HIGHWAY RESEARCH BOARD

Bulletin 233

Flexible Pavement Design Research 1959

National Academy of Sciences—
National Research Council
publication 695

HIGHWAY RESEARCH BOARD

N.R.C. HIGHWAY RESEARCH BOARD

Bulletin 233

Flexible Pavement Design Research 1959

Presented at the

38th ANNUAL MEETING

January 5-9, 1959

1959

Washington, D. C.

$1.00

Department of Design

T. E. Shelburne, Chairman
Director of Research, Virginia Department of Highways

COMMITTEE ON FLEXIBLE PAVEMENT DESIGN

R. E. Livingston, Chairman
Planning and Research Engineer,
Colorado State Highway Department

Stuart Williams, Secretary
Highway Physical Research Engineer,
Bureau of Public Roads

A. C. Benkelman, Flexible Pavement Research Engineer, AASHO Road Test, Ottawa, Illinois
T. L. Bransford, Engineer of Research and Training, Florida State Road Department
W. H. Campen, Manager, Omaha Testing Laboratories
George H. Dent, Benjamin E. Beavin Company, Baltimore
T. V. Fahnestock, Bituminous Engineer, North Carolina State Highway Commission
Charles R. Foster, National Bituminous Concrete Association, Vicksburg, Mississippi
J. E. Gray, Engineering Director, National Crushed Stone Association, Washington, D. C
John M. Griffith, Engineer of Research, The Asphalt Institute, University of Maryland
R. A. Helmer, Research Engineer, Oklahoma Department of Highways
Frank B. Hennion, Office, Chief of Engineers, Department of the Army
Raymond C. Herner, Chief, Airport Division, Technical Development and Evaluation Center, Civil Aeronautics Administration, Indianapolis
Robert Horonjeff, Institute of Transportation and Traffic Engineering, University of California, Berkeley
W. S. Housel, University of Michigan
F. N. Hveem, Materials and Research Engineer, California Division of Highways
J. A. Leadabrand, Manager, Soil-Cement Bureau, Portland Cement Association, Chicago
Allan Lee, State Roads Commission of Maryland, Baltimore
Henry J. Lichtefeld, CAA Office of Airports, Washington, D. C.
Wallace J. Liddle, Highway Design Engineer, Bureau of Public Roads
T. E. McCarty, Materials and Research Engineer, New Mexico State Highway Department, Santa Fe
Chester McDowell, Austin, Texas
Alfred W. Maner, Assistant Testing Engineer, Virginia Department of Highways
Carl E. Minor, Materials and Research Engineer, Washington State Highway Commission
A. O. Neiser, Assistant State Highway Engineer, Kentucky Department of Highways
D. J. Olinger, Aerial Engineer, Wyoming State Highway Department
L. A. Palmer, Engineering Consultant, Department of the Navy
R. L. Peyton, Assistant State Highway Engineer, State Highway Commission of Kansas
Rollin J. Smith, Engineer-Manager, Asphalt Department, Skelly Oil Company, Kansas City
Fred Sternberg, Senior Highway Engineer—Research, Connecticut State Highway Department
J. H. Swanberg, Chief Engineer, Minnesota Department of Highways
Dillard Woodson, The Asphalt Institute, University of Maryland

Contents

Development and Performance of Flexible Pavement on the New Jersey Turnpike

HAROLD W. GOLDBERGER, Assistant Engineer of Maintenance,
New Jersey Turnpike Authority, New Brunswick

● BEFORE DISCUSSING the performance of flexible pavement on the New Jersey Turnpike, it might be well to give a brief review of the Turnpike's history. The New Jersey Turnpike is 118 miles long from Deepwater in Salem County to State Highway 46 in Bergen County. Starting from the southern terminus of the Turnpike, the pavement was four lanes wide for a distance of 91 miles to the Woodbridge Interchange. From this point to the Lincoln Tunnel Interchange leading to New York City, a distance of 22 miles, six lanes were constructed. From the Lincoln Tunnel Interchange to the northern end of the Turnpike, a distance of 5 miles, the pavement was reduced to four lanes. Acceleration and deceleration lanes, each 1,200 ft long, were constructed at 17 interchanges and 10 service areas to assure additional safety in merging the traffic on the Turnpike. Since the Turnpike was opened to traffic late in 1951, 61 miles of roadway have been widened to provide six lanes where there were originally four. One additional interchange has been added at Florence, N. J. to tie in with the Pennsylvania Extension and the Pennsylvania Turnpike. In addition, the Newark Bay-Hudson County Extension was constructed between the Newark Airport Interchange and the Holland Tunnel with three new interchanges and two service areas. Two other service areas were opened to accommodate patrons traveling to and from the Pennsylvania Extension.

Because of an increase in traffic from 18,239,000 in 1952 to approximately 41,000,000 in 1958, traffic problems have developed which require special handling. It is mandatory that traffic is not interrupted for long periods while construction of new facilities by contract is under way or when the Authority Maintenance Department is performing maintenance or repair work on the roadway. Extreme precautions are taken to assure the safety of the traveling public.

PAVEMENT COMMITTEE

There were six engineering firms, in addition to the General Consultants, selected by the Turnpike Authority as Section Engineers to design and supervise construction of the Turnpike. Because of the importance of preparing a satisfactory pavement design, taking into consideration economics, loads and maintenance, a Paving Committee was formed consisting of one representative from the office of each Section Engineer and one from the office of the General Consulting Engineer. The committee was charged with two principal tasks: (1) to develop a typical grading section which would be suitable for whatever type of pavement was selected, and (2) to develop a design for both rigid and flexible pavement meeting the criteria set up by the Authority.

The typical grading section developed by the committee required excavation of all materials to a minimum of 36 in. below the finished pavement grade. The lower portion of the backfill was to consist of layers of pervious frost resistant material. The top of the finished grading section was 8 in. below grade at the center line with a 2 percent cross slope. The purpose of this was to provide temporary drainage until such time that paving was started.

The second task of developing a design for both a rigid and flexible pavement was then undertaken. Many conferences were held with various organizations, officials of neighboring state highway departments, contractors and material producers in an effort to gather data. A compromise axle load of 36,000 lb was recommended to the Authority by the Paving Committee and was approved. The flexible pavement design consisted of 6 in. of well graded aggregates of a frost resistant character placed in two 3-in. layers as subbase. Two 3-in. layers of water bound macadam and one 2-in. layer of penetration macadam comprised the base course. One $2\frac{1}{2}$-in. binder course and one 2-in. surface course made up the $4\frac{1}{2}$ in. of bituminous concrete. Side forms were called for during the installation of the bituminous concrete. The rigid pavement design consisted of 10 in. of portland cement reinforced concrete pavement on a 6-in. pervious subbase.

The Turnpike Authority desired to take alternate bids on the two types of pavement and accordingly plans were prepared. Bids were received and on November 30, 1950, the Chairman of the Authority announced that after consideration of all factors involved, it was decided to use the flexible pavement design for the New Jersey Turnpike. The decision was made in consultation with the Turnpike Engineers, those of the Paving Committee and the General Consultants.

The totals of the bids received were $39,403,000 for asphaltic concrete and $44,823,000 for reinforced portland cement concrete, a difference of $5,400,000 in favor of asphaltic concrete for the 3,900,000 sq yd of pavement involved. One other important factor which the Authority took into consideration in making the decision to use the flexible pavement was that the materials comprising that pavement probably would be least affected by the then current critical materials situation brought about by the Korean conflict.

