

$$S_r = 1.3 + .053G_w \qquad (2)$$

where G_w is the gross weight in kips. For 100 kips this results in a stress range of 6.6 k.s.i. Assuming this to be the root mean square stress range introduced earlier, substituting $S_r = 6.6$ k.s.i. into equation (1) will result in the 210 million cycles necessary to produce failure.

Again, it is evident that even under such abnormal traffic conditions, no fatigue distress will result in the plain rolled section under study. If the bridge had been constructed with beams that had partial length, end-welded coverplates, it could only withstand about 4 million cycles of a 6.6 k.s.i. stress range. However, it could withstand the more realistic root mean square stress range of 3.6 k.s.i. about 24 million times. Again, it is assumed that the appropriate Log N vs. Log S_r relationship is valid beyond 10 million cycles.

Discussion

The stress range histogram, as presented for the stresses above 3.0 k.s.i. (fig. 8), is considered meaningful and is representative of the present conditions on this structure.

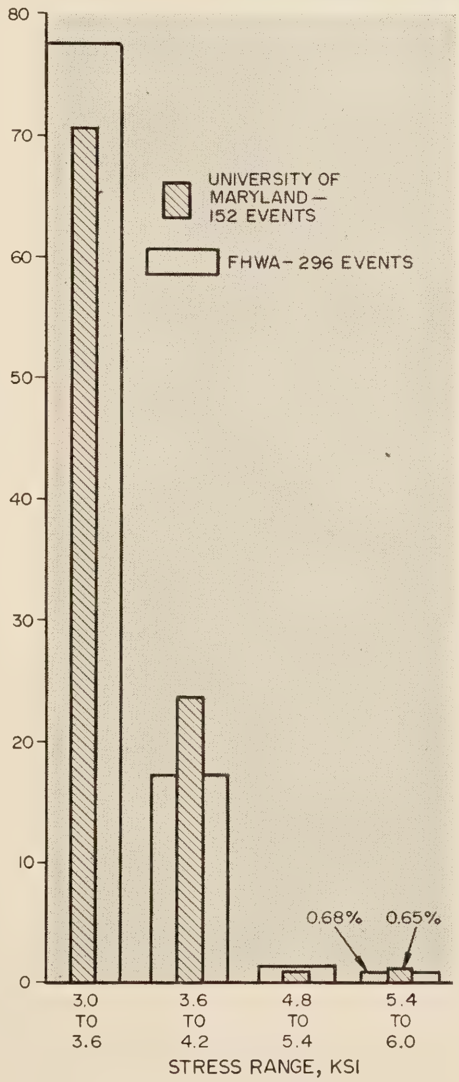

Figure 8.—Stress ranges above 3.0 k.s.i.

The truck traffic classification is also believed to be representative of true conditions. A somewhat lower reliability should be placed on the truck weight data because it was obtained the previous summer and borrowed from the adjacent roadway.

The dual truck crossing data should be regarded with caution. Apparently fewer meaningful dual crossings occur than the data indicate.

Of the ways of estimating the structure's fatigue life, the root mean square method appears most promising, although whether this approximation of a constant stress cycle is always valid must be tested further in the laboratory.

The extreme-loading method is somewhat exaggerated; however, it does serve the purpose of illustrating that for beams without stress raisers there is not likely to be a fatigue problem.

The assumption that the several Log N vs. S_r relationships remain linear beyond 10 million cycles is probably incorrect, but his assumption yields conservative results.

Conclusions

The following conclusions and recommendations are based on the field test results and the fatigue analysis.

- Significant differences in the shape of stress range histograms can occur, depending on the inclusion or exclusion of the several secondary stress ranges produced by a single vehicle.

- No significant difference in the shape of the stress range histograms resulted when only stress ranges above 3.0 k.s.i. were considered. This conclusion may not be universally applicable to other bridges; and it is recommended that in other field tests, in addition to recording the major stress range produced by each vehicle, any secondary stress ranges above 3.0 k.s.i. be recorded.

- There was a significant difference between mean of stress ranges caused by dual crossings when compared with those of single crossings. More experimental evidence on the nature and frequency of dual or multiple crossings needs to be gathered, including testing in a variety of traffic situations.

- It appears that the main load carrying members of this bridge have not and are not likely to suffer fatigue distress caused by traffic induced stresses. However, this conclusion has not been verified for the deck reinforcing steel and secondary members, such as the diaphragms.

- Further laboratory fatigue work is recommended, both to extend S–N curves to as much as 200 million cycles, and especially to determine the effects of variable loading.

(*Continued on p. 191*)

The Effect of Vehicle Positioning on Lag Acceptance

BY THE OFFICE OF RESEARCH

Reported by FRANK GATLING
and ADRIENNE WHYTE,
Research Psychologists,
Traffic Systems Division

Introduction

STUDIES on gap or lag[1] acceptance at intersections or merges can be grouped into two broad categories: (1) studies establishing the amount of time involved in the performance of gap acceptance maneuvers under varying conditions; and (2) studies concerned mainly with decision-making processes and hypotheses regarding these processes.

Greenshields, et al. (*1*),[2] who conducted the first of these studies, investigated the amount of time a driver needed to accept a gap in cross-flow traffic. It was determined that turning maneuvers required a greater amount of time than the uncomplicated crossflow gap acceptance. Studies of the same general type have been performed by Raff (*2*), Robinson (*3*), Homberger (*4*), Moore (*5*), Bissell (*6*), Solberg and Oppenlander (*7*), Herman and Weiss (*8*), and Wagner (*9*). Researchers whose studies fall into the second catagory include Brain (*10*); Hurst, Perchonik, and Seguin (*11*); and Gibbs (*12*). An excellent review of most of these investigations has been compiled by Gordon (*13*).

Many roads are so constructed that drivers of entering vehicles must seek a lag or gap from an extreme viewing angle. This is especially true of merges into moving traffic from an acceleration lane. For example, merges from a 180-degree angle require either an awkward, disorienting body and head movement or the use of a mirror just to see clearly and thus make a decision.

The purpose of this study was to investigate whether a difference exists in lag acceptance times for drivers entering the traffic flow from 90-, 135-, and 180-degree angles, with regard to the main traffic flow. At the 180-degree angle, subjects were first required to make decisions without using the rearview mirror and then using the rearview mirror only. This study was designed to test the hypothesis that the angular position of the vehicle with regard to the main stream of traffic will not affect the lag acceptance time.

[1] A lag is the period that elapses from the time a vehicle stops at an intersection and the first vehicle from the through street enters the intersection. A gap is the period that elapses between the arrival of two successive through-street vehicles in the intersection.

[2] Italic numbers in parentheses identify the references listed on page 191.

Procedure and Apparatus

Twelve subjects, seven males and five females, took part in the study. Three subjects were employees of the Federal Highway Administration and nine were hired for the experiment. Ages ranged from 17 to 53 years, with the median age being 23.5 years.

A 1966 Chevrolet 4-door sedan served as the observation car in which the subjects sat. Because of mechanical problems two approach cars were used: a 1964 Ford station wagon and a 1968 Oldsmobile 4-door sedan. Both cars were equipped with speedometers that were certified for true speed and were calibrated in increments of 2 m.p.h.

Two roads at FHWA's Fairbank Highway Research Station were used for the experiment. The approach vehicle traveled on the *main road* and the observation vehicle with its subject driver was stationed on the *side road.* A 2-×3-foot white sign, mounted on a stand, was placed on the shoulder of the main road at the experimental decision-point distances from the subject car.

For each test the subject car was positioned adjacent to the main road at one of the three angles. At the 180-degree position the subject, in one case, made a merging decision by turning his neck and body to observe the approaching vehicle. In another case the subject used the inside rearview mirror. The outside rearview mirror was kept covered at all times.

The subject assumed the position of a driver who had just halted for a stop sign before entering a main street. Right turns were made from positions of 90 and 135 degrees, but the subject continued straight ahead from the 180-degree position. While at the intersection the subject was asked to state whether or not he would merge in front of the approaching vehicle when that vehicle was even with the white sign.

Design

The complete experiment was a 2×4 factorial design, but a 2×3 analysis of variance was performed because the 180-degree angle with mirror was not a fourth angular condition. Judgment from the 90-, 135-, and 180-degree merges (with and without a mirror for 180 degrees only) were made for an approach vehicle at 125 and 175 feet. Half the subjects began their judgments with the 125-foot distance and half with the 175-foot distance. The position of the subject vehicle was also counterbalanced.

The method of limits was used to establish the judgment threshold. In a descending series, for example, the approach car changed its speed in increments of 2 m.p.h.—if the first approach was at 30 m.p.h., the second was at 28 m.p.h. and the third at 26 m.p.h. This procedure was reversed for the ascending series.

Results and Discussion

The mean threshold speeds for the angular vehicle positions and distances are given in table 1. Table 2 contains the results of the analysis of variance that was performed on the data.

The F ratio in table 2 is less than unity for the factor of angular position; therefore, the hypothesis that vehicle position will not affect lag acceptance behavior must be retained. Although not reaching the level of statistical significance, the data do indicate that subjects tend to accept higher speeds (i.e., shorter lag intervals) as the vehicle position changes from 90 to 180 degrees. This is true at both the 125- and 175-foot distances. Several t tests were performed on the data between the 180-degree position without the mirror, and with the mirror for both the 125- and 175-foot distances. The t ratios are not significant. As would be expected, there is a significant difference between the approach speed thresholds for the 125- and 175-foot judgment distances.

Table 1.—Mean threshold speeds

Angle	Vehicle speeds	
	125 feet	175 feet
degrees	*m.p.h.*	*m.p.h.*
90	17.2	25.3
135	17.6	25.3
180	19.0	26.6
180 (mirror)	19.9	28.2

Table 2.—Analysis of variance

Condition	Sums of squares	D.F.	Mean squares	F
Distance	1077.72	1	1077.72	22.40 ($p<0.01$)
Angle	34.04	2	17.02	.35 N.S.
Angle × distance	1.05	2	.52	.01 N.S.
Within replicates	3174.65	66	48.10	----------

The mean velocity thresholds were converted to time thresholds (table 3). The most interesting information from the data in table 3 is that the size of the accepted lag, when expressed as a time, is remarkably similar for both distances. It appears that a driver allows himself the same amount of time for this maneuver at both distances despite different approach speeds at 125 and 175 feet. This evidence supports a time theory of gap or lag acceptance.[3]

The analysis of variance indicates that there is no significant difference between the angles at which the vehicle is positioned, but there is a significant difference well beyond the 0.01 level for the two distances at which judgments are made.

Previous studies have reported mean lag or gap acceptance times for a right turn from a 90-degree position. Bissell (*6*) reported a time of about 5.5 seconds but Solberg and Oppenlander (*7*) indicated a time of over 7 seconds. The shorter acceptance time of the present experiment probably results from drivers not actually having to drive in front of the approach vehicle. Apparently the difference represents a lowering of the driver's normal caution.

[3] The time hypothesis is that a driver makes a precise estimate of the time of arrival of an approaching vehicle.

Table 3.—Mean thresholds converted to time (secs.)

Angle	Time	
	125 feet	175 feet
degrees	*sec.*	*sec.*
90	4.8	4.6
135	4.7	4.6
180	4.4	4.4
180 (mirror)	4.2	4.1

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Loading History of a Highway Bridge

(*Continued from p. 189*)

ACKNOWLEDGMENTS

The reported work was a cooperative venture involving several administrative divisions of the Maryland State Roads Commission, the Federal Highway Administration, and the University of Maryland. The authors express their gratitude to all those who cooperated in the research. The help and counsel of Messrs. A. D. Sartwell, W. L. Armstrong, and H. R. Laatz are especially appreciated.

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Resurfacing—one method of retiring pavement discussed in this article.

Service Lives of Highway Pavements—A Reappraisal

BY THE OFFICE OF HIGHWAY PLANNING

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The analyses presented here are based on data submitted by 19 States covering their construction and retirement activities from January 1, 1929 to January 1, 1968. Because every State did not construct every roadway-surface type, the results of some analyses are for a composite of less than 19 States. Of the 170,296 miles of highway pavements retired, 59.3 percent was resurfaced, 29.5 percent reconstructed, 1.6 percent abandoned, and 9.6 percent transferred to other public authorities.

On January 1, 1968, the average age of the intermediate-type surfaces was 15.5 years, and of high types, 11.6 years. Corresponding remaining life expectancies of the mileages in service were 6.2 and 10.9 years. The analyses indicate that average age and remaining life expectancy of the high-type surface are stabilizing after a long period of increasing age and decreasing remaining life.

Based on the service lives developed in this study, the probable miles remaining in service for the next 10 and 20 years were estimated. Of the total miles remaining in service on January 1, 1968, it was estimated that 60 percent will be retired for different reasons by January 1, 1978, and 87 percent by January 1, 1988.

Introduction

HIGHWAYS do wear out. Because they wear out, highway programs must consider continuous resurfacing and reconstruction operations to maintain highways in a usable and safe condition. Accordingly, a knowledge of service lives of highway pavements is essential.

Research in this field has become more meaningful since the fall of 1935 when road life studies were incorporated as a phase of the statewide planning surveys.

In addition to data upon which to base calculations of average service lives of roadway surfaces, these studies also provide the means for obtaining construction costs; salvage values of retired roadway elements; and service lives of structures, gradings, and rights-of-way.

This report is the fifth in a series of comprehensive analyses of the service life characteristics of roadway surfaces conducted in the last 30 years by the Federal Highway Administration. The first was reported in 1940 and the fourth in 1968 (*1, 2, 3, 4*).[1]

Average service lives of pavements were derived from statistical analyses of construction and retirement rates, using survivor curves, similar to the process used by the life insurance business. The following road surfaces on the primary rural highway system were analyzed:

(1) Bituminous surface treated (F).
(2) Mixed bituminous (G–1).
(3) Mixed bituminous (G–2).
(4) Bituminous penetration (H–1).
(5) Bituminous penetration (H–2).
(6) Bituminous concrete (I).
(7) Portland cement concrete (J).

[1] Italic numbers in parentheses identify the references listed on p. 204.

For the purpose of further analyses, the ven surface types were also combined into wo major groups, as follows:

Intermediate type: Includes bituminous sur- ace-treated (F), mixed bituminous (G–1) ombined thickness of surface and base less han 7 inches), and bituminous penetration H–1) (combined thickness of surface and ase less than 7 inches).

High type: Includes mixed bituminous G–2) (combined thickness of surface and ase 7 inches or more), bituminous penetra- on (H–2) (combined thickness of surface nd base 7 inches or more), bituminous oncrete (I), and portland cement concrete J).

Definitions of technical terms, reasons for tiring pavements, and detailed descriptions surface types are listed under *Definitions* the end of this article.

Road Life Studies Essential

In the last 10 years, the increase in highway affic and in heavy axle-load applications as placed a tremendous burden on existing ghway pavements. What effect has this affic increase had on the service lives of ghway pavements?

The highway plant itself is the proving ground from which can be obtained facts relating to past and present performance of the different types of highway construction under varying degrees of use. An evaluation of these facts is essential to sound planning and engineering design for the orderly development of a highway network. Road life studies conducted by State highway departments provide the means for obtaining data upon which such evaluations can be made. Among the major objectives of road life studies is the development of factual data relating to average age, probable life, remaining life expectancy, and what is done to the roadway surface at the time of retirement and replacement.

Facts concerning service lives of various road-surface types are in demand by industry groups who have specific interest in road types that use their products, by highway department administrators who require data on replacement intervals, and by engineers who need to know the service life of roads so that they can design them to carry desired traffic loads over the expected service life. In economic studies, knowledge of the life expectancy of both new and existing construction can be of utmost importance in assigning economic life and terminal values to existing and future investments.

Results of Analyses

The analyses presented are based on data submitted by 19 States covering their construction and retirement activities from January 1, 1929, to January 1, 1968. Because every State did not construct every surface type, the results of some analyses are for a composite of less than 19 States. Retirement methods for the 170,296 miles of highway pavements retired include the following: 59.3 percent resurfaced, 29.5 percent reconstucted, 1.6 percent abandoned, and 9.6 percent transferred to other public authorities. Approximately 89 percent of all retirements were by resurfacing and reconstruction, indicating no significant change in the percentage of retirement by method from that reported in the 1968 report (4).