Shortly after the award of the paving contracts, several suggested changes in the contract specifications were brought to the attention of the Authority in an effort to expedite construction. The suggested changes were:

1. Substitute two courses of penetration macadam totaling 7½ in. in lieu of 6 in. of dry bound and 2 in. of penetration macadam as the base course.
2. Increase the subbase from 6 in. in depth to 6½ in. in order to make up the design thickness.
3. Provide 3.2 gal. of $^{85}/_{100}$ penetration grade asphalt cement per square yard for the penetration macadam which incidentally resulted in a credit of $.10 per square yard to the Authority on the price of the pavement.
4. Eliminate side forms.
5. Substitute three 1½-in. courses of bituminous concrete in lieu of the two courses specified of one 2½-in. and one 2-in. course.

The Authority after due consideration approved the use of asphalt penetration macadam base for that originally specified throughout the Turnpike. With the concurrence of the Section Engineers and the Paving Committee, the Authority approved the substitution of the three courses and the elimination of side forms with the understanding that by eliminating the side forms, the requirements of the specifications regarding line, grade and riding qualities of the pavement would be met.

The penetration macadam proved to be a worthy substitute as the completed base provided the contractors with an excellent haul road with the result that the Turnpike Authority was able to open several sections of the roadway late in 1951 and the remainder early in 1952.

PAVEMENT AS CONSTRUCTED

The Turnpike was divided into seven sections each under the direction of a Section Engineer, and it was, therefore, necessary that proper measures be taken regarding mix control and control of paving methods to insure that the final end product was within the tolerances specified. The bituminous concrete has to be sufficiently stable to withstand high axle loads, high speed traffic and at the same time be flexible enough to resist cracking or deterioration. The surface texture was to be smooth and still maintain a high degree of skid resistance. Seventeen plants meeting Turnpike specifications were used by the various prime paving contractors or their subcontractors to produce either a 4,000- or a 5,000-lb batch. The analysis of the materials for the leveling courses and the wearing course were approximately as shown in Table 1. The average charac-

TABLE 1

AVERAGE SIEVE ANALYSIS OF MATERIALS USED FOR BITUMINOUS CONCRETE PAVEMENT

Sieve Size	Leveling Course Mix (% passing)	Wearing Course Mix (% passing)	New Jersey Turnpike Authority Specifications (%)	
			Leveling Course Mix	Wearing Course Mix
1 in.	100	None	100	None
¾ in.	95	100	-	100
⅜ in.	63	72	50-70	60-80
No. 4	38	44	32-46	40-52
No. 8	32	38	25-36	30-42
No. 30	22	24	16-26	15-25
No. 100	6	6	5-12	6-12
No. 200	3	3	1-5	2-6
Asphalt cement added	4.9	5.6	4.5-5.5	5.0-6.5

TABLE 2

AVERAGE CHARACTERISTICS OF ASPHALT MIXES

Characteristics	Leveling Course	Surface Course
Mixing temperature	295 deg	295 deg
Stability of Marshall Test	1,885	1,580
Density (theoretical maximum)	2.55	2.50
Density (average field)	96%	97%
Flow by Marshall Test	9	11

TABLE 3

RANGE OF TOLERANCES

Passing ⅜-in. and larger sieves	± 5 percent
Passing No. 4 to 30 sieves	± 3 percent
Passing No. 100 sieve	± 2 percent
Passing No. 200 sieve	± 1.0 percent
Amount of asphalt cement added	± 0.2 percent
Temperature of mix on delivery	± 15 F

teristics of the mixes were approximately as given in Table 2. All mixes were required to conform to the tolerances given in Table 3.

The penetration macadam base extended 9 in. beyond the surface course of the bitumenous concrete at the shoulders. Each intermediate course of bituminous concrete was stepped in on either side. Figure 1 indicates a typical cross-section. The Turnpike specifications contained rigid provisions for tolerances such as: (a) maximum tolerance with a 16-ft straight edge placed parallel to the center line to be no greater than $^3/_{16}$-in. variance on the leveling course and $^1/_8$-in. variance on the surface course; and (b) the surface course to be within $^1/_4$ in. measured vertically of the required grade and cross-section as shown on the plans.

SUBGRADE CONDITIONS

The southern portion of the Turnpike provided no abnormal problems of highway construction as the materials for excavation and embankment met the specifications. Two layers of select subgrade material were placed to a maximum depth of 2 ft 4 in. above the common embankment.

The lower 12-in. layer was specified as Grade B select subgrade and was limited to not more then 10 percent passing the 200 mesh sieve, a plasticity index of not more than 6 and a CBR of not less than 15 at 100 percent maximum density, saturated and soaked. The Grade A select subgrade made up the upper layer and was limited to no more than 6 percent passing the 200 mesh sieve, a plasticity index of not more than 3 and a CBR of not less than 20 at 100 percent maximum density, saturated and soaked. The specifications described the subgrade materials as pervious, free draining and frost resistant. These materials are normally classified as A-3 by the PRA classification. Compaction to 95 percent of maximum density was required and obtained in the fills. Grade A select subgrade material placed by the grading contractor, was in sufficient quantity to provide for the placement of pavement subbase by the paving contractors. Excess material was used to build up the median and shoulders.

The northern portion was to pass through the low tidal flats and marshes known as the Jersey Meadows. These marsh areas have soft deposits varying in depth from a few feet to 100 ft and presented a major construction problem for both highway and bridge construction. The Authority approved the sand drain method for constructing the embankments in this area in lieu of other more generally used methods. Approximately 5,000,000 lin ft of sand drains were driven in Jersey Meadows and vicinity.

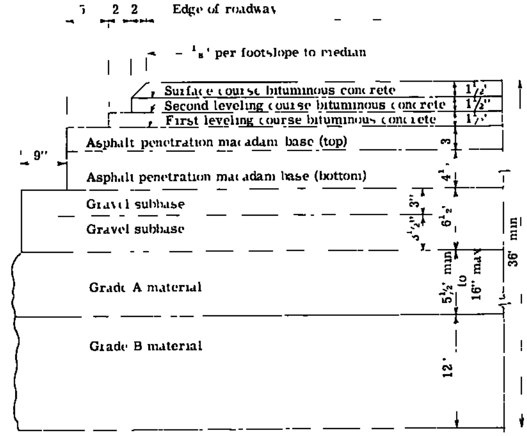


Figure 1. Typical cross-section of New Jersey Turnpike.

EFFECT OF SAND DRAIN AREAS ON FLEXIBLE PAVEMENT

Sand drains for acceleration of the drainage of water from the surrounding mud by forcing it laterally upward and outward when subjected to the pressure of overlying fill were introduced in the northern portion of the Turnpike. This method of consolidation accomplished in months what the older methods of construction would have required years to do. This fast method made it possible to fill, pave and open the section in Jersey Meadows by early 1952. In order to expedite the original stabilization, overloads at these critical areas had been removed when settlement curves had shown a definite tendency to flatten out. Longer periods of overload retention were provided at some locations when possible. It would have been advisable, had time permitted, to extend the overload retention period at other locations to accomplish more complete stabilization. The pavement did not fail in areas of complete stabilization. At other locations some settlement of the pavement created a hazard to traffic, particularly tractor-trailer units. The first signs of settlement in this area, particularly at bridge approaches, were noticed in 1952. Because of this, profile restoration by means of resurfacing was required in 1954, amounting to approximately 48,000 sq yd. In 1956, additional resurfacing amounting to approximately 28,000 sq yd was undertaken. In 1957, approximately 5,000 sq yd were placed and in 1958, the program included the resurfacing of about 4,000 sq yd. This would indicate that settlement, while continuing, is slowing down appreciably. It remains to be seen if this settlement will cease in the near future.