On January 1, 1968, the average age of the intermediate-type surface was 15.5 years, and the average age of the high-type surface was 11.6 years. The remaining life expectancies were 6.2 and 10.9, respectively. The analyses indicate that average age and remaining life expectancy of the high-type surface are

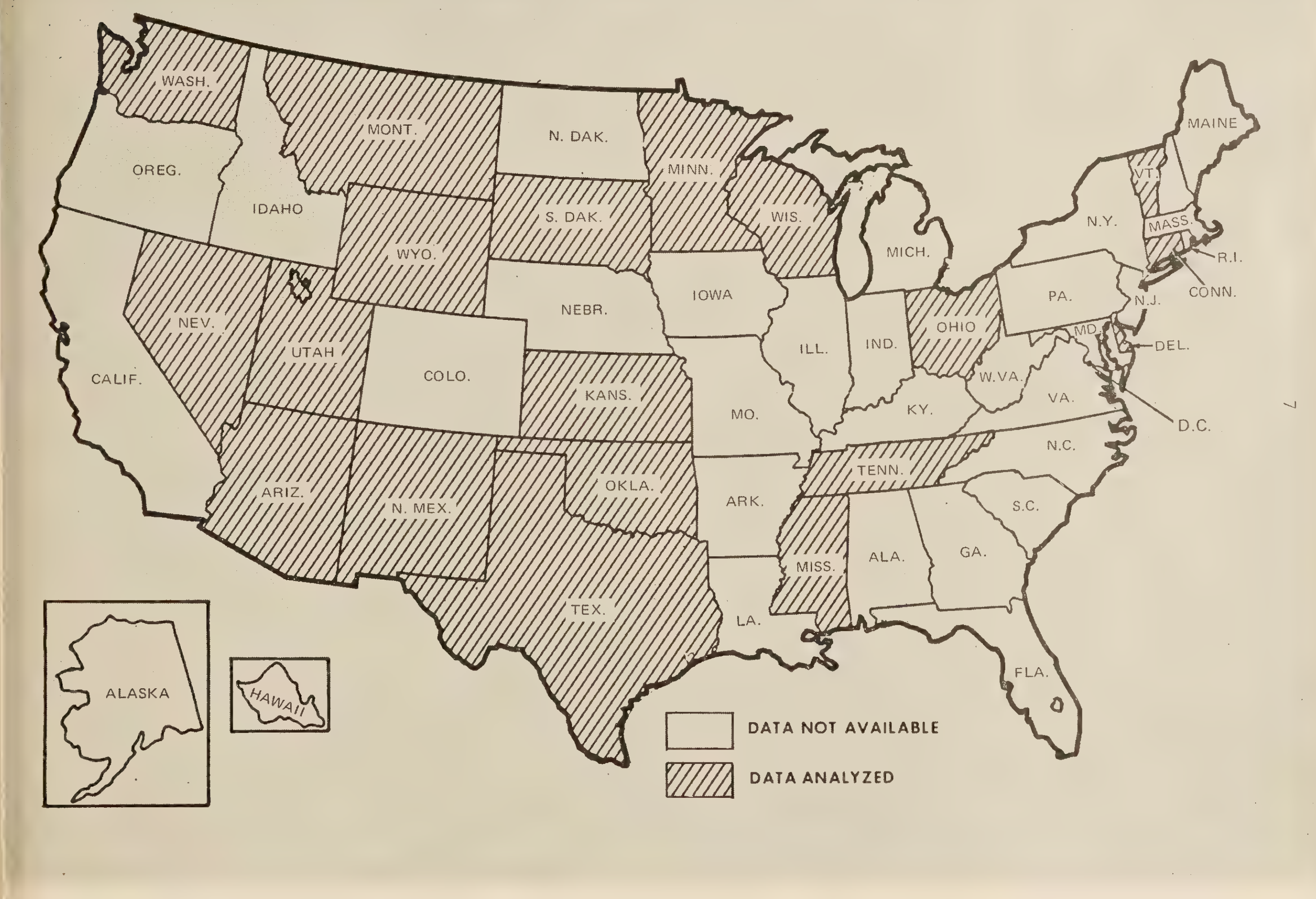


Figure 1.—States reporting road life mileage data.

stabilizing after a long period of increasing age and decreasing remaining life.

Based on the service lives developed in this study, the probable miles remaining in service for the next 10 and 20 years were estimated. Of the total miles remaining in service on January 1, 1968, it was estimated that 60 percent will be retired for different reasons by January 1, 1978, and 87 percent by January 1, 1988.

Factors Influencing Results

The average service life of a road surface is the average period after construction that the surface remains in service prior to being replaced, reconstructed, or otherwise taken out of service for any reason or by any method. In actual practice, only a small percentage of individual road sections of the same type have a service life exactly equal to the average life for that type. Service lives derived from road-life data relate to the service-life characteristics of roadway surfaces by highway system; they do not necessarily apply when a study is confined to a particular section of road, for the same reason that life insurance actuaries do not apply the average life and expectancy of humans to a particular person. It, therefore, becomes necessary when estimating the service life of a particular road section to consider such factors as age, structural condition, design features, location, and traffic conditions peculiar to that section. Only through the use of expert engineering judgment in the evaluation of these factors is it possible to arrive at an estimate of the remaining service life for a particular road section. A possible procedure for estimating remaining service life of existing pavement sections is discussed by Corvi and Bullard *(5)*.

The many factors that have an impact on construction practices and in turn influence the trend of expected service life of road surfaces include: administrative policy, availability of materials and manpower, change or influence of politics, increased activity due to State legislative action, construction activity in neighboring States and in other fields of construction, civil defense activity, and any unusual or extended nationwide highway program.

Moreover, these factors tend to influence service lives, age and expectancy, methods of retirement, and so forth. The composite influence of such factors can be determined only from the findings of road life studies conducted on a system basis. With these findings, the planning for future highway development can be undertaken with more confidence.

Basic Data Compiled

The basic data compiled for the purpose of this report cover the period from January 1, 1929, to January 1, 1968, and includes 286,889 miles of construction of the seven surfa types on rural sections of the State prima or Federal-aid primary highway system. T participating States, illustrated in figure reported their construction and retireme activity for the period January 1, 1929, January 1, 1968, the cutoff date. The bas summaries submitted to the Federal Highw Administration consisted of the following:

- Miles constructed each year and mil remaining January 1 of each year for ea surface type.
- Miles retired by method and replaceme type by 5-year periods.
- Miles retired by method of retirement 5-year periods.
- Miles transferred off the system.

Service life analyses were made for t following seven major surface types and f high and intermediate surface types. (Detail descriptions are listed under *Definitions* at e of article.)

- Bituminous surface treated (F).
- Mixed bituminous (G–1) (combined thic ness of surface and base less than 7 inch and/or low load-bearing capacity).
- Mixed bituminous (G–2) (combined thic ness of surface and base 7 inches or m and/or a high load-bearing capacity with without rigid base).
- Bituminous penetration (H–1) (combin thickness of surface and base less than 7 inches and/or low load-bearing capacity).

Table 1. — Bituminous surface treated (F) mileage constructed each year and mileage remaining in service January 1 of each yea

[Compiled from data submitted by 18 States]

Construction		Mileage remaining in service January 1, each year														
Year	Miles	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
1929	598.8	19.4	12.2	9.0	8.2	8.0	8.0	7.9	7.9	7.9	7.9	7.9	5.4	5.4	5.4	5.4
1930	1,220.9	129.4	94.1	84.9	82.7	78.8	78.4	77.1	68.3	67.7	64.6	64.6	64.6	54.0	52.2	52.2
1931	1,370.0	259.3	228.6	186.4	171.7	162.8	156.0	147.6	146.4	145.8	126.4	120.9	110.0	95.1	90.0	83.0
1932	1,456.5	232.4	221.8	185.9	165.3	154.2	147.3	139.5	136.5	94.7	94.1	94.1	94.1	93.7	93.6	93.6
1933	1,811.8	431.4	380.2	335.7	265.0	228.7	221.0	192.6	172.9	167.6	120.6	108.8	108.1	106.6	102.6	102.6
1934	1,748.4	445.0	424.7	344.0	315.4	289.8	284.2	276.6	273.0	247.0	223.9	196.1	182.9	174.9	174.9	169.4
1935	1,504.2	383.3	355.8	317.6	292.1	240.8	231.8	206.8	190.5	160.3	146.7	146.7	135.8	133.4	116.9	112.5
1936	2,322.4	734.8	639.6	573.3	548.8	507.5	456.0	432.2	417.6	388.2	377.7	358.2	342.4	311.7	300.1	283.3
1937	1,791.0	660.4	643.3	608.0	562.6	475.4	431.2	400.3	375.4	345.4	340.7	322.4	316.5	300.2	292.6	288.0
1938	3,480.8	1,330.2	1,300.3	1,202.9	1,158.2	1,070.9	1,006.8	912.3	865.9	809.4	787.5	728.8	676.5	663.3	632.2	616.0
1939	3,007.9	1,314.3	1,223.2	1,142.8	1,066.2	996.7	902.2	845.2	808.8	760.7	695.1	665.0	618.0	572.2	549.8	508.0
1940	3,508.5	1,876.3	1,749.1	1,661.9	1,568.0	1,528.2	1,453.8	1,304.5	1,288.3	1,265.4	1,203.1	1,161.5	1,113.0	1,074.5	1,032.6	1,005.6
1941	2,753.7	1,669.3	1,588.2	1,512.1	1,426.6	1,332.6	1,212.6	1,128.2	1,100.4	1,080.0	1,043.5	985.7	927.4	915.3	898.8	864.4
1942	2,330.9	1,532.7	1,424.2	1,324.8	1,230.3	1,139.1	1,090.7	1,019.9	983.5	931.6	870.4	828.8	807.6	779.6	753.5	728.4
1943	1,631.8	1,034.2	968.1	885.1	864.2	785.6	744.6	703.7	682.4	662.7	634.7	603.5	567.8	526.6	507.3	481.9
1944	1,377.0	871.5	791.2	753.7	696.1	643.1	607.4	586.0	539.7	533.1	512.6	497.4	448.9	411.9	399.5	379.8
1945	1,205.8	756.8	703.2	672.5	622.3	578.3	519.5	442.5	413.2	388.2	363.7	350.9	331.0	322.6	315.1	302.7
1946	1,902.3	1,385.1	1,334.0	1,194.9	1,105.9	1,059.8	1,001.8	928.2	885.6	836.9	815.1	803.5	774.7	736.9	702.9	678.3
1947	2,247.1	1,746.1	1,658.8	1,576.3	1,503.7	1,431.5	1,382.0	1,306.4	1,283.2	1,256.2	1,220.4	1,172.5	1,135.1	1,087.6	1,059.7	1,040.0
1948	2,606.7	2,117.2	1,983.4	1,841.7	1,711.3	1,587.5	1,472.8	1,357.3	1,239.6	1,140.5	1,092.8	1,052.4	1,014.9	974.0	937.4	852.9
1949	2,430.8	2,120.3	2,012.1	1,961.5	1,860.7	1,731.2	1,660.5	1,573.9	1,504.5	1,451.0	1,435.0	1,397.0	1,358.2	1,277.4	1,234.2	1,211.0
1950	2,349.4	2,160.5	2,046.3	1,962.7	1,819.8	1,690.5	1,614.4	1,463.4	1,438.9	1,342.0	1,264.0	1,237.7	1,200.5	1,160.6	1,140.3	1,100.9
1951	2,119.8	1,963.0	1,845.2	1,759.5	1,651.4	1,592.3	1,490.7	1,371.3	1,274.4	1,186.1	1,084.6	1,054.2	1,005.7	953.9	909.9	890.5
1952	2,169.0	2,096.8	1,965.2	1,887.5	1,746.7	1,665.3	1,539.8	1,430.5	1,376.5	1,329.4	1,285.0	1,222.5	1,147.3	1,097.1	1,079.7	1,045.9
1953	2,179.0	2,170.7	2,066.3	1,967.1	1,873.2	1,856.5	1,831.8	1,769.8	1,631.7	1,564.0	1,527.7	1,492.7	1,408.5	1,356.0	1,272.5	1,198.3
1954	2,009.5	--------	1,985.6	1,832.6	1,740.0	1,662.4	1,586.5	1,493.5	1,466.2	1,424.6	1,361.0	1,294.0	1,213.6	1,177.7	1,124.9	1,092.7
1955	1,972.6	--------	--------	1,947.3	1,818.1	1,760.8	1,677.1	1,582.3	1,550.5	1,502.0	1,413.8	1,315.0	1,194.1	1,128.8	1,102.0	1,043.1
1956	1,938.5	--------	--------	--------	1,912.1	1,809.1	1,725.2	1,589.7	1,517.6	1,412.2	1,350.3	1,237.1	1,183.2	1,124.9	1,028.5	994.0
1957	1,512.8	--------	--------	--------	--------	1,480.9	1,367.6	1,300.0	1,246.1	1,169.9	1,121.5	1,058.8	982.3	947.9	917.7	882.0
1958	1,521.6	--------	--------	--------	--------	--------	1,510.6	1,428.1	1,376.5	1,334.9	1,291.3	1,237.6	1,187.7	1,149.8	1,120.1	1,070.3
1959	1,475.0	--------	--------	--------	--------	--------	--------	1,431.0	1,291.0	1,236.8	1,154.9	1,086.5	1,048.0	1,016.8	950.7	916.8
1960	1,046.0	--------	--------	--------	--------	--------	--------	--------	1,032.3	988.8	938.7	885.0	853.8	806.8	785.0	741.7
1961	1,077.7	--------	--------	--------	--------	--------	--------	--------	--------	1,068.7	1,040.1	1,006.3	933.8	858.8	824.1	752.9
1962	1,078.5	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,054.4	984.0	951.6	889.8	851.2	821.3
1963	1,143.7	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,127.6	1,075.9	1,011.3	943.3	865.6
1964	1,170.5	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,123.3	1,015.7	859.9	792.7
1965	885.5	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	830.2	709.6	657.6
1966	635.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	628.0	573.0
1967	711.5	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	709.9
Total	69,302.9	29,440.4	29,644.7	29,731.7	29,786.6	29,548.3	29,412.3	28,848.3	28,585.3	28,299.7	28,063.8	27,905.7	27,642.2	27,143.0	26,498.7	26,008.2

Table 2.—Mixed bituminous (G-1) mileage constructed each year and mileage remaining in service January 1 of each year

[Compiled from data submitted by 16 States]