TRAFFIC

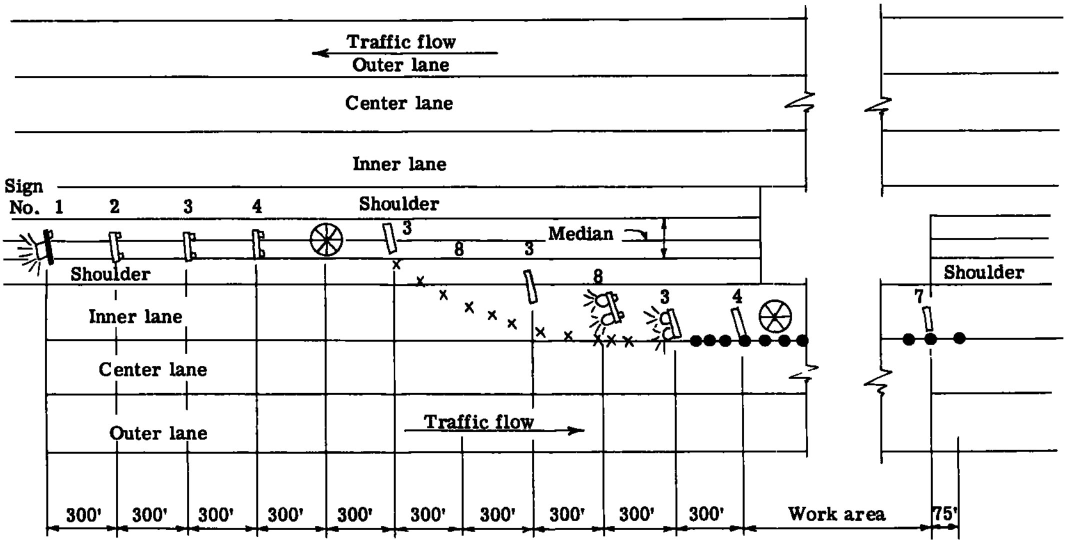
It might be important, before continuing, to give a brief accounting of traffic conditions encountered on the New Jersey Turnpike since it was opened late in 1951. In 1952, the first year that the Turnpike was fully opened to traffic, the road carried 18,239,000 vehicles for a total of 769,154,000 vehicle miles. In 1954, two years later, 24,706,000 vehicles used the Turnpike for a total of 929,324,000 vehicle miles. It became apparent in 1954 that, due to traffic build-up and the pending construction of the Pennsylvania Extension, the section between the Woodbridge Interchange and the North Camden Interchange that had originally been constructed as a four lane highway should be widened to provide for an additional two lanes of traffic. The same traffic build-up was true between the Lincoln Tunnel Interchange and the George Washington Bridge Interchange on Route 46.

The widening construction was undertaken early in 1955 and the third lane was opened to traffic late in December of the same year. The project was not fully completed until July 1956. Traffic in 1955 amounted to 26,000,000 vehicles for a total of 940,000,000 vehicle miles which was below the anticipated volume for this period. In 1956, approximately 32,000,000 vehicles used the Turnpike for a total of over one billion vehicle miles. In 1957, 39,500,000 vehicles were on the Turnpike for a total of 1,206,000,000 vehicle miles. It was anticipated that in 1958, some 41,000,000 vehicles would pass over the New Jersey Turnpike. It might be interesting to note that the average mileage traveled, per vehicle, has been reduced from 40 in 1952 to 30 in 1957 and that the average toll per vehicle decreased from $.90 in 1952 to $.74 in 1957. The total revenue has increased from $16,000,000 in 1952 to $29,000,000 in 1957 with an anticipated total of $31,000,000 in 1958. The average daily number of vehicles using the Turnpike has increased from 49,800 in 1952 to 108,000 in 1957.

WIDENING WORK

As stated earlier, widening work was started early in 1955. Four major contracts were awarded covering every item of work needed to construct the two third lanes and shoulders. The paving specifications from subbase to surface course were written to conform to the original specifications of the Turnpike. One Section Engineer was chosen by the Authority to design and supervise the widening from the Woodbridge Interchange to the North Camden Interchange, a distance of 55 miles. The other Section Engineer was designated by the Authority to design and supervise the widening from the Lincoln Tunnel Interchange to the George Washington Bridge Interchange, a distance of 6 miles.

Rigid regulations governing the protection of traffic were provided in the supplementary specifications. These included the erection of continuous wooden curbs with reflectorized railing, warning and traffic signs, cones, and the use of uniformed flagmen. Contractors' vehicles were prohibited from making U turns and workers were prohibited



Start of lane closing—Sign 1 Work area ahead Size 4' x 4'
Sign 2 Reduce speed Size 4' x 4'
Sign 3 Lane closed Size 4' x 4'
Sign 4 Speed limit 45 Size 4' x 4'
Sign 7 Resume normal speed 4' x 4'
Sign 8 Striped barricades 8' x 8'
⊛ Flagman
X Traffic cones 50-ft intervals
● Traffic cones 75-ft intervals

Figure 2. Traffic protection for one lane closing.

from crossing the roadway. Construction work was permitted only during daylight hours. Turnpike traffic could be limited to one lane for brief periods provided written permission of the Section Engineer was obtained. In each contract one foreman was called for to supervise and instruct watchmen, guards and workers in safety measures. It was also the foreman's responsibility to maintain and install all protective devices. A two-way radio for liaison with the Turnpike police was called for in the specifications and was to be installed in the Traffic Protection Foreman's vehicle. This provision of the contract proved to be instrumental in preventing potentially dangerous traffic tie-ups. Figure 2 shows the location of cones, signs and flagmen required when a lane was closed to traffic to permit construction. Adequate traffic safety control has been a major consideration of the Authority during construction or maintenance work. Approximately $1,000,000 was included in the widening contracts for traffic protection.

Approximately 930,000 sq yd of bituminous concrete surfacing were installed between the Woodbridge Interchange and the North Camden Interchange. Similarly, 77,500 sq yd of bituminous concrete surfacing were installed between the Lincoln Tunnel Interchange and the northern terminus of the Turnpike.

The installation of the subbase material, two courses of penetration and two leveling courses of bituminous concrete, was similar to that of the original construction in 1951. Prior to the installation of the new surface course, the original surface course was cut back an average of 10 in. by means of a mechanical saw capable of making straight and true cuts to the depth of 1½ in. The section of pavement was removed by the contractor by means of a grader blade. No difficulty was experienced in this removal. The overlap of the surface course provided a water stop.

Provisions were made in the contracts to mud-jack concrete bridge approach slabs of existing structures which had settled. Provisions were also made to install some 40,000 tons of bituminous concrete for resurfacing areas of the original two lanes which had settled or had gone out of contour. This resurfacing provided a means to even up the old surface and match the new one. The widening contracts provided for the installation of 3 in. of penetration macadam for inside and outside shoulders. Longitudinal and transverse porous pipe drains were installed in the shoulders as directed by the Engineer. The installation of these drains prevented side slope erosion as sub-surface drainage was concentrated at the drains. Asphalt lip curbs were installed against guardrail posts and down spouts of corrugated metal pipe were placed in the side slopes to drain off storm water from the low spots in the shoulder. Due to the measures which were taken in the widening program a minimum of erosion has taken place.