Construction		Mileage remaining in service January 1, each year														
Year	Miles	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
1929	895.8	25.4	16.7	16.7	16.7	12.2	11.8	11.7	11.7	11.7	11.7	11.1	11.1	11.1	11.1	11.1
1930	1,708.3	126.8	108.7	90.9	86.2	76.1	69.4	64.8	64.3	59.1	59.1	48.6	30.5	29.8	29.8	29.8
1931	3,242.4	499.7	422.8	301.9	262.0	244.2	221.8	180.4	146.1	116.8	111.3	109.5	82.5	65.3	52.1	51.4
1932	3,352.5	554.4	505.9	447.2	371.5	310.0	267.9	209.6	203.3	201.3	187.5	176.1	165.6	157.5	145.9	138.8
1933	1,535.0	194.6	160.7	138.5	128.1	107.2	49.5	49.5	49.4	49.4	45.1	45.1	43.9	41.5	39.3	39.3
1934	2,794.7	401.0	368.2	345.0	321.2	293.2	256.5	250.9	214.0	198.8	183.9	171.3	169.3	165.0	160.2	160.2
1935	2,529.4	287.8	257.1	226.6	213.1	205.2	193.3	180.1	157.3	147.8	119.7	113.5	101.1	84.0	78.1	67.9
1936	2,918.6	679.1	598.1	526.2	446.7	412.9	403.1	384.5	365.4	356.5	349.5	334.0	317.6	303.7	293.7	289.8
1937	4,320.1	677.4	600.6	553.5	501.7	472.9	441.6	411.6	399.5	372.9	352.2	334.6	331.5	331.5	327.6	305.9
1938	3,286.7	601.0	552.7	487.0	446.3	414.6	406.8	405.2	376.5	353.5	343.1	325.3	300.2	296.5	290.9	289.8
1939	2,102.7	357.8	334.1	303.0	255.6	247.3	236.2	224.4	213.7	207.0	200.4	189.9	184.4	183.2	176.0	176.0
1940	862.2	250.8	214.9	152.7	129.4	117.3	103.4	98.7	76.8	73.1	63.0	62.5	56.8	54.4	49.9	49.9
1941	1,213.1	354.2	322.5	284.2	262.1	207.7	178.8	169.6	167.2	149.3	148.1	128.9	118.7	101.2	87.7	64.5
1942	594.2	228.5	214.1	186.7	156.7	141.2	125.2	125.0	89.1	73.5	64.5	58.0	48.1	43.6	43.6	43.6
1943	339.1	68.5	53.5	46.2	46.2	38.8	38.8	37.8	34.1	34.1	29.7	29.7	28.5	25.9	23.2	23.2
1944	899.5	221.0	156.6	121.4	89.2	60.5	59.5	51.6	43.6	43.6	39.2	39.2	38.1	36.2	36.2	30.4
1945	1,064.1	427.6	317.8	285.8	217.2	172.0	165.2	159.3	126.1	115.8	99.4	87.2	79.3	70.2	50.5	49.3
1946	1,434.2	804.8	707.2	624.9	554.1	526.7	448.8	395.9	362.3	321.9	293.7	271.9	264.3	257.5	226.0	208.5
1947	1,276.0	925.3	867.0	803.7	689.3	589.6	486.3	424.4	374.7	337.0	286.4	258.8	232.2	228.3	214.5	214.5
1948	1,678.7	1,292.3	1,216.3	1,123.1	1,012.1	920.3	843.1	709.6	613.2	553.5	525.0	504.2	480.9	448.1	414.4	391.6
1949	1,266.9	983.3	882.1	798.8	728.6	695.1	634.1	609.2	554.6	478.1	445.8	408.9	394.7	388.1	357.6	339.9
1950	1,073.3	896.4	820.8	754.9	691.6	625.6	575.7	549.4	507.7	469.6	434.4	406.7	394.1	378.4	363.2	326.5
1951	974.0	839.6	803.3	759.6	727.7	685.8	660.1	606.9	554.0	536.5	498.3	475.1	460.8	424.3	406.7	393.8
1952	1,048.9	1,014.9	983.8	957.2	881.5	860.0	836.6	808.6	727.9	706.3	648.4	600.5	557.0	489.9	437.7	400.2
1953	732.8	732.6	698.8	656.5	629.7	620.0	592.8	573.7	558.2	545.1	535.8	487.8	438.1	421.4	358.5	316.0
1954	784.9	--------	784.4	753.6	717.3	685.4	663.2	617.4	596.9	585.4	555.2	524.1	495.5	456.1	434.6	411.4
1955	427.6	--------	--------	423.8	419.2	387.7	386.6	369.3	334.8	311.2	293.7	288.1	282.9	270.4	251.1	244.3
1956	411.5	--------	--------	--------	411.5	410.1	389.8	376.5	365.0	359.0	339.5	330.8	306.4	289.7	273.1	273.1
1957	350.1	--------	--------	--------	--------	347.1	332.3	322.7	315.6	313.0	301.0	300.2	295.2	284.0	278.2	269.6
1958	447.6	--------	--------	--------	--------	--------	447.1	435.0	426.0	425.7	420.1	413.5	402.6	380.6	366.9	363.1
1959	314.4	--------	--------	--------	--------	--------	--------	314.4	314.1	301.7	289.2	288.2	282.2	280.1	273.8	269.0
1960	298.2	--------	--------	--------	--------	--------	--------	--------	298.2	297.8	297.6	287.6	281.5	272.5	272.5	255.0
1961	437.6	--------	--------	--------	--------	--------	--------	--------	--------	437.6	433.4	424.1	408.2	406.6	403.1	396.7
1962	375.4	--------	--------	--------	--------	--------	--------	--------	--------	--------	375.4	368.2	368.2	362.4	352.0	349.9
1963	384.9	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	384.9	383.3	381.2	371.8	337.4
1964	247.7	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	247.7	226.8	224.6	224.4
1965	178.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	178.0	172.0	167.6
1966	229.3	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	229.3	229.3
1967	198.4	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	198.4
Total	48,228.8	13,444.8	12,968.7	12,169.6	11,412.5	10,886.7	10,525.3	10,127.7	9,641.3	9,543.6	9,380.3	9,288.1	9,083.0	8,825.0	8,577.4	8,401.1

Table 3.—Mixed bituminous (G-2) mileage constructed each year and mileage remaining in service January 1 of each year

[Compiled from data submitted by 19 States]

Construction		Mileage remaining in service January 1, each year														
Year	Miles	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
1929	672.4	141.9	134.8	128.2	110.3	102.3	99.2	95.0	84.6	77.2	73.2	59.2	52.2	52.2	27.8	27.8
1930	1,269.9	537.2	493.1	481.5	450.2	432.2	395.5	372.6	362.4	353.8	349.9	322.1	304.6	302.1	279.0	279.0
1931	1,619.0	715.6	676.1	615.2	576.9	544.5	508.2	466.3	458.9	442.5	431.3	428.4	404.5	369.0	355.9	355.5
1932	1,433.2	662.7	572.9	547.6	515.6	487.1	435.2	409.2	408.0	391.5	360.4	330.3	310.2	285.7	245.9	229.8
1933	1,374.6	619.6	577.7	542.8	512.7	461.8	425.0	388.5	357.1	296.8	281.7	252.0	230.5	216.0	208.5	198.8
1934	2,263.8	1,189.8	1,124.4	1,042.8	1,017.7	963.8	914.8	821.8	772.4	739.7	690.3	656.2	625.2	582.0	521.2	499.7
1935	1,534.1	838.1	780.7	757.2	726.5	695.7	658.0	607.2	560.5	525.5	487.2	456.9	413.9	385.2	368.5	353.3
1936	1,711.1	873.3	785.8	722.3	690.3	650.1	624.1	608.4	575.4	531.2	485.6	465.6	434.4	418.8	411.3	386.3
1937	2,035.1	1,101.5	1,031.2	989.9	956.1	922.7	895.2	860.2	838.3	808.4	778.3	766.3	722.6	689.0	662.2	612.8
1938	2,055.1	1,228.2	1,186.5	1,137.6	1,034.2	996.3	946.4	898.6	838.0	785.2	747.8	734.7	685.8	656.4	613.7	578.4
1939	1,313.4	702.4	632.1	615.1	561.8	532.2	491.7	461.0	427.7	398.6	382.1	375.2	337.0	327.8	320.8	313.3
1940	1,231.7	764.5	749.3	737.7	714.8	709.0	676.7	642.1	613.4	585.1	573.5	528.4	491.1	460.0	441.8	415.0
1941	1,399.5	650.3	615.5	579.7	497.2	474.0	449.9	437.3	416.6	381.3	328.1	313.4	298.7	284.8	284.2	262.5
1942	1,111.4	462.6	450.6	420.9	395.8	369.9	358.8	356.4	331.6	279.5	278.2	275.1	272.6	263.6	236.4	217.1
1943	1,041.2	453.1	429.4	404.8	374.7	348.6	323.0	285.3	238.5	208.9	173.1	162.2	139.9	138.4	133.6	133.6
1944	1,138.6	472.6	421.8	387.3	343.3	327.2	300.1	267.0	249.3	234.3	229.8	191.2	163.0	153.1	140.0	130.1
1945	1,011.6	507.5	474.8	420.4	377.9	330.3	314.2	288.5	267.3	263.9	238.2	229.1	199.5	193.0	175.0	174.2
1946	1,569.6	1,050.2	962.0	897.0	792.7	737.4	660.3	616.7	560.8	524.3	505.4	466.8	438.9	411.9	386.5	361.5
1947	2,174.8	1,684.9	1,565.2	1,504.1	1,421.6	1,386.9	1,278.4	1,202.1	1,141.4	1,080.5	1,054.1	995.3	944.4	847.5	794.3	736.3
1948	2,378.1	1,876.7	1,754.9	1,635.7	1,527.8	1,443.3	1,351.0	1,306.0	1,234.8	1,123.3	1,055.0	1,000.7	944.3	892.3	827.0	785.4
1949	1,894.3	1,570.5	1,504.2	1,394.6	1,280.5	1,186.1	1,123.6	1,095.1	1,031.3	994.9	964.2	928.6	885.0	807.1	772.1	754.4
1950	2,254.5	2,035.3	1,973.5	1,873.8	1,758.4	1,662.9	1,599.5	1,531.5	1,477.3	1,438.9	1,398.9	1,341.0	1,311.3	1,225.3	1,115.7	1,077.0
1951	2,453.4	2,287.8	2,231.5	2,122.6	2,049.0	1,945.9	1,827.5	1,775.1	1,722.5	1,663.3	1,598.9	1,538.2	1,465.8	1,390.6	1,318.6	1,229.6
1952	3,184.8	3,116.4	3,022.2	2,937.7	2,830.3	2,679.0	2,555.1	2,407.2	2,283.4	2,184.5	2,113.5	2,024.9	1,914.6	1,810.4	1,676.7	1,568.2
1953	2,558.0	2,539.8	2,485.3	2,433.3	2,355.1	2,305.2	2,216.0	2,157.2	2,135.3	2,059.3	1,998.7	1,944.2	1,809.4	1,708.4	1,581.8	1,486.2
1954	2,753.6	--------	2,727.5	2,654.4	2,576.4	2,515.5	2,437.8	2,384.7	2,334.8	2,284.7	2,193.7	2,113.3	2,026.8	1,987.2	1,900.5	1,871.1
1955	2,741.3	--------	--------	2,725.0	2,621.8	2,549.3	2,470.8	2,406.3	2,357.3	2,320.5	2,271.1	2,200.5	2,072.1	2,002.4	1,937.5	1,863.2
1956	2,798.1	--------	--------	--------	2,781.3	2,734.6	2,691.0	2,645.7	2,627.9	2,547.4	2,469.5	2,421.7	2,357.7	2,306.4	2,252.8	2,227.6
1957	2,206.4	--------	--------	--------	--------	2,192.5	2,114.7	2,053.1	2,006.3	1,963.0	1,892.0	1,829.8	1,756.5	1,708.1	1,649.2	1,625.0
1958	2,205.7	--------	--------	--------	--------	--------	2,192.5	2,117.1	2,053.0	2,018.9	1,985.2	1,948.3	1,859.4	1,758.3	1,712.2	1,659.7
1959	2,138.6	--------	--------	--------	--------	--------	--------	2,131.7	2,079.3	2,024.6	1,956.1	1,903.3	1,787.5	1,745.2	1,677.1	1,588.9
1960	1,858.6	--------	--------	--------	--------	--------	--------	--------	1,857.9	1,836.3	1,803.7	1,767.5	1,711.5	1,663.7	1,639.6	1,610.2
1961	1,903.7	--------	--------	--------	--------	--------	--------	--------	--------	1,898.5	1,891.9	1,819.2	1,798.8	1,769.2	1,738.3	1,665.3
1962	1,833.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,822.6	1,785.4	1,748.4	1,699.1	1,647.1	1,581.7
1963	1,924.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,920.9	1,912.6	1,888.9	1,869.6	1,815.0
1964	2,163.4	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	2,161.8	2,146.1	2,109.0	2,095.2
1965	1,469.3	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,469.1	1,425.4	1,402.4
1966	1,472.8	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,469.9	1,429.9
1967	1,256.9	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	1,256.9
Total	71,408.6	28,082.5	29,363.0	30,709.2	31,850.9	32,686.3	33,334.2	34,094.9	34,703.3	35,266.3	35,863.2	36,495.9	36,992.5	37,014.3	36,926.7	36,857.9

Table 4.—Bituminous penetration (H-1) mileage constructed each year and mileage remaining in service January 1 of each year

[Compiled from data submitted by 7 States]

Construction		Mileage remaining in service January 1, each year														
Year	Miles	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
1929	8.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1930	19.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
1931	16.0	5.3	5.3	.8	.8	.8	.8	.7	.7	.2	.2	.2	.2	.2	.2	.2
1932	39.4	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
1933	63.6	12.0	7.4	.6	.6	.6	.6	.6	.6	.6	.6	.6	.6	.6	.1	.1
1934	40.1	24.5	24.5	20.2	17.7	17.7	17.7	17.7	17.7	17.7	17.7	17.7	17.7	17.7	17.7	17.7
1935	27.1	17.6	10.2	7.4	7.4	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
1936	67.9	4.5	4.5	4.5	4.5	4.5	4.5	4.5	4.5	.0	.0	.0	.0	.0	.0	.0
1937	46.2	16.2	15.3	15.3	15.2	13.2	13.2	7.2	7.2	5.0	5.0	5.0	5.0	5.0	5.0	5.0
1938	20.3	8.4	8.4	8.4	3.6	3.3	3.3	3.3	.1	.1	.1	.1	.1	.1	.1	.1
1939	15.1	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
1940	25.5	14.2	14.1	14.1	13.8	13.8	13.8	13.8	13.8	13.8	12.9	10.9	10.9	10.9	10.9	10.8
1941	27.3	5.3	4.7	4.7	4.7	4.7	4.7	.2	.2	.2	.2	.2	.2	.2	.2	.2
1942	43.1	16.4	13.2	10.5	3.1	3.1	3.1	3.1	3.1	.0	.0	.0	.0	.0	.0	.0
1943	28.9	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3	3.3
1944	17.8	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
1945	48.7	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	10.4	3.9	3.9	3.9	3.9
1946	27.1	8.2	8.2	8.2	7.5	7.5	4.8	4.8	4.7	4.7	4.7	4.7	4.7	4.7	4.7	4.7
1947	9.7	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	2.5	2.5
1948	28.9	22.0	22.0	22.0	22.0	18.1	18.1	18.1	18.1	18.1	18.1	18.1	18.1	18.1	11.8	10.3
1949	36.7	34.8	34.8	21.4	21.4	21.4	21.4	21.4	21.4	21.4	21.4	21.4	21.4	17.9	13.9	12.3
1950	6.1	6.1	6.1	3.3	3.3	3.3	3.3	.4	.4	.4	.4	.4	.4	.4	.4	.4
1951	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4
1952	34.4	33.9	33.9	32.2	32.2	32.2	32.2	32.2	15.1	15.1	14.5	14.5	14.5	6.2	6.2	6.2
1953	8.7	8.7	8.7	8.7	8.7	8.7	8.7	8.7	8.7	8.1	8.1	8.1	8.1	8.1	8.1	8.1
1954	6.3	--------	6.3	5.3	5.3	5.3	5.3	5.3	5.3	4.5	4.2	4.2	4.2	4.2	4.2	.0
1955	4.8	--------	--------	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8	4.8
1956	23.4	--------	--------	--------	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4	23.4
1957	.7	--------	--------	--------	--------	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0	.0
1958	.5	--------	--------	--------	--------	--------	.4	.4	.4	.4	.4	.4	.4	.4	.4	.4
1959	.0	--------	--------	--------	--------	--------	--------	.0	.0	.0	.0	.0	.0	.0	.0	.0
1960	.1	--------	--------	--------	--------	--------	--------	--------	.1	.1	.1	.1	.1	.1	.1	.1
1961	6.7	--------	--------	--------	--------	--------	--------	--------	--------	6.7	6.7	6.7	2.6	2.6	2.6	2.6
1962	8.3	--------	--------	--------	--------	--------	--------	--------	--------	--------	8.3	8.3	8.3	.0	.0	.0
1963	.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	.0	.0	.0	.0	.0
1964	11.5	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	11.5	11.5	11.5	6.6
1965	.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	.0	.0	.0
1966	2.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	2.0	2.0
1967	.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	.0
Total	771.2	255.7	245.2	210.0	217.6	204.0	201.7	188.2	167.9	162.9	169.4	167.4	168.3	148.2	138.4	126.1

Table 5.—Bituminous penetration (H-2) mileage constructed each year and mileage remaining in service January 1 of each year

[Compiled from data submitted by 10 States]