PENNSYLVANIA EXTENSION

The New Jersey Turnpike Authority and the Pennsylvania Turnpike Commission entered into an agreement in 1954 to construct a high level bridge over the Delaware River from Florence, New Jersey to Edgely, Pennsylvania. The bridge, built as a joint venture of the Authority and the Commission, is maintained by the Pennsylvania Turnpike Commission, however, one-half of the cost is borne by the New Jersey Turnpike Authority.

To connect with the bridge known as the Delaware River Turnpike Bridge, the New Jersey Turnpike Authority constructed a 6-lane highway $5\frac{1}{2}$ miles long, starting from just south of the Bordentown Interchange. Its construction is similar to that of the Turnpike itself in all respects except that the median is 10 ft wider than that on the southern portion of the Turnpike. Due to delays in the grading work in 1955, the major portion of the paving which consisted of approximately 280,000 sq yd of pavement, was installed between the latter part of March 1956 and the middle of May 1956. The completion date set jointly by the Commission-Authority was May 25, 1956 and this date was met. Vehicles leaving the New Jersey Turnpike to travel to Pennsylvania pay their toll at new Interchange No. 6, immediately east of the bridge.

AN EVALUATION OF THE PRESENT CONDITION OF THE TURNPIKE PAVEMENT

As stated early in this paper, it was necessary to provide resurfacing at approaches to bridges, at some locations in the sand drain areas and at several cross drains or similar structures due to settlement. In the past, due to the hard bituminous concrete, the pavement was corrected by removing high spots by the heater planer method and reinstalling bituminous concrete to a new surface grade as established by survey and profile. In order to prevent "feather edging" of resurfacing, all areas to receive resurfacing had the existing bituminous concrete surface removed to such limits and depths as to insure that the resurfacing course would have a minimum thickness of $1\frac{1}{2}$ in. Care was taken that the adjacent or underlying pavement was not damaged by the heater planer. Adjacent shoulders are brought to grade by means of a bituminous concrete plant mix plus a seal coat of cut back asphalt and stone chips. At present, in order to reduce the cost of resurfacing, bituminous concrete is being installed as an overlay in the low areas only, without the use of the heater planer. The ends are "feather edged" with a hot mix of silica sand asphalt. Not only does this result in considerable saving in money expended but inconvenience to motorists is reduced. The completed roadway while not as smooth as visualized in 1950 and constructed in 1951, provides, nevertheless, a comfortable level riding surface with a high degree of skid resistance.

Examination of settled concrete bridge approach slabs revealed that, due to settlement, the end resting on the abutment back wall was higher than the bridge slab. Consequently, it became necessary to either mud jack the slab to its original elevation to meet adjacent resurfacing or, where mud jacking was not practicable, to remove the high concrete by means of power saws and pavement breakers and then cover the entire slab with bituminous concrete. The cost of accomplishing this corrective work is extremely high because of the hand work involved.

In 1956, some longitudinal cracks in all three lanes were observed in fill areas adjacent to bridges and in the sand drain areas. Early in 1958, further longitudinal cracks were found either in the wheel paths or in the center of the middle lane, of fill areas where the new third lane had been added in 1955. It is believed that the cracking in the fill areas is due in part to the weakened condition of the pavement caused by trapped water in the subbase material or in the penetration macadam base. The pavement deflects or rebounds at these locations and the surface is strained in tension or compression or both. The conclusion is that flexural stresses set up by moving traffic lead to crack formations when they exceed the flexural limit of the pavement. The cracks found in the Turnpike pavement vary in depth and width from $^1/_{16}$ in. to $^3/_4$ in.

In the sand drain areas the cracking is due to the uneven settlement of the pavement while the cracking in the center lane probably may be traced to the loss of lateral support during the widening operation.

Early in 1958 small areas of pavement of the new third lane had become deteriorated. The deterioration caused local pot-holing and disintegration of the surface course of the bituminous concrete pavement. Cores were taken and submitted for test and analysis. The results showed that voids due to poor compaction during construction existed in the top 3 in. of the pavement, which could have been the cause of the disintegration particularly as these failures developed during periods of freezing and thawing. Test results in unaffected areas showed relatively low voids and high densities indicating further consolidation of the pavement.

One source of annoyance to the motorist and to the Authority has been the grooving of the pavement, especially in the outside lane. This lane carried the bulk of the truck traffic, causing channelization and grooving in the wheel paths.

WHAT THE AUTHORITY IS DOING TO CORRECT PAVEMENT DEFICIENCIES

Resurfacing of the pavement has been going on since 1954. The cost of resurfacing has increased from $4.38 per sq yd to $7.70 in 1957. The increased cost is due in part to the higher cost of labor and materials, however, the increase in traffic control is the major cause of the higher cost per square yard. Due to traffic volume increase, protection costs have more than doubled in three years because of the necessity of having additional flagmen, large signs, flasher lights and reflectorized traffic cones with heavy rubber bases. The loss of productive hours of labor due to traffic regulations prohibiting work Monday mornings and Friday afternoons, plus the time lost on other working days due to weather or other conditions, influences higher costs. The specification limits for the resurfacing material are shown in Table 4, while the recommended job mix formula is shown in Table 5.

The sealing of longitudinal cracks is the responsibility of the Maintenance Department of the Authority. This operation is accomplished by pouring asphalt emulsion, Grade RS-1 into the crack and then covering the emulsion with a sharp hard manufactured stone sand. Excessive loose material will be whipped off by traffic.

In August 1958, in an effort to improve this operation, the Maintenance Department undertook some test sections with the use of 3 percent solid neoprene added to each 55 gal. of emulsion. The percentage of neoprene was based on the solid asphalt of the emulsion. This mixture was found very satisfactory and this procedure has become a standard method of maintenance because better sealing and bonding is accomplished. In a normal working day the sealing crew is able to cover about one mile of 3-lane pavement. The cost of this operation is variable but a good average for labor costs would be about $75 per day.

TABLE 4

RESURFACING SPECIFICATIONS LIMITS

Square Sieve Size	Total Percent Passing (by weight)
$^5/_8$ in.	100
$^3/_8$ in.	60-85
No. 4	35-55
No. 10	30-45
No. 20	15-33
No. 40	10-26
No. 80	5-15
No. 200	2-8
Asphalt cement (percent total weight of aggregates)	5.5-7.5 ($^{85}/_{100}$ pen.)
Anti-stripping compound	0.5 percent by weight of AC
Temperature, F	275-325

TABLE 5

RECOMMENDED CURVES

Square Sieve Size	Total Percent Passing (by weight)
$^5/_8$ in.	100
$^3/_8$ in.	75
No. 4	45
No. 10	38
No. 20	28
No. 40	20
No. 80	8
No. 200	5
AC	6.0
Anti-stripping compound	0.5 percent by weight of AC
Temperature, F	300

A number of deteriorated areas about 6 ft square were removed and replaced in the spring of 1958 at a cost of about $30 per ton of bituminous concrete. This is a total cost, including traffic protection. Since that time no new areas have appeared.

No attempt has been made as yet to correct the grooving. Since expanded use of silica sand asphalt has been proven satisfactory by various agencies, it is possible that the Authority will experiment with that material in the longitudinal depressions in the near future, however, at the present time no definite plans have been established.

CHANGES MADE SINCE 1951

Since the Turnpike was opened to traffic in 1951, a number of important improvements connected with the pavement have been made, chief among them are: (1) elimination of about 80 percent of the surface treated gravel shoulders, inside and outside, in favor of 3-in. penetration macadam which has substantially reduced maintenance costs; (2) increase in width of outside shoulders in the widened area from 10 ft to 12 ft, thus providing greater safety to the motorist who stops for emergency repairs; and (3) installation of bituminous concrete lip curb along the guardrail posts in order to concentrate storm drainage to drainage structures and as an erosion control measure.

In 1958 one-half of the original remaining 38 miles of gravel inside shoulders was removed and a 3-in. hot plant mix installed. This was done to expedite construction and to lessen the restriction of traffic. Lower maintenance costs will result from this improvement. Plant mix was also installed on inside and outside shoulders, which are used as deceleration and acceleration lanes to and from maintenance areas, in lieu of penetration macadam or gravel. This has reduced potential traffic hazards as well as maintenance costs.

CONCLUSIONS

The easiest way to summarize the performance of the Turnpike pavement over the past seven years of operation is to say that the design is adequate and that the traffic load is unique in the history of highways. With proper maintenance and repair the Turnpike pavement will be of service for a good many years, however, the type of future maintenance must be determined in advance from experience and experimentation on the roadway itself.

The time is near when a decision must be reached as to the methods of maintenance to be undertaken. This determination must include the type of materials which should be used, costs, expected life and safety. Careful analysis of all conditions which affect the behavior of the pavement, such as, excessive loads and water and base conditions, should be made to determine the necessary remedial measures.

It is apparent that there is a need for the formation of an advisory committee, similar to the Paving Committee established by the Authority in 1950, to study the unusual and intricate pavement maintenance problems of the New Jersey Turnpike. This committee should be composed of experts from various organizations dealing mainly in paving problems. The committee's reports of its findings would assist the Authority in determining proper action to take in the future to insure many years of service to the public traveling on the "fabulous" Turnpike.

ACKNOWLEDGMENTS

Grateful acknowledgment is made to J. Izatt, Shell Oil Company; Haller Testing Company of New Jersey, Testing of Cores; H. Thompson, District Engineer, Asphalt Institute; and John M. Griffith, Engineer of Research, Asphalt Institute.

Dynamic Forces Exerted by Moving Vehicles On a Road Surface

R. P. H. BONSE and S. H. KUHN, National Institute for Road Research, South African Council for Scientific and Industrial Research, Pretoria

This paper describes an apparatus for measuring the forces exerted at a point on a road surface by the wheels of moving vehicles. Detailed results are presented of measurements of three force components: vertical force, and longitudinal and transverse horizontal force components. The investigation included a study of the influence on these forces of tire inflation pressure, speed, acceleration, wheel load, height of measuring stud above road surface, etc.

Seven different vehicles were used covering a range of wheel (tire) load from 135 kg to 2,540 kg (300 lb to 5,600 lb), and a speed range of 15 kph to 75 kph (6 mph to 47 mph).

● MANY PAPERS have been written concerning the interaction between wheel and road, right from the early days of the self-propelled vehicle. In 1913 a study (1) was made of the suspension system of motor vehicles leading to an early theory of the dynamic behavior of cars and trucks. In this and subsequent work an attempt was made to find the magnitude of the impact forces applied by the wheel and how they are influenced by tire and spring stiffness, by shock absorbers, damping factors, shape and height of obstacles, and speed, etc.

Much of the experimental work done after World War I, until the early thirties, was concerned with comparisons between solid and pneumatic tires to help in questions of legislation on vehicle wheels and tires. Various methods for determining the vertical impact forces and their relation to the static wheel load have been developed. They can be divided into the following groups: (1) recording the deflection of the tire or of the sidewalls of the tire, (2) recording the deflection of the main springs and the acceleration of the unsprung masses, (3) recording the acceleration of sprung and unsprung masses, (4) measuring the deflection of the road surface, (5) measuring the variation of inflation pressure, and (6) direct measurement of the forces on the road surface.

It is not proposed to discuss these methods here, but it may be noted that most of the experimental work was carried out in order to determine the relation between static and dynamic wheel loads and the relative importance of the various influencing factors. A method for comparing the forces exerted by various tires at different speeds was published in 1934 (2), where a pipe filled with water was installed in the road and the increase in water pressure under a passing wheel was measured electrically.

A more recent publication (3) described a system developed for weighing vehicles in motion using electronic recording. The influence of the self-oscillations of vehicles on the forces applied to the road has also been studied (4). Before the solid tire had disappeared from the roads, some tests were carried out to measure the distribution of the three components of the force per unit area under static conditions (5, 6). It was found that the average vertical force per unit area (calculated from wheel load and area of contact) was up to 30 percent smaller than the actual measured maximum. If a tire rolls over a hemisphere of 11.3-mm diameter the force per unit area under static conditions underneath this obstacle increases up to 30 kg per sq cm for pneumatic tires, and is roughly independent of wheel load and position in the contact area. It was assumed that under dynamic conditions, that is, short-time loading, these forces per unit area would be increased by about 50 percent.

Just before World War II Markwick and Starks (7) made some measurements of the vertical and horizontal forces exerted on a small area (a fraction of a square inch) in a road surface by stationary and moving wheels. Some of their findings are of interest to compare with the results described in this paper. Pneumatic tires were found to exert maximum vertical stresses approximately $1\frac{1}{2}$ times the inflation pressure, inde-

TABLE 1

RANGE OF VARIABLES

Vehicle	Speed Range, kph (mph)	Acceleration Range, percent g	Deceleration Range, percent g	Range of Inflation Pressure, kg per sq cm	Stud Height, mm
Lloyd 600 Kombi	15-56 (9-35)	10-20	20-30	1.75	0, 2.5, 5.5
Land-Rover	10-30 (6-19)	20-30	30-40	1.75-2.1	0, 5.5
Chevrolet Sedan	10-75 (6-47)	10-30	20-40	2 1	0, 2.5, 5.5
Chevrolet Brookwood (loaded)	10-50 (6-31)	15-25		1.75-3.4	0, 2.5; 5.5
Chevrolet 3-ton Truck (loaded)	15-55 (9-34)	10-15	20-30	F 2.45-4.2 R 4.2 -4.9	0, 2.5, 5.5
Ford V8 Truck (loaded)	15-48 (9-30)			F 3.15 R 4.9	0, 2.5, 4.0; 5.5
Chevrolet 6-ton Truck (loaded)	15-30 (9-19)	10-20		R 4.9-7.0	0, 2.5

pendently of speed. Horizontal stresses under a stationary wheel were found to be directed inwards, and were up to 50 psi, whereas under a solid tire these stresses were directed outwards. Under a moving tire and with dry conditions, the horizontal stresses showed rapid alternations as the tire left the road whereas under wet conditions these alternations did not occur.

The present paper describes a systematic study of the three components of the force exerted by the tire of a moving vehicle on a circular flat stud one square inch in area lying in the road surface. The investigation has included a study of the influence of all the more important factors, listed in Table 1. In all, seven different vehicles were used enabling a range of single wheel load from 135 kg (for a light car) to 2,540 kg (for a loaded truck) to be covered. The latter figure would correspond (using the normal dual wheel arrangement of a truck) to an axle load of 10,160 kg (approximately 22,000 lb). Table 2 gives some relevant particulars of the vehicles.