Construction		Mileage remaining in service January 1, each year														
Year	Miles	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
1929	48.5	26.5	26.3	25.9	25.5	25.1	24.4	23.3	22.1	20.6	20.3	20.3	20.3	20.3	20.3	20.3
1930	37.1	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.8	7.5	7.5	7.5	7.5	7.5
1931	489.0	157.1	155.7	113.8	102.4	102.4	98.1	96.4	96.2	76.5	76.5	60.3	54.9	54.9	44.5	44.5
1932	676.1	132.9	117.1	117.1	114.0	111.4	98.4	98.4	81.0	81.0	72.3	64.3	58.0	55.6	49.6	49.6
1933	365.1	107.5	104.5	104.5	101.8	97.4	79.6	69.5	69.5	69.3	63.4	55.4	47.3	47.3	40.0	37.8
1934	600.8	302.5	289.7	276.2	260.7	251.8	216.6	213.5	210.8	192.8	182.4	181.4	156.9	148.9	121.0	108.9
1935	914.2	586.4	545.2	513.9	485.0	437.9	392.3	362.3	335.0	324.1	306.6	280.1	268.6	241.0	231.2	197.1
1936	583.2	390.1	343.8	311.8	276.0	267.8	245.9	237.8	230.1	209.7	196.7	196.3	166.7	155.2	148.6	134.6
1937	414.0	273.8	268.4	241.3	227.1	226.4	189.4	181.8	166.0	162.5	142.5	139.7	123.1	120.5	111.9	111.7
1938	176.5	96.6	95.6	93.1	80.7	80.7	67.0	66.8	61.6	60.3	53.2	50.4	47.6	43.7	43.7	43.7
1939	209.4	97.4	94.1	81.1	81.1	81.1	81.1	73.9	73.9	73.9	62.2	62.2	56.6	48.6	45.7	45.7
1940	200.2	90.8	62.3	48.4	44.6	40.3	40.2	32.9	32.9	32.7	32.6	32.5	32.5	31.1	30.1	30.1
1941	127.2	64.1	56.1	51.8	51.4	38.7	38.7	38.7	38.5	36.1	35.5	33.6	33.6	33.3	33.1	25.4
1942	247.9	137.8	137.8	137.6	114.1	93.3	81.1	79.6	79.5	79.5	69.6	59.6	59.6	59.6	59.6	57.9
1943	241.9	117.5	106.5	84.8	54.2	42.0	41.7	23.7	23.6	23.0	23.0	22.3	21.2	19.6	18.7	18.7
1944	218.5	106.9	94.8	77.3	70.7	70.7	59.4	52.4	52.1	52.1	51.4	40.3	31.4	29.2	29.2	29.2
1945	127.7	44.0	40.4	37.8	25.2	25.2	25.2	22.1	22.1	22.1	21.7	21.6	10.6	10.6	10.6	10.6
1946	132.2	93.0	71.2	69.3	69.3	51.5	47.0	26.9	26.9	26.6	26.1	25.3	21.3	21.2	21.2	21.2
1947	95.1	79.1	79.1	74.6	66.7	60.2	59.9	59.9	59.9	48.2	33.3	33.3	32.6	28.6	26.6	26.2
1948	181.3	126.2	106.9	104.6	98.2	92.9	77.0	59.6	44.3	44.3	42.2	42.2	40.5	38.0	38.0	38.0
1949	88.3	64.1	53.5	44.0	43.2	43.0	35.5	35.5	32.4	32.4	32.3	31.7	31.7	30.4	29.0	29.0
1950	121.1	98.9	98.7	92.6	84.9	81.2	72.2	64.2	63.7	59.1	59.1	53.8	45.8	45.4	44.6	44.4
1951	79.7	79.7	77.0	77.0	74.8	51.6	51.2	51.2	50.8	50.7	50.5	49.6	43.0	41.5	41.4	38.1
1952	91.5	88.2	85.7	85.6	78.5	78.4	78.4	77.1	77.1	76.2	70.7	53.6	53.6	53.6	51.6	51.6
1953	26.4	26.4	25.1	19.6	14.3	14.3	14.3	14.2	14.2	14.2	14.2	6.7	6.7	6.5	6.5	6.5
1954	133.4	--------	133.4	123.8	123.8	123.7	121.9	108.7	105.2	95.4	90.3	90.0	75.8	75.3	74.7	74.5
1955	35.8	--------	--------	34.5	34.5	34.5	34.5	33.7	28.9	27.4	26.8	26.8	26.8	26.6	26.6	19.7
1956	96.1	--------	--------	--------	96.0	95.4	95.4	93.7	76.2	46.8	43.0	41.5	41.5	41.5	41.4	40.3
1957	82.1	--------	--------	--------	--------	82.1	77.3	74.2	73.8	73.5	73.1	71.0	71.0	70.7	69.2	69.1
1958	69.0	--------	--------	--------	--------	--------	68.6	67.4	66.6	66.0	64.8	64.7	64.5	61.9	57.5	50.2
1959	59.2	--------	--------	--------	--------	--------	--------	59.2	59.2	59.1	45.3	44.4	42.4	35.0	32.3	25.9
1960	17.2	--------	--------	--------	--------	--------	--------	--------	17.2	17.2	17.1	17.1	17.1	17.1	17.1	16.4
1961	47.9	--------	--------	--------	--------	--------	--------	--------	--------	47.7	47.6	46.6	36.6	28.6	28.6	28.6
1962	48.3	--------	--------	--------	--------	--------	--------	--------	--------	--------	46.3	41.4	41.4	38.3	37.8	37.8
1963	4.9	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	4.9	4.3	4.3	4.3	4.3
1964	34.4	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	34.4	34.4	33.0	27.6
1965	16.3	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	16.3	16.3	16.3
1966	19.5	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	19.5	16.9
1967	28.3	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	28.3
Total	7,185.3	3,395.3	3,276.7	3,049.8	2,906.5	2,808.8	2,620.1	2,506.4	2,399.1	2,308.8	2,200.4	2,072.4	1,927.4	1,842.1	1,762.5	1,684.2

Table 6.—Bituminous concrete (I) mileage constructed each year and mileage remaining in service January 1 of each year

[Compiled from data submitted by 19 States]

Construction		Mileage remaining in service January 1, each year														
Year	Miles	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
1929	183.0	52.1	39.7	36.6	35.1	33.2	33.2	32.8	30.2	27.0	25.4	19.0	13.5	13.5	13.5	13.5
1930	230.2	18.0	8.1	7.9	7.3	7.3	7.3	7.3	7.3	7.3	7.3	7.3	7.3	7.3	7.3	7.3
1931	243.9	83.7	83.7	56.5	43.8	24.5	24.3	6.9	6.7	6.7	4.7	3.7	3.7	3.7	3.7	3.7
1932	277.2	31.0	27.3	26.7	25.7	25.7	24.3	21.0	21.0	13.4	13.0	11.4	11.4	11.4	11.4	11.4
1933	239.9	67.6	55.6	41.2	33.7	30.9	19.2	6.0	6.0	6.0	6.0	6.0	6.0	6.0	5.8	5.8
1934	448.6	260.8	235.9	228.0	214.7	203.1	193.8	172.4	171.2	168.0	162.8	153.4	152.4	143.4	141.3	138.4
1935	255.7	131.4	107.9	95.5	91.2	87.3	84.5	77.1	75.4	75.4	75.4	69.2	65.8	62.2	57.5	57.5
1936	451.3	179.2	168.4	153.6	148.9	137.9	133.0	117.9	116.0	113.5	112.5	110.9	106.7	106.7	99.7	99.7
1937	511.0	295.9	280.2	254.5	231.0	218.8	209.5	208.0	184.1	179.4	178.6	175.9	168.9	163.8	150.1	144.3
1938	581.1	218.0	208.0	186.8	167.8	154.2	146.2	143.6	128.0	127.9	123.4	123.0	116.6	112.2	112.2	102.6
1939	562.2	309.1	284.8	255.0	210.8	196.4	151.0	140.1	131.9	125.0	119.7	114.2	114.2	103.4	103.4	103.4
1940	496.9	310.2	284.3	271.7	253.9	241.6	175.5	165.8	139.7	134.8	114.4	110.0	109.2	103.5	103.5	101.7
1941	595.1	339.4	321.9	319.8	305.3	281.7	275.7	241.6	228.1	226.6	216.5	216.5	216.2	207.2	205.6	203.6
1942	668.0	337.0	293.8	268.7	220.7	212.9	205.1	187.7	164.2	144.8	135.7	131.0	124.3	124.1	105.0	105.0
1943	716.6	467.8	423.2	396.0	325.2	286.1	256.5	226.5	206.2	191.0	190.6	165.3	165.3	162.5	155.3	147.2
1944	1,192.5	786.6	732.6	623.9	550.7	496.4	435.7	384.7	320.3	289.7	283.0	253.6	225.3	223.0	200.3	180.5
1945	780.0	470.0	428.5	412.9	322.2	277.2	240.6	180.8	165.1	143.8	143.2	139.1	120.7	116.6	111.1	91.6
1946	1,053.9	908.0	836.9	774.0	663.8	599.0	547.5	497.3	442.0	378.7	352.1	305.8	280.8	270.0	263.0	255.6
1947	1,266.6	1,150.2	1,124.1	1,037.5	953.6	866.6	753.0	652.3	586.3	476.4	455.8	403.9	361.2	331.5	311.3	282.3
1948	1,492.5	1,340.9	1,300.8	1,219.4	1,108.2	1,033.1	955.9	833.3	763.2	671.4	515.6	432.8	352.3	327.0	276.5	252.8
1949	1,509.3	1,429.1	1,398.7	1,339.2	1,267.7	1,146.3	1,039.8	939.9	827.7	736.0	685.0	619.8	569.2	530.0	459.3	417.2
1950	1,390.4	1,306.4	1,269.0	1,225.2	1,103.8	994.9	875.5	777.9	729.9	664.1	624.9	541.9	487.4	432.3	404.3	371.3
1951	1,734.8	1,685.5	1,604.4	1,581.6	1,480.2	1,411.1	1,295.9	1,150.8	1,034.9	929.9	880.5	762.7	619.3	557.7	528.4	490.5
1952	2,114.5	2,097.1	2,078.7	2,004.6	1,873.5	1,783.7	1,690.5	1,494.2	1,348.9	1,173.8	1,079.6	960.3	857.7	717.8	624.5	585.4
1953	2,145.2	2,142.7	2,115.5	2,095.9	2,056.0	2,018.1	1,962.9	1,738.3	1,594.2	1,458.7	1,310.0	1,235.0	1,146.9	1,054.5	926.8	805.6
1954	2,544.0	--------	2,542.2	2,523.7	2,484.8	2,402.8	2,348.4	2,178.3	2,041.5	1,836.8	1,730.9	1,628.4	1,473.1	1,289.4	1,121.8	936.2
1955	2,732.3	--------	--------	2,731.3	2,684.2	2,650.7	2,527.6	2,375.7	2,218.2	2,010.5	1,862.3	1,722.0	1,589.1	1,365.6	1,245.8	1,123.8
1956	2,897.1	--------	--------	--------	2,887.2	2,881.5	2,827.2	2,732.4	2,634.7	2,486.8	2,368.3	2,197.2	2,030.2	1,807.0	1,656.2	1,534.5
1957	2,808.7	--------	--------	--------	--------	2,794.6	2,763.5	2,685.1	2,609.8	2,488.6	2,373.9	2,205.5	2,019.4	1,872.8	1,743.5	1,606.1
1958	3,157.2	--------	--------	--------	--------	--------	3,154.5	3,114.8	3,025.1	2,962.3	2,847.9	2,739.8	2,559.3	2,368.6	2,127.0	1,930.4
1959	3,161.5	--------	--------	--------	--------	--------	--------	3,147.1	3,132.8	3,072.9	2,987.7	2,948.6	2,830.1	2,612.8	2,457.9	2,273.0
1960	3,095.4	--------	--------	--------	--------	--------	--------	--------	3,088.8	3,072.3	3,043.5	2,993.0	2,925.4	2,695.1	2,518.5	2,400.7
1961	2,818.1	--------	--------	--------	--------	--------	--------	--------	--------	2,817.0	2,789.3	2,758.0	2,742.7	2,607.2	2,476.0	2,337.0
1962	2,467.9	--------	--------	--------	--------	--------	--------	--------	--------	--------	2,452.2	2,440.2	2,421.9	2,371.4	2,266.2	2,174.3
1963	2,541.5	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	2,536.3	2,494.8	2,459.2	2,428.2	2,269.2
1964	2,783.2	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	2,782.8	2,734.4	2,692.7	2,640.9
1965	3,522.9	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	3,520.7	3,501.5	3,453.6
1966	3,944.4	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	3,925.2	3,905.1
1967	4,427.7	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	4,410.2
Total	64,042.3	16,417.7	18,254.2	20,167.7	21,751.0	23,497.6	25,357.6	26,637.6	28,179.4	29,216.5	30,271.7	31,240.7	32,271.1	33,595.5	35,541.3	37,972.9

Table 7.—Portland cement concrete (J) mileage constructed each year and mileage remaining in service January 1 of each year

[Compiled from data submitted by 19 States]

Construction		Mileage remaining in service January 1, each year														
Year	Miles	1954	1955	1956	1957	1958	1959	1960	1961	1962	1963	1964	1965	1966	1967	1968
1929	1,250.0	790.9	743.5	696.7	663.2	607.9	567.1	522.6	492.1	462.0	416.4	410.6	378.1	355.0	334.5	281.1
1930	1,712.6	1,143.1	1,093.1	1,064.4	1,034.1	998.6	957.0	864.6	794.3	719.6	676.6	638.7	607.7	583.3	531.9	168.0
1931	2,372.1	1,581.7	1,512.3	1,452.1	1,374.8	1,266.3	1,145.6	1,036.5	969.9	948.7	881.4	843.5	807.1	753.7	670.4	633.9
1932	2,148.6	1,543.9	1,519.5	1,421.8	1,316.3	1,297.1	1,237.2	1,159.2	1,057.8	1,019.2	965.2	925.6	890.4	832.5	754.0	671.1
1933	1,528.6	1,155.4	1,071.9	1,021.7	939.1	884.1	843.9	806.0	769.9	730.4	709.4	618.9	573.3	523.9	480.1	449.3
1934	850.1	595.9	584.1	554.0	503.5	468.7	434.0	402.3	389.3	372.5	334.7	323.1	297.4	273.9	247.4	219.4
1935	667.8	430.4	417.3	404.1	364.1	350.9	308.4	289.7	266.5	255.7	234.9	209.5	195.4	177.3	171.7	153.8
1936	948.1	738.8	705.0	685.2	651.1	627.9	581.5	546.6	530.5	502.3	465.1	429.7	402.4	363.7	324.9	308.7
1937	1,249.9	1,019.7	971.9	961.3	915.1	911.4	883.5	833.8	803.5	790.1	768.7	753.7	736.2	685.9	649.4	621.6
1938	1,138.6	1,012.0	994.7	941.8	909.2	880.6	878.1	855.5	843.9	789.9	766.8	741.6	712.3	683.9	656.0	627.6
1939	854.7	737.9	700.1	659.5	646.3	621.1	591.7	580.9	553.1	532.7	504.2	468.1	460.3	421.1	411.8	401.5
1940	479.4	372.1	354.1	334.6	325.2	317.0	303.4	292.1	271.7	263.5	245.8	210.4	196.0	182.8	173.6	157.4
1941	480.9	393.4	375.4	350.1	329.0	323.1	320.6	308.4	304.1	291.0	280.3	253.0	249.7	233.3	199.8	188.0
1942	402.5	305.7	289.8	272.6	257.7	250.6	245.4	223.3	211.3	192.6	185.9	184.0	176.9	169.5	160.4	156.7
1943	229.7	160.0	154.3	143.7	132.5	124.0	105.9	91.9	72.8	68.9	67.5	58.4	45.2	41.3	34.5	33.6
1944	157.5	102.5	96.1	96.1	91.2	75.9	47.6	25.7	25.7	25.4	19.4	19.1	18.8	18.1	16.9	15.8
1945	95.8	66.5	66.5	59.7	59.7	54.3	51.4	47.1	46.1	43.1	43.1	43.1	40.0	40.0	35.0	25.3
1946	290.7	288.4	284.3	280.4	279.2	275.2	271.2	259.0	253.2	246.2	237.9	230.7	222.3	197.7	187.9	187.4
1947	397.5	385.9	385.5	383.4	373.1	363.9	356.2	351.1	345.2	335.1	332.2	309.0	291.6	280.6	271.9	267.6
1948	429.3	428.5	427.3	426.3	424.3	417.3	415.9	399.6	392.7	376.0	366.6	361.9	347.7	343.6	338.8	328.1
1949	430.3	429.3	428.0	420.5	419.4	417.5	416.5	412.7	410.6	406.8	405.4	379.2	371.7	349.0	318.7	308.2
1950	309.8	308.7	308.7	308.7	295.5	294.8	286.6	283.9	279.9	273.2	262.0	256.1	241.8	240.6	235.2	219.9
1951	342.4	342.2	341.6	341.6	337.7	335.0	331.1	329.1	322.5	316.1	308.6	299.7	280.9	278.7	249.8	231.2
1952	433.2	429.1	425.2	425.2	413.8	411.0	399.4	396.8	396.1	381.7	371.7	367.5	359.2	343.7	342.4	312.2
1953	334.4	334.4	333.0	333.0	332.5	332.3	330.9	330.6	328.6	327.3	325.9	325.5	322.1	312.2	298.8	297.6
1954	300.9	--------	300.9	298.2	298.1	296.1	294.7	294.1	288.5	285.5	283.4	283.0	282.7	259.3	253.4	250.2
1955	355.6	--------	--------	355.4	354.2	354.2	353.6	352.0	346.6	344.7	341.3	337.2	336.4	326.8	326.6	319.9
1956	395.0	--------	--------	--------	394.9	394.9	386.2	385.2	378.8	377.9	374.4	362.7	357.2	354.5	351.1	337.6
1957	313.4	--------	--------	--------	--------	313.4	306.1	303.4	301.5	297.7	292.2	290.8	286.1	279.5	279.5	273.0
1958	477.2	--------	--------	--------	--------	--------	473.9	469.0	442.8	433.8	433.1	433.1	432.4	420.2	419.4	410.4
1959	698.8	--------	--------	--------	--------	--------	--------	698.8	698.3	686.4	684.1	666.8	665.1	661.1	649.0	633.2
1960	386.6	--------	--------	--------	--------	--------	--------	--------	386.6	380.2	374.6	372.4	369.8	368.7	364.1	355.6
1961	292.6	--------	--------	--------	--------	--------	--------	--------	--------	292.6	291.0	289.0	288.3	285.1	279.8	275.5
1962	481.1	--------	--------	--------	--------	--------	--------	--------	--------	--------	481.1	479.0	479.0	477.8	471.4	457.4
1963	529.0	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	529.0	527.3	521.3	518.0	513.9
1964	601.4	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	601.1	592.7	590.0	589.8
1965	486.6	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	485.5	485.2	485.2
1966	517.2	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	517.2	515.7
1967	580.3	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	--------	580.3
Total	25,950.2	15,096.4	14,884.1	14,692.1	14,434.8	14,265.1	14,124.6	14,151.5	13,974.4	13,768.8	13,730.9	13,703.6	13,849.9	13,717.8	13,600.5	13,562.7