DESCRIPTION OF APPARATUS AND TEST SITE

The apparatus for measuring road surface stresses consists essentially of a stress recorder box, which is installed under the road surface, and electronic and photographic gear housed in a mobile laboratory on the roadside (Fig. 1).

The stress recorder box is installed in a special manhole on the center line of the road. A manhole cover, surfaced with similar material to that of the road, is fitted over the recorder box leaving the road surface continuous except for the circular force-measuring stud, which is 1 sq in. in area and can be made either flush with the surface or projecting above it by adjustable amounts (Fig. 2). On either side of the stud on the center line of the road two dip switches project with which the vehicle speed is measured and the oscilloscope triggered. The operation of these switches is described in more detail in Appendix A.

TABLE 2

DATA ON VEHICLES

Vehicle	Total Weight, kg	Wheel Load		Tire				Wheel Base, m
		Front, kg	Rear, kg	Wheel	Size, in.	Ply Rating	Width of Tread, cm	
Lloyd 600 Kombi	750	205 (driven)	135	Front Rear	5.00-15	-	9.8 8.3	2.35
Land-Rover	1,460	370	380	Front Rear	6.00-16	-	11.1 11.4	2.24
Chevrolet Sedan	1,650	420	405	Front Rear	6.70-15	6	11.6 9.9	2.93
Chevrolet Brookwood (loaded)	2,300	480	650	Front Rear	8.00-14	4	12.7 12.1	2.98
Chevrolet 3-ton Truck (loaded)	5,080	680	$\frac{1780}{2}$	Front Rear	7.00-20 7.50-20	10 10	12.4 12.6	4.15
Ford V8 Truck (loaded)	5,230	700	2,330	Front Rear	7.00-20 9.75-18	8 12	12.1 18.9	3.96
Chevrolet 6-ton Truck (loaded)	6,600	680	2,540	Front Rear	8.25-20 9.00-20	10 10	17.5	3.98

Figure 1. General view of the test site.

The force-measuring stud is supported by two independent spring systems, as shown in Figure 3, which are deflected proportionally to the vertical and horizontal components of the force applied to the stud.

Normally the longitudinal horizontal force is measured. In order to measure the transverse horizontal force, the stress recorder box is turned through 90 deg and measurements are then made in the normal way.

Condenser plates fitted to the spring systems in conjunction with the electronic gear described in Appendix A are used to convert the mechanical deflections of the spring systems into electrical signals which can be photographed on an oscilloscope screen.

The spring systems together with the associated force transfer beams and stud have been so designed that a maximum horizontal force of 75 kg and a maximum vertical force of 190 kg can be measured. It was further ensured that the resonant

Figure 2. Close-up showing stress-recorder in position.

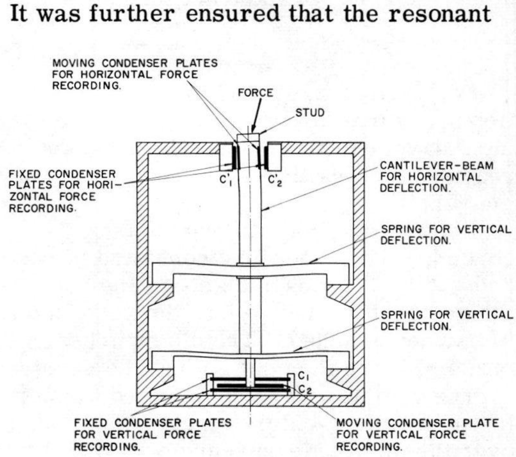


Figure 3. Diagrammatic sketch of stress-recorder.

frequencies of both spring systems were considerably higher than the frequency corresponding to the shortest loading time likely to be investigated. For a vehicle speed of 80 kph (50 mph) the induced frequency corresponds to about 60 cps, whereas the natural resonance of both spring systems was about 500 cps. However, it was found that the recorded traces were modulated by the natural spring vibrations at vehicle speeds in excess of 60 kph (37 mph).

The experimental setup is shown in Figure 4. An oscillator supplies an input signal of about 8 volts R.M.S. at 150 kilocycles per sec to the stress recorder. Electrical signals from the stress recorder are filtered and adjusted in a sub-chassis before being fed to a double-beam oscilloscope where the traces are photographed. A regulated power supply serves both the stress recorder and sub-chassis. The triggering signals from the dip switches are fed to the oscilloscope via a two-way switch and multi-vibrator. The signals for starting and stopping the electronic timer are obtained from a parallel switch attached to one dip switch and reach the timer via the sub-chassis which contains the battery supplying the current. The recorder cables are taken via an underground pipe from the manhole to the mobile laboratory at the side of the road.

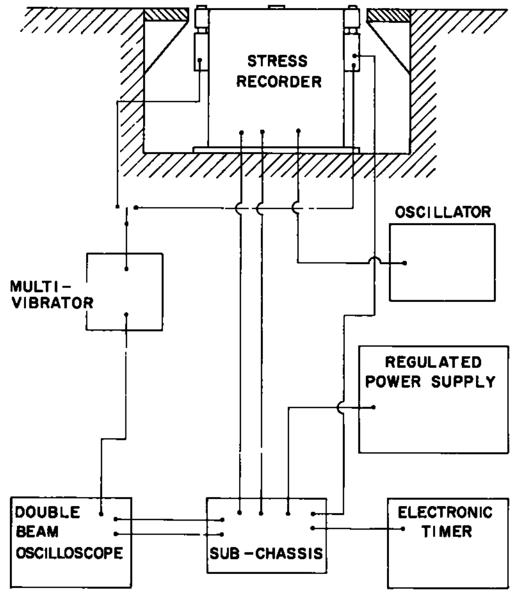


Figure 4. Block diagram of apparatus.

The experiments were carried out on a lightly trafficked provincial road near Pretoria. The road had a fairly even profile, both transverse and longitudinal, and was fairly horizontal and straight at the test site. A continuous white line was painted for about 60 m (200 ft) in both directions along the center line of the road to assist in driving the vehicles accurately over the stud.

To facilitate the determination of the exact point on the tire at which the stud was contacted, dust was lightly applied to the road prior to each run at a point one circumference of the tire away from the stud in the direction from which the vehicle was to approach. This enabled a print of the tire to be obtained on a strip of paper next to the stud which showed the transverse position of the tread relative to the stud.

EXPERIMENTAL PROCEDURE

For each of the vehicles used, the wheel load, wheel base, tire size and tire inflation pressure were recorded, the tire pressure being checked at regular intervals during a day's work. The acceleration and deceleration of the vehicle was found by taking readings on a calibrated braking efficiency meter clamped to the steering column of the vehicle.

Before each test was carried out the apparatus was adjusted and set ready for recording. The procedure consisted of balancing out the DC potentials for the horizontal and vertical forces and setting the oscilloscope sensitivity and beam-sweep speed to the required values so that the recorded trace would fall within the area of the screen to be photographed. The dip switches, as well as the control switches on the sub-chassis, were then set so that the required wheel would trigger the oscilloscope.

The vehicle was then signalled to approach and the camera exposure knobs pressed just before the vehicle reached the stud, and released again after the wheel had passed over the stud. Measurements of both horizontal and vertical forces were made from the photographic traces. Actual vehicle speeds were calculated from the timer readings and the length of the wheel base.

Figure 5. View of lever arrangement used for horizontal force calibration.