Table 8.—Mileages constructed and mileages and percentages remaining in

Construction-year period	Bituminous surface treated (F)			Mixed bituminous (G-2)			Bituminous penetration (H-2)			Portland cement concrete (J)		
	Constructed	Remaining in service		Constructed	Remaining in service		Constructed	Remaining in service		Constructed	Remaining in service	
	Miles	*Miles*	*Percent*	*Miles*	*Miles*	*Percent*	*Miles*	*Miles*	*Percent*	*Miles*	*Miles*	*Percent*
1929-33	6,458.0	336.8	5.2	6,369.1	1,090.9	17.1	1,615.8	159.7	9.9	9,011.9	2,503.4	27.8
1934-38	10,846.8	1,469.2	13.5	9,599.2	2,430.5	25.3	2,688.7	596.0	22.2	4,854.5	1,931.1	39.8
1939-43	13,232.8	3,588.3	27.1	6,097.2	1,341.5	22.0	1,026.6	177.8	17.3	2,447.2	937.2	38.3
1944-48	9,339.0	3,253.7	34.8	8,272.7	2,187.5	26.4	754.8	125.2	16.6	1,370.8	824.2	60.1
1949-53	11,247.9	5,446.6	48.4	12,345.0	6,115.4	49.5	407.0	169.6	41.7	1,850.1	1,369.1	74.0
1954-58	8,955.0	5,082.1	56.8	12,705.1	9,246.6	72.8	416.4	253.8	61.0	1,842.1	1,591.1	86.4
1959-63	5,820.8	4,098.3	70.4	9,657.9	8,261.1	85.5	177.5	113.0	63.7	2,388.1	2,235.6	93.6
1964-67	3,402.6	2,733.2	80.3	6,362.4	6,184.4	97.2	98.5	89.1	90.5	2,185.5	2,171.0	99.3
Total	69,302.9	26,008.2	37.5	71,408.6	36,857.9	51.6	7,185.3	1,684.2	23.4	25,950.2	13,562.7	52.3

• Bituminous penetration (H-2) (combined thickness of surface and base 7 inches or more and/or a high load-bearing capacity with or without rigid base).

• Bituminous concrete (I) (with or without rigid base).

• Portland cement concrete (J) (with or without bituminous surface less than 1 inch thick).

Tables 1-7 are a composite of the mileage data submitted by 19 States. Tables similar in arrangement were submitted by each State for each surface type. Columns 1 and 2 show the year built and the miles constructed; columns 3 to 17 indicate the mileage of each year's construction that remained in service on January 1 of each year after construction.

With the information shown in these tables,[2] analyses can be made that give distributions of retirements by age, survivor curve, and average service lives. The methods of analyses are those discussed by Winfrey (*6*), which for many years has been a standard reference for this work as well as similar work by private industries and public utilities. The methods are closely related to those used by life insurance actuaries. One product of the analysis is a survivor curve that indicates the percentage of the original construction remaining in service at different ages. Examples of curve types developed are illustrated in figures 2 and 3.

Analysis Procedures

In the 1968 report (*4*), the service lives were calculated by three procedures—two using the original group method and the other using the retirement rate or annual rate method.

For the purpose of calculating the average service lives, all mileages constructed during a given calendar year are considered to have been placed in service on July 1 of that year. Thus, mileages remaining in service are one-half year old on January 1 of the calendar year following the year of construction, and one and one-half years old on January 1 of the second year after construction, and so forth.

In the *original group method*, the probable average life of the miles constructed each year

Table 9.—Miles retired for each surface type and percentage distribution according t method of retirement

[Total for 1967 and prior years]

Surface type	Retired	Method of retirement				
		Resur-faced	Recon-structed	Aban-doned	Trans-ferred	Total
	Miles	*Percent*	*Percent*	*Percent*	*Percent*	*Percent*
Bituminous surface treated (F)	42,977.4	54.6	35.2	2.0	8.2	100.0
Mixed bituminous:						
G-1	40,416.6	63.6	27.5	1.4	7.5	100.0
G-2	30,300.7	58.6	31.8	1.5	8.1	100.0
Bituminous penetration:						
H-1	857.6	63.8	26.8	1.5	7.9	100.0
H-2	7,602.2	47.5	38.4	2.3	11.8	100.0
Bituminous concrete (I)	27,185.8	64.7	20.7	1.2	13.4	100.0
Portland cement concrete (J)	20,955.3	58.6	26.6	1.6	13.2	100.0
Total	170,295.6	59.3	29.5	1.6	9.6	100.0

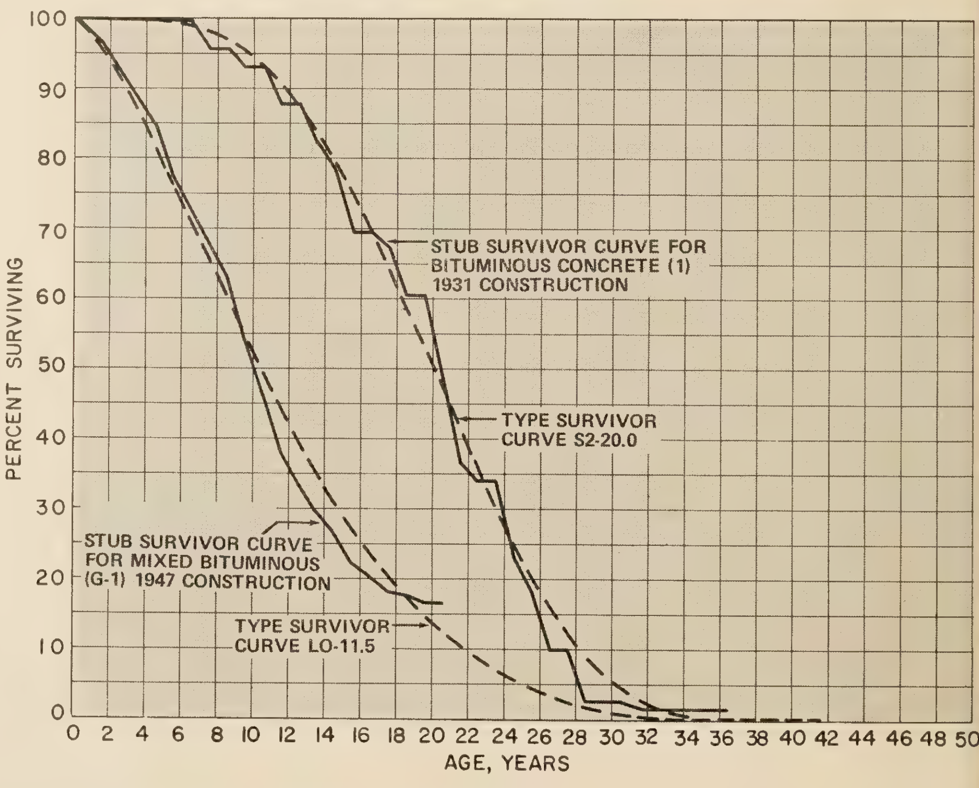
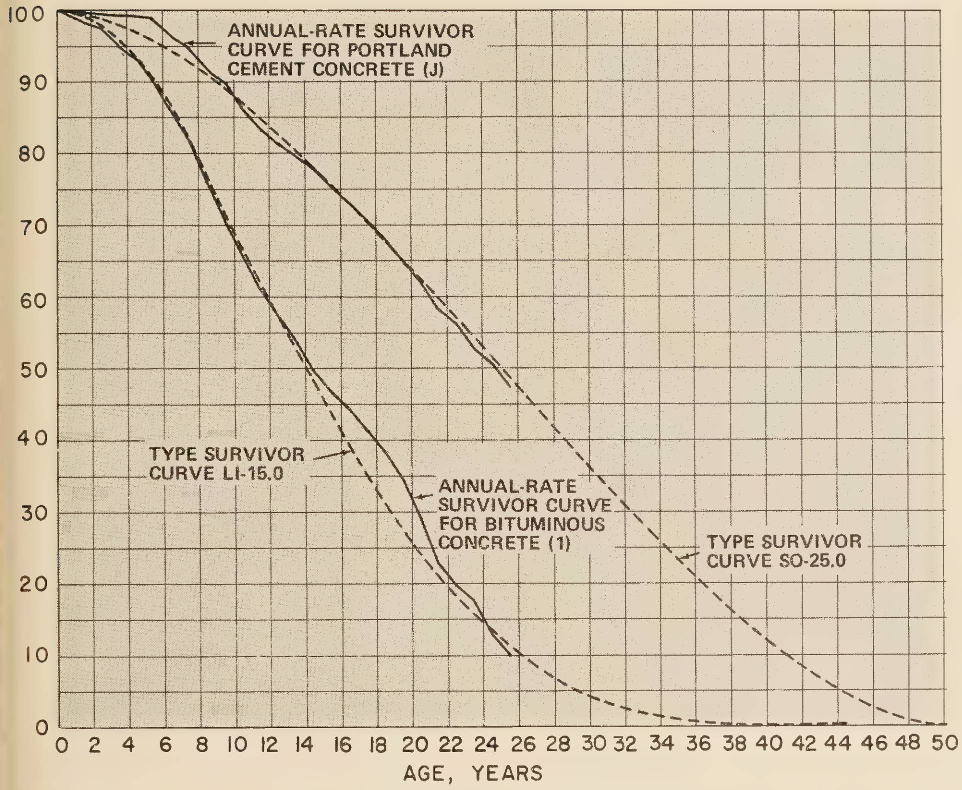


Figure 2.—Survivor curves of mixed bituminous (G-1) and bituminous concrete (I) for 1947 and 1931 construction, calculated using original group method (composite data).

[2] Some tables are a composite of less than 19 States as indicated in the tables.

rvice on January 1, 1968, for 8 construction-year periods by surface type

Mixed bituminous (G-1)			Bituminous penetration (H-1)			Bituminous concrete (I)			Total all surface types		
Constructed	Remaining in service		Constructed	Remaining in service		Constructed	Remaining in service		Constructed	Remaining in service	
Miles	*Miles*	*Percent*	*Miles*	*Miles*	*Percent*	*Miles*	*Miles*	*Percent*	*Miles*	*Miles*	*Percent*
10, 734. 0	270. 4	2. 6	146. 9	0. 3	0. 2	1, 174. 2	41. 7	3. 6	35, 509. 9	4, 403. 2	12. 4
15, 849. 5	1, 113. 6	7. 0	201. 6	22. 8	11. 3	2, 247. 7	542. 5	24. 1	46, 288. 0	8, 105. 7	17. 5
5, 111. 3	357. 2	7. 0	139. 9	14. 3	10. 2	3, 038. 8	660. 9	21. 7	31, 093. 8	7, 077. 2	22. 8
6, 352. 5	894. 3	14. 1	132. 2	21. 4	16. 2	5, 785. 5	1, 062. 8	18. 4	32, 007. 5	8, 369. 1	26. 1
5, 095. 9	1, 776. 4	34. 9	86. 3	27. 4	31. 7	8, 894. 2	2, 670. 0	30. 0	39, 926. 4	17, 574. 5	44. 0
2, 421. 7	1, 561. 5	64. 5	35. 7	28. 6	80. 1	14, 139. 3	7, 131. 0	50. 4	40, 515. 3	24, 894. 7	61. 4
1, 810. 5	1, 608. 0	88. 8	15. 1	2. 7	17. 9	14, 084. 4	11, 454. 2	81. 3	33, 954. 3	27, 772. 9	81. 8
853. 4	819. 7	96. 1	13. 5	8. 6	63. 7	14, 678. 2	14, 409. 8	98. 2	27, 594. 1	26, 415. 8	95. 7
48, 228. 8	8, 401. 1	17. 4	771. 2	126. 1	16. 4	64, 042. 3	37, 972. 9	59. 3	286, 889. 3	124, 613. 1	43. 4



ure 3.—Annual rate and type survivor curves for bituminous concrete (I) and portland cement concrete (J) surfaces retired 1950–54 (composite data).

calculated by using ages and mileages naining in service (tables 1–7). The mile-s remaining in service January 1 of each ır are expressed as percentages of the ori-al construction mileage. These percentages plotted by using the percentages remain-in service as the ordinate and the ages years as the abscissa. The plotted points then connected by a straight line to form ginal survivor curves. These curves are n matched with the Iowa-type curves (*6*) determine the type curve designation and rage life. Examples of the curves are il-rated in figure 2.

The *annual rate method* also used ages and eages remaining in service (tables 1–7) to ulate the probable average life. However, he annual rate method, the rate of retire-nt is calculated for each like-age group of miles in service during the observation period that includes 1 or more years. By this method, calculation of values for a survivor curve requires two sets of data: (1) The number of miles retired during the period of observation and their ages at retirement, and (2) the number of miles in service at the beginning of the observation period and their ages.

From the last two tabulations, the rate of retirement is calculated for each age corresponding to the age of the miles in service. As the rate is usually calculated for a year's retirement, it is generally the annual rate or, in other words, the percentage of the miles of a given age in service at the beginning of a certain year that were retired during the following year. If at the beginning of the observation period the data includes miles installed the previous year and each successive preceding year, annual rates for each age interval from 0–1 to the age of the oldest unit in service would result. By applying these annual rates successively to the percentage surviving at the beginning of each age interval, starting with 100 percent at zero age, an annual rate survivor curve would result. Matching the survivor curve with the Iowa-type curves determines the average life. Figure 3 illustrates curves developed using this method. A more detailed description of the service life analysis and an explanation of the mechanics involved in computing average service lives using these two methods are discussed in the 1968 report (*4*) and literature by Winfrey (*6, 7*).

In the 1968 report in the first procedure, using the original group method, the service life of each pavement type was determined individually State by State. Service lives were then weighted by total miles constructed in each vintage year. In the second procedure, using the same method, the miles constructed and miles remaining in service each January were added to consolidate the 26 States and Puerto Rico data into one table for each pavement type. Average service lives were then calculated directly from these tables.

Table 10.—Percentage retired for various periods by the four methods for all surface types combined

Method of retirement	1930 and prior	1931–35	1936–40	1941–45	1946–50	1951–55	1956–60	1961–65	1966–67	Total 1967 and prior
Resurfaced	77. 9	66. 9	63. 1	70. 1	59. 9	56. 7	52. 0	55. 7	60. 0	59. 3
Reconstructed	18. 3	21. 5	24. 6	22. 5	31. 4	33. 1	35. 7	29. 5	26. 6	29. 5
Abondoned	. 4	2. 1	2. 4	1. 4	1. 7	1. 2	1. 8	1. 3	1. 4	1. 6
Transferred	3. 4	9. 5	9. 9	6. 0	7. 0	9. 0	10. 5	13. 5	12. 0	9. 6
Total	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0	100. 0

Table 11.—Percentage distribution of total retired mileages for each surface type by method of retirement and replacement type for 1967 and prior retirements

Replacement type	Retirement method																			
	Bituminous surface treated (F)					Mixed bituminous (G–1)					Mixed bituminous (G–2)					Bituminous penetration (H–1)				
	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total
None	(1)	(1)	0.3	0.8	1.1			0.2	0.8	1.0			0.3	2.8	3.1				0.3	0.3
C	(1)	0.1	(1)	.1	.2	(1)	1.0	(1)	.4	1.4		1.1	(1)	.2	1.3				.2	.2
D	(1)	.3	.4	1.5	2.2		.1	(1)	.1	.2		.1	.1	.2	.4		(1)	0.2	5.6	5.8
E	0.4	1.9	.1	.6	3.0	1.2	3.9	.2	1.6	6.9	0.1	1.5	.1	.7	2.4		0.3		.4	.7
F	12.8	7.0	.1	1.2	21.1	1.1	3.7	.1	.3	5.2	.6	2.0	.1	.6	3.3	0.5				.5
G–1	6.5	1.8	(1)	.2	8.5	46.7	3.1	.3	.7	50.8	1.3	.5	(1)	.1	1.9	1.8	1.0			2.8
G–2	7.1	3.8	(1)	.9	11.8	10.2	10.1	.4	2.2	22.9	42.0	18.0	.6	2.1	62.7	5.7	.5			6.2
H–1	5.3	12.3	.7	.7	19.0	.2	.3	(1)	(1)	.5	.3	1.8	.2	.1	2.4	35.2	11.4	0.9	.1	47.6
H–2	3.1	.8	(1)	(1)	3.9	.6	(1)			.6	.8	.3	(1)	(1)	1.1	4.9	4.8		.2	9.9
I	16.8	7.4	.2	.8	25.2	1.8	1.9	.1	.5	4.3	10.8	5.7	.2	1.0	17.7	14.7	9.2		.2	24.1
J	.2	1.9	.1	1.7	3.9	.5	4.3	.1	1.1	6.0	.2	2.5	(1)	.9	3.6		.9	.3	.7	1.9
K, L, and M	(1)	.1		(1)	.1	(1)	.2		(1)	.2	(1)	.1			.1					
Total	52.2	37.4	1.9	8.5	100.0	62.3	28.6	1.4	7.7	100.0	56.1	33.6	1.6	8.7	100.0	62.8	28.1	1.4	7.7	100.0
Total miles retired [2]	20,421.5	14,615.7	746.3	3,316.6	39,100.1	23,669.9	10,855.6	540.0	2,934.9	38,000.4	15,862.6	9,485.6	463.2	2,457.0	28,268.4	458.9	205.1	10.3	56.1	730.4

Replacement type	Retirement method																			
	Bituminous penetration (H–2)					Bituminous concrete (I)					Portland cement concrete (J)					Total all surface types				
	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total
None	(1)		0.1	0.6	0.7		(1)	0.2	4.6	4.8		(1)	0.8	3.5	4.3	(1)	(1)	0.3	2.1	2.4
C		(1)	(1)	.3	.3		0.2		.1	.3		0.2	(1)	.5	.7	(1)	0.5	(1)	.3	.8
D	(1)	0.5	.3	4.0	4.8	(1)	(1)	.1	.6	.7		(1)	.1	.8	.9	(1)	.2	.2	.8	1.2
E	0.4	1.0	(1)	.2	1.6		.1	(1)	.1	.2	(1)	.2	(1)	.3	.5	0.4	1.8	.1	.7	3.0
F	.7	1.9	.2	.4	3.2	0.3	.1	(1)	.1	.5	0.1	.5	(1)	.1	.7	3.7	3.1	.1	.5	7.4
G–1	4.6	1.1	(1)	(1)	5.7	.2	.2	(1)	(1)	.4	.1	.2	(1)	.1	.4	13.4	1.4	.1	.3	15.2
G–2	2.9	1.2		(1)	4.1	1.5	.4	(1)	.1	2.0	10.2	3.6	(1)	.4	14.2	13.5	7.2	.2	1.2	22.1
H–1	1.0	17.4	1.1	1.2	20.7	.2	4.0	.4	.4	5.0	.1	.5	.1	.1	.8	1.7	5.1	.3	.3	7.4
H–2	18.5	3.2	(1)	2.3	24.0	.3	.2		.1	.6	(1)	(1)	(1)	.1	.1	2.0	.5	(1)	.1	2.6
I	17.7	12.2	.3	.7	30.9	58.3	14.9	.4	1.7	75.3	43.7	15.8	.3	1.2	61.0	21.7	8.1	.2	1.0	31.0
J	.1	1.1	.2	2.4	3.8	(1)	2.7	.3	7.0	10.0	.7	7.3	.4	7.4	15.8	.3	3.3	.2	2.9	6.7
K, L, and M	.2	(1)			.2	.1	.1	(1)	(1)	.2	.2	.4			.6	.1	.1	(1)	(1)	.2
Total	46.1	39.6	2.2	12.1	100.0	60.9	22.9	1.4	14.8	100.0	55.1	28.7	1.7	14.5	100.0	56.8	31.3	1.7	10.2	100.0
Total miles retired [2]	3,312.2	2,848.0	161.6	871.0	7,192.8	14,592.0	5,490.9	330.7	3,540.9	23,954.5	10,522.1	5,490.0	335.4	2,758.7	19,106.2	88,839.2	48,990.9	2,587.5	15,935.2	156,352.8

[1] Less than 0.05 percent.

[2] Total miles retired shown will not agree with those in table 9, as data from one State is omitted.

The first procedure was used in the 1948 a 1956 studies (3, 8). It was also used in t 1968 study (4) to show a direct comparis between the two procedures using t original group method. According to the 19 report, the two procedures give essentially t same results. However, the second procedu which consolidated the data into a composi table for each pavement type, was consider preferable.

For the study reported here the service liv were calculated first by the second procedu using the original method, and then by t procedure using the retirement or annu rate method.

Mileage in Service

Table 8 summarizes by 5-year groupings t miles constructed and the miles and perce ages remaining in service as of January 1, 19 for each of the seven pavement types. Ba cally, there are no significant changes from t 1968 report—the percentage remaining is st much less for the earlier vintage years than r the later years. Approximately 43 percent the total miles of pavements constructed fr 1929 to 1967 were remaining in service January 1, 1968. Except for the H–2 surf e type, over 50 percent of the miles construc d for the higher type pavements were remain g in service on January 1, 1968. This may b a result of lower type pavements being repla d with higher types.

Methods of Retirement

States record the miles retired according the method by which they were retir. Methods of retirement are classified as follo: (1) Resurfaced, (2) reconstructed, (3) ab doned, and (4) transferred. These classifi tions are general in character and should be so interpreted. Pavement retirements re lt from the following reasons: structural dete ration, functional obsolescence, combinat of both structural deterioration and functi al obsolescence, construction of related high y improvements, and other factors. Stri ly speaking, the transfer of a pavement to ano er highway authority is not a retirement in the sense that the road has rendered its t al service, but it is a retirement from that par c- ular highway system of which it was a p t. (See *Definitions* at end of article for a m re detailed description.)

Tables 9 and 10 show the miles retired by method of retirement, and tables 11 and 12 show the miles retired by method and repl e- ment type. As indicated by the footnot in tables 11 and 12, one State's data were ot used in the analysis of miles retired by e- placement type, but were used in the analsis of those retired by retirement method. Co- sequently, the totals shown for each sur ce type and for all surface types combined re not in complete agreement with tables 9 nd 10.

Table 9 gives the total mileages retired nd the percentage distribution by metho of retirement. The overall relationship show in table 9 has been fairly consistent throug ut the years. More than half of all retirem ts

Table 12.—Percentage distribution of all surface types combined by method of retirement and replacement type by retirement years

Replacement type	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total	Resurfaced	Reconstructed	Abandoned	Transferred	Total
	1930 and prior years					1931–35					1936–40				
None	—	[1]	[1]	0.4	0.4	—	—	0.5	0.4	0.9	—	[1]	0.4	1.3	1.7
C	—	0.8	0.1	1.0	1.9	—	0.7	[1]	1.3	2.0	[1]	1.2	.1	.6	1.9
D	—	[1]	.1	.2	.3	[1]	.6	.8	3.2	4.6	[1]	.5	.2	2.7	3.4
E	0.6	1.0	.1	.4	2.1	0.9	4.4	[1]	1.8	7.1	.3	3.8	.1	.9	5.1
F	20.2	.1	[1]	.2	20.5	6.8	.7	.1	.4	8.0	4.1	4.4	.1	.7	9.3
G-1	44.2	.8	[1]	[1]	45.0	36.0	1.3	[1]	.1	37.4	31.3	2.8	.3	.8	35.2
G-2	4.9	.8	—	—	5.7	8.5	1.8	.2	.5	11.0	13.9	3.6	.3	.8	18.6
H-1	.8	.4	—	[1]	1.2	3.1	.2	[1]	.1	3.4	3.5	1.6	.2	.5	5.8
H-2	3.8	2.2	—	—	6.0	6.5	1.3	[1]	[1]	7.8	3.3	.6	[1]	.1	4.0
I	2.2	.4	—	[1]	2.6	3.6	1.5	[1]	[1]	5.1	7.0	1.3	.1	.1	8.5
J	[1]	10.8	.1	1.6	12.5	.8	8.3	.2	2.6	11.9	.3	4.1	.2	1.6	6.2
K, L, and M	.6	1.2	[1]	—	1.8	[1]	.8	—	[1]	.8	.1	.2	—	[1]	.3
Total	77.3	18.5	0.4	3.8	100.0	66.2	21.6	1.8	10.4	100.0	63.8	24.1	2.0	10.1	100.0
Total miles retired [2]	2,398.6	574.7	13.5	115.8	3,102.6	6,688.4	2,181.2	188.7	1,048.3	10,106.6	12,087.8	4,558.6	377.6	1,919.3	18,943.3
	1941–45					1946–50					1951–55				
None	—	[1]	.1	.6	.7	—	[1]	.1	1.2	1.3	[1]	[1]	.2	1.7	1.9
C	—	.5	[1]	.2	.7	[1]	.3	[1]	[1]	.3	—	.7	[1]	.1	.8
D	—	.2	.1	1.1	1.4	.1	.1	.3	.7	1.2	[1]	.1	.1	.3	.5
E	2.6	2.2	.2	.8	5.8	.3	2.8	.3	1.1	4.5	[1]	1.6	[1]	1.0	2.6
F	2.5	5.6	[1]	.3	8.4	1.9	6.6	.1	.6	9.2	3.3	3.9	.2	.7	8.1
G-1	16.8	1.6	.1	.2	18.7	14.6	2.5	.2	.1	17.4	8.1	1.3	[1]	.3	9.7
G-2	20.1	3.4	.1	.4	24.0	18.1	9.7	.5	1.3	29.6	14.0	10.4	.2	2.2	26.8
H-1	5.5	3.8	.4	.3	10.0	2.1	4.6	.2	.4	7.3	.4	7.1	.4	.3	8.2
H-2	5.3	.3	[1]	.1	5.7	1.9	.2	[1]	.1	2.2	.6	.4	—	.3	1.3
I	17.6	1.8	.1	.3	19.8	17.2	3.5	.1	.3	21.1	25.2	8.8	.2	.6	34.8
J	.1	2.9	.1	1.5	4.6	.2	3.3	.1	2.1	5.7	.4	2.0	.1	2.8	5.3
K, L, and M	[1]	.2	—	—	.2	.1	.1	—	—	.2	—	[1]	—	—	[1]
Total	70.5	22.5	1.2	5.8	100.0	56.5	33.7	1.9	7.9	100.0	52.0	36.3	1.4	10.3	100.0
Total miles retired [2]	11,063.2	3,535.7	178.6	903.3	15,680.8	12,969.3	7,727.6	441.2	1,801.9	22,940.0	13,386.5	9,348.7	366.4	2,659.4	25,761.0
	1956–60					1961–65					1966–67				
None	—	[1]	.3	2.4	2.7	—	[1]	.4	4.9	5.3	[1]	[1]	.9	3.6	4.5
C	—	.4	—	[1]	.4	—	.1	[1]	.2	.3	—	.1	—	.1	.2
D	[1]	.1	[1]	.2	.3	—	[1]	—	[1]	[1]	—	[1]	—	[1]	[1]
E	[1]	.5	—	.3	.8	—	.3	—	.2	.5	—	[1]	—	—	[1]
F	3.3	1.6	.2	.7	5.8	3.5	.8	[1]	.1	4.4	3.0	.6	.1	.1	3.8
G-1	2.9	.6	[1]	.2	3.7	4.4	.6	[1]	.2	5.2	2.6	.6	—	.1	3.3
G-2	10.2	9.7	.2	1.7	21.8	11.1	8.3	.2	1.0	20.6	12.4	4.4	.1	.3	17.2
H-1	.3	7.0	.6	.2	8.1	.3	7.1	.3	.5	8.2	.2	5.3	.1	.2	5.8
H-2	.5	.5	[1]	.1	1.1	.6	.2	—	.1	.9	.4	.1	—	.4	.9
I	29.8	16.1	.4	1.3	47.6	32.2	13.0	.4	2.1	47.7	36.7	16.5	.3	3.8	57.3
J	.2	2.9	.3	4.2	7.6	.2	2.1	.1	4.5	6.9	.2	2.5	.1	4.2	7.0
K, L, and M	.1	[1]	—	—	.1	[1]	[1]	—	—	[1]	—	[1]	—	—	[1]
Total	47.3	39.4	2.0	11.3	100.0	52.3	32.5	1.4	13.8	100.0	55.5	30.1	1.6	12.8	100.0
Total miles retired [2]	12,664.8	10,570.9	532.1	3,021.3	26,789.1	12,231.8	7,596.7	334.2	3,230.7	23,393.4	5,348.8	2,896.8	155.2	1,235.2	9,636.0

Replacement type	Resurfaced	Reconstructed	Abandoned	Transferred	Total
	Total 1967 and prior years				
None	[1]	[1]	.3	2.1	2.4
C	[1]	.5	[1]	.3	.8
D	[1]	.2	.1	.9	1.2
E	.4	1.8	.1	.7	3.0
F	3.7	3.1	.1	.5	7.4
G-1	13.5	1.4	.1	.2	15.2
G-2	13.5	7.2	.2	1.2	22.1
H-1	1.7	5.0	.4	.3	7.4
H-2	2.0	.5	[1]	.1	2.6
I	21.7	8.1	.2	1.0	31.0
J	.2	3.4	.2	2.9	6.7
K, L, and M	.1	.1	[1]	[1]	.2
Total	56.8	31.3	1.7	10.2	100.0
Total miles retired [2]	88,839.2	48,990.9	2,587.5	15,935.2	156,352.8

[1] Less than 0.05 percent.

[2] Total miles retired shown will not agree with those in table 9, as data from one State is omitted.

r each pavement type were resurfaced. Also, pproximately 89 percent of all retirements ere by resurfacing and reconstruction, which consistent with the result of previous studies.

Table 10 shows the percentage distribution miles retired for the four methods by 5-year tervals from 1930 and prior to 1966–67. ist as in the 1968 report, table 10 shows that ice World War II the percentage of rerfaced roads decreased while the percentage reconstructed roads increased. Following e 1956–60 period, however, the trend versed.

This reversal could be attributed to the accelerated Interstate highway program of the 1960's. Undoubtedly, many primary miles that would have been reconstructed were resurfaced to keep them in service temporarily. Possibly other mileages, which normally would have been replaced because of poor location, were also kept in service until they could be rebuilt.

Table 11, for each of the seven pavement types, shows replacement type for each retirement by method of retirement. As shown in the table, except for portland cement concrete (J), most pavement types retired were resurfaced or reconstructed of the same pavement type and, to a lesser degree, with a higher quality pavement. For example, for mixed bituminous (G–2), 42.0 percent was resurfaced with the same G–2-type pavement and 10.8 percent was resurfaced with bituminous concrete (I). Also, for the mixed bituminous (G–2), 18.0 percent was reconstructed with the same G–2 pavement and 5.7 percent reconstructed with bituminous concrete (I). For bituminous concrete (I), 58.3 percent was resurfaced with bituminous concrete. On the other hand, 43.7 percent of portland cement

Table 13.—Average service lives of construction vintages as calculated by consolidating the basic 19 States data before calculating the service life using original-group method