Calibration of the apparatus was carried out at regular intervals during a day's testing. For vertical-force calibration weights were stacked on the stud in increments up to 180 kg and, with the oscilloscope on continuous sweep, readings taken of the voltages corresponding to the weights on the stud. A similar calibration was used for the horizontal calibration using a lever arrangement by which known horizontal loads could be applied in increments up to 60 kg on the stud in either direction (Fig. 5). Typical calibration curves are given in Figure 6. Measurements of the "deviation," that is, the relative position of the tire center line to the stud, were made for each test run, as described above. All the testing was done under dry conditions.

DISCUSSION OF RESULTS OBTAINED

General

The following factors, which influence the forces applied at a point on the road surface, were varied:

1. Wheel load.
2. Tire size.
3. Inflation pressure.
4. Vehicle speed.
5. Measuring stud height above road surface.
6. Acceleration and deceleration.
7. Front and rear wheel.

8. Transverse position of contact, that is, distance between the center of the stud and the center line of the contact area of the tire tread, referred to in this paper as the "deviation."

9. Type of tread (normal road or cross-country type).

Other factors which may also influence the forces applied, but which were not studied, are:

1. Shape of contact area of the measuring stud.
2. Size of contact area of the stud.
3. Variation of the coefficient of friction between tire and stud, that is, roughness of the stud surface, material of the stud, wetness of the tire and/or the stud surface, etc.

The stud was made of aluminum and had a flat surface, serrated to a depth of 0.3 mm.

Of the three components of the total force exerted on the stud, only two could be measured at a time; for example, the vertical component and one of the horizontal components. Most of the tests done consisted of measuring the vertical component and the longitudinal horizontal component, that is, the horizontal component in the direction of motion of the vehicle. The order of magnitude of the transverse component, that is, horizontal at right angles to the direction of motion of the vehicle, was, however, also established in a small series of tests.

Before discussing the test results in detail, some comments on the scatter in the results should be made. It is well known that the load on a wheel is not constant under dynamic conditions, that is, as soon as the vehicle starts moving (4, 8, 9), the load varies in magnitude and in frequency, and is influenced by irregularities in the road surface, inflation pressure, speed, running conditions such as acceleration or cornering, the suspension system of the vehicle and its general dynamic behavior. Scatter was also caused by other experimental factors. The driver had to concentrate on running the right wheel over the stud in the required position and could not always watch the speedometer carefully, thus leading to small differences of speed in a given test series. It can be assumed, therefore, that the vehicle was not always in the same phase of self-oscillation during the contact-time, that is, the load on the wheel varied. Furthermore, there is the influence of tread pattern and of small out-of-balance effects of wheel and tire, which could lead to variation in the forces applied to the stud. The so-called "deviation," that is, the position of the stud relative to the center line of the tread, was measured as exactly as possible but, here again, small errors were unavoidable. Another potential source of variation in the longitudinal horizontal components under rear wheel tests was the unavoidable variation in applied torque at constant speed.

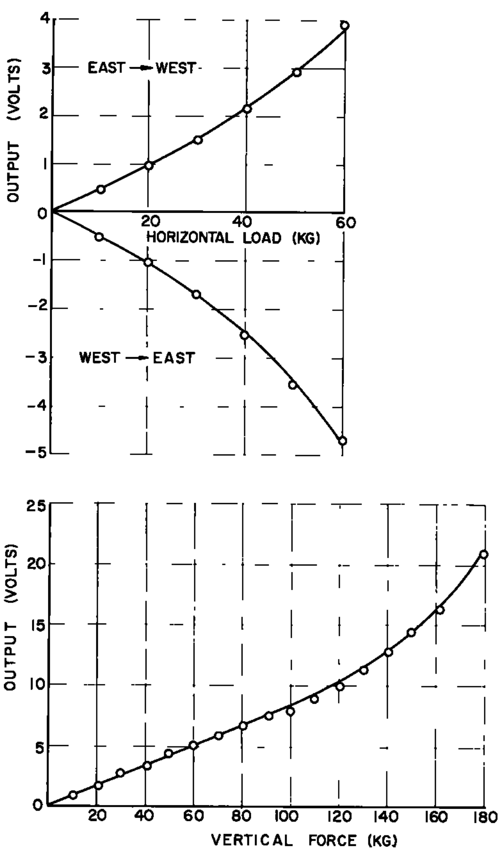


Figure 6. Voltage-force calibration curves.

Shape of the Force Recordings

Before discussing the results obtained and the influence of the different variables, some general comments can be made on the shape of the force-time traces recorded on the oscilloscope.

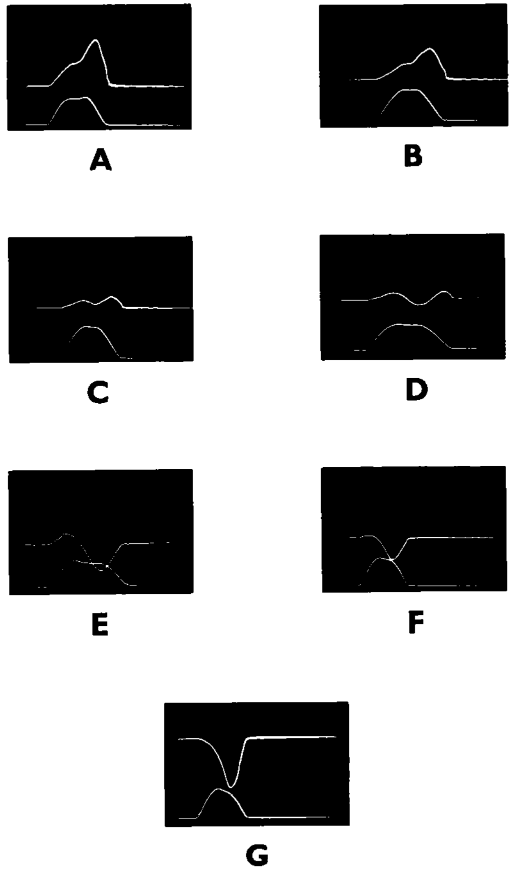


Figure 7. Typical force recordings showing longitudinal and vertical traces. Chevrolet Sedan: rear wheel; tire, 6.70-15; wheel load, 405 kg; stud height, 5.5 mm; inflation pressure, 2.1 kg per sq cm; deviation, -3 to +0.5 cm.

A. Speed: 25 kph (15.5 mph), acceleration: 0.37 g.
B. Speed: 22 kph (13.5 mph), acceleration: 0.23 g.
C. Speed: 25 kph (15.5 mph), acceleration: 0.10 g.
D. Speed: 19 kph (12 mph), acceleration: none.
E. Speed: 43.5 kph (27 mph), acceleration: none.
F. Speed: 34 kph (21 mph), deceleration: 0.13 g.
G. Speed: 34 kph (21 mph), deceleration: 0.39 g.

Vertical Force Component. Typical force recordings are shown in Figure 7. In each record the lower trace represents the vertical force component. It will be noted that the side slopes of the force-time traces are almost linear, and that the top part is either horizontal or of a saddle-like form, usually having two peaks of similar height. When the deviation is great, so that only a part of the surface area of the stud is in contact with the tire, the shape of the vertical force recording approaches that of a half sine-wave.

Tread pattern, stud height above surface and acceleration or deceleration all influence the shape of the vertical force recording, but the basic shape for the tread's center line does not vary much. For vehicle speeds in excess of 60 kph (37 mph) the vertical and horizontal force diagram is modulated by a vibration with a frequency of 500 cps, which is attributed to the natural frequency of the mechanical part of the recorder system. The amplitude of this vibration is, however, very small in comparison with the signal.