Construction year	Bituminous surface treated (F)	Mixed bituminous (G-1)	Mixed bituminous (G-2)	Bituminous penetration (H-1)	Bituminous penetration (H-2)	Bituminous concrete (I)	Portland cement concrete (J)
1929	[1] 7.1	L_0-8.9	L_0-16.0	S_2-10.0	[1] 25.6	L_2-21.5	S_2-28.5
1930	L_0-10.2	L_0-10.4	L_0-22.5	S_2-11.4	L_0-16.3	R_2-17.6	S_2-30.0
1931	L_0-11.6	L_0-12.8	L_1-22.0	L_0-11.2	L_1-19.0	S_2-19.7	S_2-27.5
1932	L_0-10.4	L_0-11.9	L_1-21.0	R_2-11.8	L_1-16.2	L_1-13.7	S_1-29.0
1933	L_0-11.7	L_0-8.8	L_1-20.5	L_0-9.2	L_1-17.6	L_1-14.7	S_1-28.0
1934	L_0-13.2	L_0-8.9	L_1-22.0	L_1-24.0	L_1-21.0	L_1-24.0	S_1-26.0
1935	L_0-11.4	L_0-7.7	L_0-22.0	R_2-15.2	S_0-22.0	L_1-20.5	S_1-23.0
1936	L_0-13.5	L_0-11.0	L_0-19.0	L_1-10.0	R_1-21.0	L_0-18.0	S_1-26.0
1937	L_0-13.7	L_0-8.0	L_0-20.0	L_1-13.6	R_1-20.0	L_0-21.0	L_1-33.0
1938	L_0-15.0	L_0-9.5	L_0-20.5	L_1-10.4	L_0-19.0	L_0-15.3	S_1-30.5
1939	L_0-14.5	L_0-9.7	L_0-17.0	L_1-3.6	L_0-17.0	L_1-16.0	S_1-26.0
1940	L_0-16.5	L_0-10.0	L_0-22.5	L_1-17.5	L_0-13.0	L_1-17.0	S_1-23.0
1941	L_0-17.0	L_0-10.0	L_0-13.7	L_0-7.5	L_0-14.0	L_0-18.5	S_0-23.0
1942	L_0-17.0	L_0-10.3	L_3-12.9	L_0-8.3	L_0-15.0	L_1-13.5	S_0-20.0
1943	L_0-15.0	L_0-7.0	L_0-11.1	L_0-4.7	L_1-11.0	L_1-14.5	S_0-15.0
1944	L_0-14.0	L_0-7.0	L_0-10.5	S_4-3.2	L_0-11.0	L_1-13.0	S_0-13.0
1945	L_0-12.5	L_0-8.5	L_0-11.0	L_0-6.3	L_0-8.2	L_0-11.5	L_0-16.0
1946	L_0-14.0	L_0-10.5	L_0-13.5	L_0-7.5	L_0-11.0	L_1-14.0	S_2-24.0
1947	L_0-16.0	L_0-11.5	L_0-16.0	L_0-8.8	S_0-14.5	L_2-14.0	S_1-25.0
1948	L_0-13.5	L_0-12.0	L_0-14.0	R_1-14.0	L_0-10.0	L_2-13.0	S_2-24.0
1949	L_0-15.5	L_0-11.5	L_0-15.0	R_1-14.0	L_0-10.0	L_1-14.0	S_2-23.0
1950	L_0-15.0	L_0-11.5	L_0-17.0	R_2-7.7	L_0-12.0	L_1-12.5	S_1-23.0
1951	L_0-13.0	L_0-13.0	L_0-17.0	S_6-20.0	R_1-14.5	L_1-12.5	L_2-22.5
1952	L_0-13.0	L_0-14.5	L_0-16.5	L_2-11.0	R_1-15.0	L_1-12.0	S_0-23.0
1953	L_0-16.0	L_0-14.0	L_0-18.0	S_4-17.0	R_1-8.1	L_1-13.0	S_2-24.0
1954	L_0-15.0	L_0-15.0	L_0-20.0	R_1-10.1	R_1-12.0	L_1-12.5	S_1-22.0
1955	L_0-13.5	L_0-15.0	L_0-20.0	S_6-16.0	R_2-12.0	L_1-11.5	S_1-25.0
1956	L_0-11.0	L_0-16.5	S_0-19.0	S_6-16.0	L_1-8.0	L_1-12.5	S_0-25.0
1957	L_0-12.0	R_1-15.5	L_0-18.0	S_6-0.3	R_1-16.0	L_1-12.0	R_1-25.0
1958	L_0-15.0	S_0-17.0	L_0-18.0	R_2-14.0	R_3-11.5	L_1-12.5	S_0-21.0
1959	L_0-12.0	R_1-18.0	L_0-16.0	(3)	S_0-7.5	L_1-13.5	S_0-25.0
1960	L_0-12.0	L_1-17.0	L_0-23.0	R_2-7.5	S_2-15.0	L_1-13.0	S_0-25.0
1961	L_0-12.0	L_1-17.0	L_0-22.0	S_1-5.0	L_1-7.0	L_1-14.0	S_0-25.0
1962	L_0-12.0	L_1-17.0	L_0-17.0	S_6-3.0	L_0-10.0	L_1-14.0	S_0-25.0
1963	L_0-12.0	L_0-14.0	L_0-17.0	(2)	L_0-10.0	L_1-14.0	S_0-25.0
1964	L_0-12.0	L_0-14.0	L_0-17.0	S_6-4.0	L_0-10.0	L_1-15.0	S_0-25.0
1965	L_0-12.0	L_0-14.0	L_0-17.0	(2)	L_1-12.0	L_1-15.0	S_0-25.0
1966	L_0-12.0	L_0-14.0	L_0-17.0	S_6-5.0	L_0-10.0	L_1-15.0	S_0-25.0
1967	L_0-12.0	L_0-14.0	L_0-17.0	(2)	L_1-12.0	L_1-15.0	S_0-25.0

[1] An acceptable fit was not found among the original 18 type survivor curves. [2] No miles constructed.

concrete (J) retirements were resurfaced with bituminous concrete but only 0.7 percent resurfaced with the same J-type pavement. These relationships are generally consistent with those indicated in the 1968 report (*4*).

The trend in pavement replacement type is indicated in table 12 for each of the four retirement methods. Over the years about 65 percent of the retirements were replaced by the same four pavement types, mixed bituminous (G-1 and G-2), bituminous concrete (I), and portland cement concrete (J). The percentage of retirements replaced with mixed bituminous (G-1 and G-2) was 50.7 percent prior to 1930, but decreased to a low of 20.5 percent by 1966-67. For bituminous concrete (I) and portland cement concrete (J), the percentage decreased from 25.1, prior to 1930, to a low of 14.7 in 1936-40, after which it increased to 64.3 percent in 1966-67. These findings substantiate what has already been stated: because of higher type standards and specifications in the later years, retired pavements were usually replaced with higher quality pavements.

Tables 11 and 12 indicate a replacement type for abandonments and transfers, which, of course is illogical. Abandonments are not replaced, and transfers are to another highway system, usually without change in surface type. The explanation for these illogical entries was that data coding for machine analysis was not adjusted to separate the abandonments and transfers. They were assigned replacement types according to the resurfacing or reconstruction work on the major highway locations that caused the abandonment or transfer.

Trends in Service Life

Two procedures to determine the service life of each of the seven pavement types were used in the study reported here. One procedure employed the original group method or vintage method; the other employed the retirement rate or annual rate method. In both procedures the basic data on miles constructed and miles remaining in service, submitted by the States, were consolidated in one table for each pavement type. These tables were then used to calculate the average service lives.

Procedure 1—vintage, or original group method

Using tables 1-7, the percentage surviving each year following construction for each vintage was calculated and plotted to form survivor curves for each vintage. Survivor curves were then matched with the Iowa type curves to determine the type curve designation and the average service life. Figure 2 illustrates the type survivor curves obtained by this procedure.

Table 14.—Weighted probable average lives for various construction-year periods for each surface type

Construction year period	Average life [1]						
	Bituminous surface treated (F)	Mixed bituminous (G-1)	Mixed bituminous (G-2)	Bituminous penetration (H-1)	Bituminous penetration (H-2)	Bituminous concrete (I)	Portland cement contrete (J)
	Years	*Years*	*Years*	*Years*	*Years*	*Years*	*Years*
1929-33	10.7	11.2	20.9	10.5	17.7	17.7	28.6
1934-38	13.7	9.0	20.7	14.4	21.0	19.5	28.4
1939-43	15.0	9.7	15.6	8.6	14.0	15.8	22.8
1944-48	14.2	10.3	13.6	8.0	10.7	13.2	22.5
1949-53	14.5	12.8	16.8	12.7	12.5	12.6	23.1
1954-58	13.3	15.7	19.1	14.6	11.8	12.2	23.5
1959-63	12.0	16.5	18.9	3.9	8.8	13.7	25.0
1964-67	12.0	14.0	17.0	4.2	10.9	15.0	25.0

[1] Weighted in accordance with constructed mileage and the estimate of average service life. Average life is shown to nearest 0.1 year, but should not be presumed accurate to this extent. Averages would be materially affected by excluding States or by adding others.

Table 15.—Comparison of type survivor curves and average lives presented in 1968 report with those listed in the present report

[Determined by retirement-rate method and by various retirement year bands for each surface type]

Surface type	5-year retirement bands							3-year	10-year retirement bands			8-year
	1930–34	1935–39	1940–44	1945–49	1950–54	1955–59	1960–64	1965–67	1945–54	1950–59	1955–64	1960–67
Bituminous surface treated (F)	L_0– 6.0	L_0– 8.5	L_0–15.0	L_0–14.5	L_0–15.0	L_0–14.5	L_0–17.5	L_0–16.5	L_0–14.5	L_0–15.0	L_0–16.0	L_0–18.0
1968 report	L_0– 7.0	L_0–10.0	L_0–14.0	L_0–15.5	L_0–15.0	L_0–13.5	----------	----------	L_0–15.0	L_0–14.0	----------	----------
Mixed bituminous:												
G-1	L_0– 7.0	L_0– 6.0	L_0–13.0	L_0–10.0	L_0–10.0	L_0–12.5	L_0–17.0	L_1–18.0	L_0–10.5	L_0–11.5	L_0–14.5	L_0–17.5
1968 report	L_0– 9.0	L_0– 8.5	L_0–17.5	L_0–12.5	L_0–11.0	L_0–13.0	----------	----------	L_0–12.0	L_0–12.5	----------	----------
G-2	L_1–12.0	L_0–16.0	L_0–17.0	L_0–15.0	L_0–14.5	L_0–16.5	L_0–20.5	L_0–21.0	L_0–15.0	L_0–16.0	L_0–19.0	L_0–21.0
1968 report	L_1–14.5	L_2–12.0	R_1–21.5	L_1–19.0	L_0–17.5	L_0–17.0	----------	----------	L_0–19.0	L_0–17.5	----------	----------
Bituminous penetration:												
H-1	L_1– 5.0	L_1–11.0	L_0– 8.5	L_0– 5.5	L_2–16.5	L_1–14.5	L_0– 9.5	L_0– 9.0	L_0– 9.0	L_1–16.0	L_0–16.0	L_0– 8.5
1968 report	L_0–12.5	R_1–14.5	R_1–16.0	L_0–13.0	L_2–20.5	S_0–16.5	----------	----------	L_0–16.5	S_0–18.0	----------	----------
H-2	L_2–10.5	L_2–14.0	L_1–14.5	L_0–15.0	L_0–12.5	L_0–12.5	L_0–15.0	L_0–18.5	L_0–15.0	L_0–14.0	L_0–13.5	L_0–16.5
1968 report	S_1–18.0	R_1–21.5	L_1–19.0	R_1–21.0	L_0–18.0	L_0–16.0	----------	----------	S_0–20.0	L_0–17.5	----------	----------
Bituminous concrete (I)	S_0–18.0	S_0–18.0	L_0–16.0	R_1–16.0	L_1–15.0	L_1–13.0	L_0–14.5	L_0–14.5	L_1–16.0	L_0–14.0	L_0–14.0	L_1–15.0
1968 report	S_0–21.0	S_0–20.5	L_1–18.5	S_0–19.0	L_1–17.5	L_1–16.5	----------	----------	L_1–18.5	L_1–16.5	----------	----------
Portland cement concrete (J)	S_1–25.0	S_2–24.0	R_2–28.0	R_2–23.0	S_0–25.0	S_0–23.0	S_0–25.0	S_0–23.5	R_2–24.0	S_0–25.0	S_0–24.5	S_0–25.0
1968 report	S_2–23.5	R_3–26.0	R_4–25.5	R_3–26.5	R_1–25.0	S_0–25.5	----------	----------	R_2–26.5	R_1–25.0	----------	----------

Service lives by individual construction years are summarized in table 13. Because retirements for the years 1959–67 were not sufficient to indicate a reliable trend, the average lives for those years were based on past retirement trends and judgment. Service lives for the years 1959–67 were developed to obtain an insight into the expected service lives for those years. Except for pavements F and H–1, service lives for most pavement types increased.

Table 14 was prepared from data shown in tables 1–7 and 13 by combining individual construction years into 8 construction-year groupings. Averages were then calculated by weighing the estimated average service life for a particular pavement type during a given year with the mileage constructed during that year.

Although the calculated service lives shown in both tables 13 and 14 indicate no marked trend, interesting facts are revealed for the following surface types:

- Bituminous surface treated (F) increased in service life from 1929 to 1943 and decreased in service life from 1944 to 1967.
- Portland cement concrete (J) decreased in service life from 1929 to 1948 and increased in service life from 1949 to 1967.
- Bituminous concrete (I) increased slightly in service life from 1929 to 1938, decreased from 1939 to 1958, and increased slightly again from 1959 to 1967.

The increase in service life for the higher type pavements, in the later years, may result from improved design standards, specifications, and construction practices. Other contributing factors may be the construction in recent years of full-depth asphalt pavements and the practice of placing bituminous overlays on old portland cement concrete pavements. On the other hand, the decrease in service life of the portland cement concrete (J) pavement type from 1929 to 1948 may have resulted from the deteriorating effect of the increases in traffic volume and axle-load applications on roads built under unfavorable conditions, particularly during and after the war. Moreover, the practice of treating old gravel and stone roads with bituminous in the early years may have contributed to the increase in service life from 1929 to 1943 for the bituminous surface treated (F) pavement type.

Procedure 2—retirement rate method

The retirement rate, or annual rate method may be applied to a single year's retirement experience or to a band of one or more years during which the exposure to retirement and the retirements from each vintage are available. This method results in an average service life that is a measure of the retirement activity during the series of observation years. The service life is then a composite experience of all pavements in service during the series of years. This method considers not only current retirements, but also miles remaining in service, and uses them according to both their amount and age. Such calculations result in a true picture of the retirement rate. Figure 3 illustrates the type survivor curves obtained using the annual rate method.

Table 15 shows a comparison of the service lives presented in the 1968 report with those reported here for the same 5- and 10-year bands. Generally, the trends are similar. Although a decrease in service life from the 1950–54 band to the 1955–59 band is still indicated for most pavement types, an increase in service life for the next 5-year band is indicated for all but the H–1 pavement.

In some pavement types there are certain differences between the service lives presented in the 1968 report and those listed in this report for the same 5-year bands, particularly for the H–1 and H–2 pavements. These differences are insignificant, as variations in service lives are not uncommon when the analyses are based upon different State groupings. Unusual construction practices in one State may have considerable effect on the average for a group of States. Although data from 16 States included in the 1968 report are also included in this report, the addition of one State or the exclusion of another from a group of States may also affect the results of service life analysis.

Life Expectancy

The probable life of the mileage in service is equivalent to its age plus its expectancy. Under certain conditions, it is possible for the average age of mileage in service to exceed the average lives shown in table 14.

The average lives shown in table 14 are the average expectancies at the time of construction or at age zero. As the highway system develops and becomes older, the average age of the surfaces increases and the remaining life expectancy decreases. Also, as the system becomes older, mileages of earlier vintages are taken out of service, thus leaving in service those mileages whose lives will exceed the average life of the total original constructions.

Table 16 shows the average age, remaining life expectancy, and total probable life of the mileages in service at 5-year intervals from January 1, 1934, to January 1, 1964, and a 4-year interval to January 1, 1968. In this table the seven surface types have been combined into two major groups, intermediate

Table 16.—Average age, life expectancy, and probable life of mileage in service at 5-year intervals [1]

Date (January 1 of each year)	Intermediate type			High type		
	Age	Expectancy	Probable life	Age	Expectancy	Probable life
	Years	*Years*	*Years*	*Years*	*Years*	*Years*
1934	2.0	11.3	13.3	2.3	22.3	24.6
1939	3.6	12.2	15.8	4.7	20.1	24.8
1944	6.1	11.3	17.4	7.8	16.7	24.5
1949	7.8	10.6	18.4	9.7	13.8	23.5
1954	8.9	10.4	19.3	10.2	12.6	22.8
1959	10.7	9.9	20.6	10.4	12.0	22.4
1964	13.0	8.2	21.2	11.2	11.3	22.5
1968	15.5	6.2	21.7	11.6	10.9	22.5

[1] Based on analyses of data submitted by 19 States.

and high. The intermediate type includes F, G–1, and H–1 pavements; the high type includes G–2, H–2, I, and J pavements.

Table 16 indicates an increasing age and decreasing life expectancy of the miles in service for both major surface types. However, since 1954 the high type seems to have reached a period of stability. This information is graphically illustrated in figure 4. The major points are the increasing age and decreasing remaining life expectancy of the intermediate type pavements from 1934 to 1968 and the leveling off of the increase in age and decrease in remaining life expectancy for the high type pavements from 1954 to 1968.

In recent years, when intermediate type pavements have been retired, usually they have been replaced with high type pavements. Consequently, the intermediate type pavements now in service are older and have a shorter remaining life expectancy than the high type pavements.

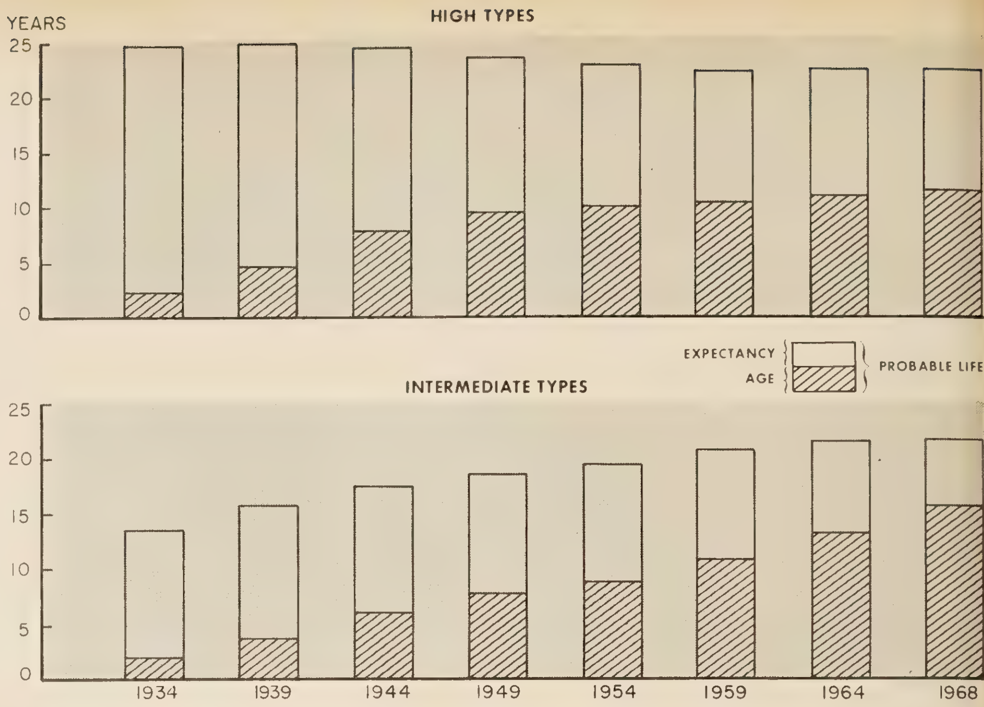


Figure 4.—Trends in average age, expectancy, and probable life, 1934–68.

Future Survivals

Table 17 gives the total mileages in service on January 1, 1968, and the probable amounts from prior vintages that will remain in service for the next 10 and 20 years. This information was compiled by applying the survivor curve with its appropriate service life to the pavements surviving January 1, 1968, and calculating the miles remaining on January 1, 1978, and January 1, 1988. Based on the assumption that the would-be retirements are not replaced, table 17 indicates that approximately 60 percent of the miles in service in 1968 will be retired for various reasons by 1978, and 87 percent will be retired by 1988. Thus emphasizing that highways do wear out.

Table 17.—Mileage in service on January 1, 1968, and estimated mileages and percentages remaining in service on January 1, 1978, and 1988, by surface types, without new construction

Surface type	In service Jan. 1, 1968	Remaining in service on [1]			
		Jan. 1, 1978		Jan. 1, 1988	
	Miles	*Miles*	*Percent*	*Miles*	*Percent*
Bituminous surface treated (F)	26, 008. 2	6, 880. 4	26. 5	1, 280. 3	4. 9
Mixed bituminous (G-1)	8, 401. 1	2, 375. 9	28. 3	593. 7	7. 1
Mixed bituminous (G-2)	36, 857. 9	17, 802. 1	48. 3	7, 022. 7	19. 1
Bituminous penetration (H-1)	126. 1	5. 4	4. 3	0. 7	0. 5
Bituminous penetration (H-2)	1, 684. 2	156. 3	9. 3	12. 9	0. 8
Bituminous concrete (I)	37, 972. 9	15, 444. 4	40. 7	3, 835. 0	10. 1
Portland cement concrete (J)	13, 562. 7	6, 867. 4	50. 6	3, 287. 1	24. 2
Total	124, 613. 1	49, 531. 9	39. 7	16, 032. 4	12. 9

[1] Calculated on the basis that would-be retirements are not replaced. The type curve and service lives from table 13 were used to calculate surviving mileages for 1978 and 1988.

Summary

Highway programs, particularly those of long-range estimates of highway improvements, must consider the average life of existing construction. Even though the exact average life of pavement already retired can be determined, the average life of existing pavement cannot be determined with certainty. Therefore, the only logical approach is to analyze the average life of past construction from which established facts and trends can be used as a basis for arriving at a reasonable estimate of the average life of existing and future construction.

Based on the trend of past construction, service lives of pavements will become or approach a reality only as long as conditions that caused the trends continue to maintain the same influences. Fluctuations in condition of service, standards of design and construction, and management policies can affect the predicted average lives of highway pavements. Therefore, in a given State or for a particular project, the service life may differ appreciably from the service lives resulting from combining the data from the 19 States.

The service lives presented in the following tabulation are based on the road life data submitted by the 19 States participating in the study. They were derived from the service life information shown in table 15 and are the best indications now available of the service-life characteristics of these seven pavement types. When local experience does not indicate otherwise, these service lives may be used.

Surface type	*Type curve*	*Average service life years*
Bituminous surface treated (F).	L_0	16. 0
Mixed bituminous (G–1)	L_0	16. 0
Mixed bituminous (G–2)	L_0	20. 0
Bituminous penetration (H–1).	L_1	12. 0
Bituminous penetration (H–2).	L_0	16. 0
Bituminous concrete (I)	L_1 or S_0	15. 0
Portland cement concrete (J).	S_1	25. 0

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DEFINITIONS [2]

General

Construction and reconstruction. — The construction of a new highway or the reconstruction of a highway or of its component parts to a degree that new, supplementary, or substantially improved traffic service is provided, and significant geometric or structural improvements are effected.

Betterments. — The improvements, adjustments, or additions to a highway that more than restore it to its former good condition and that result in better traffic serviceability without major changes in the original existing construction. Such betterments properly should be termed *capital betterments* as the funds used to pay the cost are considered a capital investment on which the return is increased service to the traveling public, much the same as funds used for new construction or reconstruction, but in the interests of convenience the single word *betterment* is used herein.

Resurfacing. — Laying on top of existing pavement a new surfacing material of 1 inch or more in thickness that in effect provides for a new riding surface but utilizes the former pavement structure in whole or in part as the base or foundation of the new surface. Resurfacing may or may not result in a change in the surface type classsification.

Physical maintenance. — The preservation and upkeep of a highway, including all of its elements, in as nearly as practicable its original as-constructed condition or its subsequently improved conditions.

Traffic services. — The operation of a highway facility, and services incidental thereto, to provide safe, convenient, and economical highway trasportation.

Retirement. — The removal from service of a significant portion of a highway facility through abandonment or reconstruction to a different type.

Abandonment. — A retirement in which the roadway, facility, structure, or other property is discarded in place.

Surface types

Primitive road (A). — An unimproved route (on which there is no public maintenance) usable by 4-wheel vehicles and publicly traveled by small numbers of vehicles.

Unimproved road (B).—A road using the natural surface and maintained to permit bare passability for motor vehicles, but not conforming to the requirements for a graded and drained earth road. The road may have been bladed, and minor improvements may have been made locally.

Graded and drained earth road (C).—A road of natural earth alined and graded to permit reasonably convenient use by motor vehicles and drained by longitudinal and transverse drainage systems (natural or artificial) sufficiently to prevent serious impairment of the road by normal surface water, with or without dust palliative treatment or a continuous course of special borrow material to protect the new roadbed temporarily and to facilitate immediate traffic service.

Soil-surfaced road (D).—A road of natural soil, the surface of which has been improved to provide more adequate traffic service by the addition of (a) a course of mixed soil having A–1 or A–2 characteristics, such as sandclay, soft shale, or topsoil, or (b) an admixture such as bituminous material, portland cement, calcium, chloride, sodium chloride, or fine granular material (sand or similar material).

Gravel or stone road (E).—A road the surface of which consists of gravel, broken stone, slag, chert, caliche, iron ore, shale, chat, disintegrated rock or granite, or other similar fragmental material (coarser than sand) with or without sandclay, bituminous, chemical, or portland cement stabilizing admixture or light penetrations of oil or chemical to serve as a dust palliative.

Bituminous surface-treated road (F).—An earth road, a soil-surfaced road, or a gravel or stone road to which has been added by any process a bituminous surface course, with or without a seal coat, the total compacted thickness of which is less than 1 inch. Seal coats include those known as chip seals, drag seals, plant-mix seals, and rock asphalt seals.

Mixed bituminous road (G).—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material, mixed with bituminous material under partial control as to grading and proportions.

G–1—the base course of which is of other than types J, K, or L and the combined compacted thickness of surface and base is less than 7 inches.

G–2—on a base of types J, K, or L, or on any other type of base where the combined compacted thickness of surface and base is 7 inches or more (or equivalent).

Bituminous penetration road (H).—A road the surface course of which is 1 inch or more in compacted thickness composed of gravel, stone, sand, or similar material bound with bituminous material introduced by downward or upward penetration.

H–1—the base course of which is of other than types J, K, or L, and the combined compacted thickness of surface and base is less than 7 inches.

H–2—on a base of types J, K, or L, or on any other type of base where the combined compacted thickness of surface and base is 7 inches or more (or equivalent).

Bituminous concrete (I), sheet asphalt, or rock asphalt road.—A road on which has been constructed a surface course 1 inch or more in compacted thickness consisting of bituminous concrete or sheet asphalt, prepared in accordance with precise specifications controlling gradation, proportions, and consistency of composition, or of rock asphalt. The surface course may consist of combinations of two or more layers such as a bottom and a top course, or a binder and a wearing course.

Portland cement concrete road (J).—A road consisting of portland cement concrete with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Brick road (K).—A road consisting of paving brick with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Block road (L).—A road consisting of stone block, wood block, asphalt block or other form of block, except paving brick, with or without a bituminous wearing surface less than 1 inch in compacted thickness.

Retirement methods

Resurfacing.—Roads which are resurfaced or used as a base for the replacement type are so classified when the old surface is utilized more or less intact (with the exception of

[2] Definitions are from references (1), (3), (8), and (9).

necessary scarifying, reshaping, or partial reworking of the surface) in the new construction which retires the old surface. Examples of this method are the retirement of a soil-surfaced road by surface treating, or the retirement of a gravel or stone road by utilizing it as a base or foundation for a mixed bituminous road or a bituminous penetration road. For surfaces that are retired by this method, it is obvious that the new or replacement construction must necessarily be along the same alinement and practically the same grade.

Reconstruction.—When surfaces are retired by reconstruction there is little or no salvage of the old surface and base into the new type constructed. This classification includes old surfaces and bases that are torn up and not reused. Usually, for types that are retired by this method, the replacement type is built along the same general alinement (generally within the limits of the existing right-of-way) involving only minor improvements in horizontal curvature. Substantial improvements are usually made with respect to grades, however.

Abandonment.—When the new construction is on new location, the old road is classified as abandoned when it is no longer maintained or kept in service at public expense. The abandoned road may revert to a private road, may be barricaded to public travel, or may be torn up and removed. Sometimes, because of changes in land usage, such as abandonment of factories, and removal or construction of railroad facilities, roads may be abandoned without involving new construction that may be considered as replacing the mileage abandoned.

Transfer.—A retirement by transfer is similar to an abandonment, except that the old road is continued in service after being dropped from the State or Federal-aid system and is maintained by county or other authority responsible for the upkeep of the roads not on the State or Federal-aid system. A transfer is not a retirement in the sense that the road has rendered its total service to the public, but merely that it has rendered its complete service as a primary State or Federal-aid highway. Retirements by transfer are generally the result of functional obsolescence involving alinements and grades that are unsatisfactory for existing traffic conditions. A new road is built on new alinement and improved grades, and the old road remains in service usually because of the necessity of providing for local traffic usage. After the new road is placed in service on the State or Federal-aid highway system, the State will no longer desire to continue responsibility for further upkeep of the old road, and the county or other local authority generally takes over this responsibility. If the road is entirely discontinued from service, it is considered an abandonment.

Highway Research and Development Reports Available from the National Technical Information Service

The following highway research and development reports are available from the National Technical Information Service (formerly the Clearinghouse for Federal Scientific and Technical Information), Sills Building, 5285 Port Royal Road, Springfield, Va. 22151. Paper copies are priced at $3 each and microfiche copies at 95 cents each. To order, send the stock number of each report desired and a check or money order to the National Technical Information Service. Prepayment is required.

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ERIAL EKTACHROME INFRARED PHOTOGRAPHY FOR ERRAIN AND PAVEMENT EVALUATION

For obtaining information on terrain conditions identifiable from egetation, Ektachrome infrared (IR) Aero film has superior resolution s compared with color prints from color negatives. Ektachrome IR istinguishes better the difference in hues of varieties of vegetation, nd ranges of soil types. Natural color film, though it identifies natural olors of features in terrain analysis, does not match the superior ontrast offered by infrared for different vegetation and soil types.

Aerial infrared color photographs also have applications valuable to aintenance personnel for identifying a variety of pavement distress onditions. Comparison by a specially developed technique indicates at Ektachrome IR Aero transparencies yield the most information on avement distress features. For reconnaissance a scale of 500 ft./in. adequate; for detailed studies a scale of 200 ft./in. is suggested.

These conclusions were developed from an extended 1-year experiental study of aerial color photo coverage for a short section of interate highway in Maine.

Color Aerial Photography Research, Maine State Highway Commission report, ate Study No. 58520. NTIS No. PB–183300.

IFLUENCE OF STEAM CURING ON URABILITY OF CONCRETE

Steam curing of precast concrete at temperatures of 130° and 160° , produces optimum 24-hour compressive strengths and also enhances e resistance of concrete to sulfate attack and volume changes. It was so found that cement composition influences durability of steam ıred concrete much more than does variation in steaming procedures. ıe durability determinations were based on the accelerated freezing ıd thawing tests, exposure to sulfate solution, and volume changes om wetting and drying.

An Investigation of the Durability of Steam-Cured Concrete, Phase II, 1969, Virnia Study No. 0461, Virginia Highway Research Council report, NTIS No. 3–184946.

ASE STABILIZATION ON LOW TRAFFIC ROADS

Stabilization of the upper portion of sandy subgrade soils with either asphalt emulsion or a medium-cure cut-back asphalt was found to be satisfactory substitute for gravel base on lightly traveled Minnesota ads, and to be significantly economical in sandy soils where granular aterials are becoming scarce. Performance showed considerable milarity for all test sections with no significant differences for 2- and nch-base sections. Stabilization to a depth of 4 inches topped with a nd seal-coat was the most economical. The State plans further investition of this type construction for different soil types and traffic nditions.

Experimental Bituminous Stabilization Project, (Wadena County) 1968, Minsota Department of Highways Report No. 307. NTIS No. PB–185446.

CONTROL OF SURFACE ROUGHNESS DURING RIGID PAVEMENT CONSTRUCTION

Measuring the surface roughness index of freshly placed plastic concrete pavements requires methods not yet adequately perfected. However, a profilograph record made within a few hours after placement of the fresh concrete facilities prompt corrective action by identifying significant causes of rough finishing. Excessive roughness was found to be distinctly related to five factors: (1) Backing up of rear finishing machine, (2) absence of a surface float in the finishing train, (3) less than three screeds in the finishing train, (4) use of surface crowned cross section, and (5) single-lane paving. These causes, plus other construction phenomena contributing to surface roughness, were found in a 2-year study using the California profilograph on 184 sections of one- and two-lane pavements on 62 New York projects employing seven different form-type finishing machines and three different slipform pavers.

Construction Control of Rigid Pavement Roughness, Interim report, 1969, New York Department of Transportation, Study No. 25–4.

STABILIZING HIGHLY EXPANSIVE SUBGRADE SHALES

An experimental field study in South Dakota indicates that lime treatment of in-situ Pierre shales may be more effective and economical in reducing warp of the pavement surface than replacing highly expansive shale with stable soils. The experiment also yielded favorable results from soil treatment with lime-asphalt and a Products Development Company stabilizer (PDC).

The major criteria in the evaluation were degree of improved stability and related reduction in warping. The evaluation was aided by test of moisture-density, modified field and laboratory CBR, plate load, annual comparative liquid limit, and PI, pH, and $CaCO_3$ tests.

Precise level data and High-Speed Roughometer results were correlated to determine warping effects on ridability. Frost penetration, patching, and crack studies were made to provide relative maintenance costs.

Experimental Stabilization of Pierre Shale—Project F 039-1(1), Lyman County, South Dakota, April 1969, South Dakota Department of Highways report, Study No. 612(64).

TRAFFIC SIGNAL ZONE OF INFLUENCE

The results of a recent study of traffic platooning characteristics will help traffic engineers determine the maximum spacing at which traffic signals can be effectively interconnected under certain conditions. The study measured the degree of platoon dispersion as the vehicles moved down-stream from a signalized intersection. Dispersion can be used not only to determine maximum spacing for interconnecting traffic signals, but also to justify signal installation to interrupt continuous traffic flow (for example, to permit pedestrian crossing).

Zone of Influence of a Traffic Signal, September 1969, Missouri State Highway Commission report, Study No. 69–6.

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Standard Plans for Highway Bridges:
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Standard Specifications for Construction of Roads and Bridg on Federal Highway Projects (1969). $3.50.

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Ultrasonic Testing Inspection for Butt Welds in Highway a Railway Bridges (1968). 40 cents.