Longitudinal Force Component. The shape and magnitude of the longitudinal horizontal force component depends very much on the condition of vehicle motion. Figure 7 gives a typical example of a driven wheel for acceleration, constant speed and deceleration. The upper trace in every diagram of Figure 7 represents the longitudinal force component, a trace above the zero line representing force on the stud against the direction of vehicle travel, and below the line with the direction of travel. If a large torque is applied to a wheel, as in accelerating or decelerating a vehicle, the longitudinal force trace has one predominant peak in the latter half of the time of contact, as shown in Figure 7a and b for accelerating, and 7f and g for decelerating. The traces show clearly the variation of the direction of the longitudinal peak force with degree of acceleration and deceleration. Running the car with constant speed over the stud (Fig. 7d and e), the longitudinal force trace of the driving wheel has in most cases three peaks, but sometimes only two. The first and the third one are always opposite to the direction of motion, the second one mostly in the direction of motion. At constant speed the difference between the shapes of the recordings for rear and front wheel are small, except in the case of the loaded trucks, where the driving wheel develops a comparatively high first peak which results in a greater area under that part of the force-time curve, representing the force driving the vehicle forwards.

Transverse Force Component. Figure 8 gives, as an example, two typical recordings of the transverse component. The shape of the force-time diagram is similar to a half sine-wave if the stud area is not in contact with the center line of the tire tread. These forces acting on the stud are always directed towards the center line of the tread, and are very small at small deviations.

A B

Figure 8. Typical force recordings showing transverse and vertical traces. Chevrolet Brookwood (loaded): rear wheel; tire, 8.00-14; wheel load, 650 kg; stud height, zero; inflation pressure, 2.8 kg per sq cm.

A. Speed: 17 kph (10.5 mph), acceleration: none, deviation: 3 cm.
B. Speed: 15 kph (9.5 mph), acceleration: none, deviation: 5.5 cm.

Resultant Forces

A vectorial picture of the resultant horizontal forces is given in Figure 9 for the rear wheel of the loaded Chevrolet Brookwood using a stud height of zero. It can be taken as a typical example for constant speed conditions, showing the instantaneous distribution of the horizontal resultant during the length of contact and over the contact area. Static tests carried out by Martin (6) gave a similar picture with force vectors pointing roughly to the center of the contact ellipse.

For the same conditions, Figure 10 shows the instantaneous resultant forces in planes perpendicular to the road surface and parallel to the direction of motion of the vehicle, that is, the resultant of the vertical and longitudinal components during the time of contact. It will be noticed that some of the series of forces parallel to the center line of the tire tread represent a longitudinal force-time curve having two peaks, some having three peaks. No reason can be given for the difference in the number of peaks; there was no obvious relationship between longitudinal peak number and stud height, deviation, speed, torque applied to the wheel, inflation pressure, tire size or wheel load. It is therefore reasonable to assume that the tread pattern is the cause; in other words, the number of peaks for the constant-speed condition depends on what

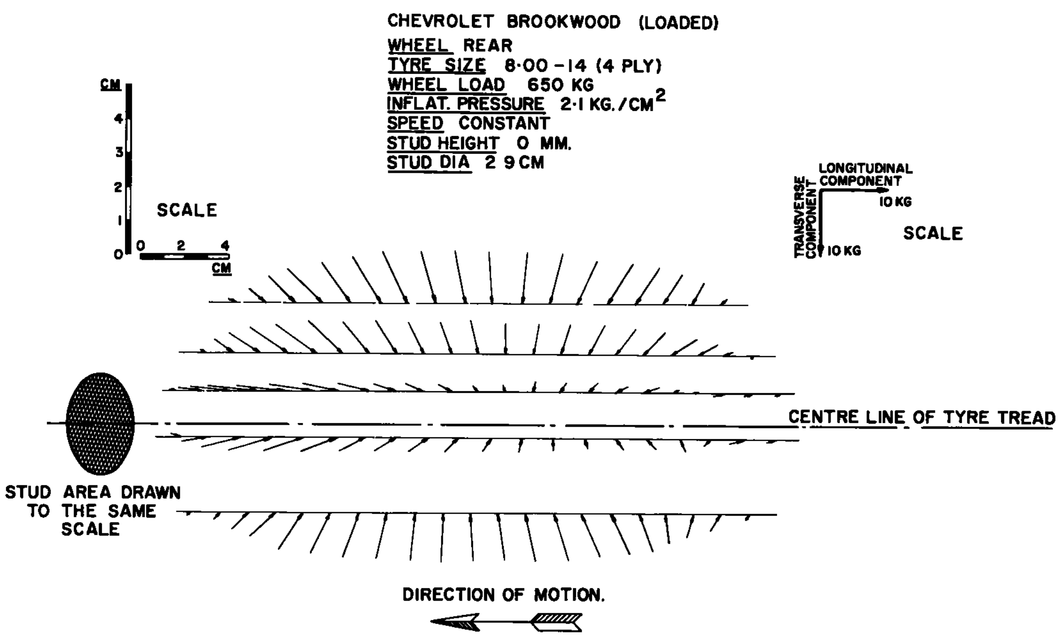


Figure 9. Vectorial diagram of resultant forces in the plane of the road over the contact area.

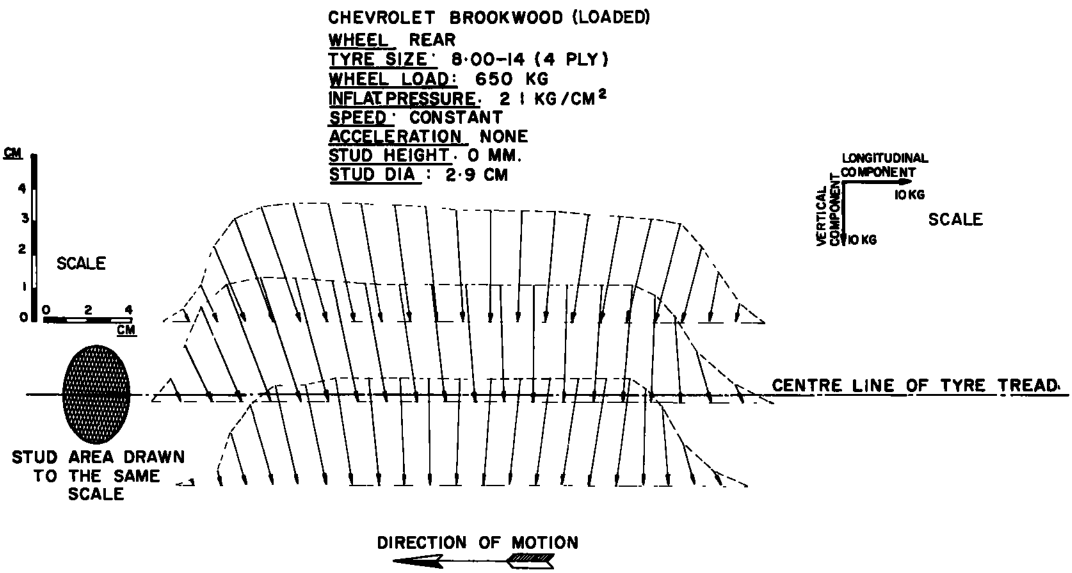


Figure 10. Vectorial diagram of resultant forces in three vertical longitudinal planes along the contact area.

part of the pattern comes into contact with the stud. In general about 75 percent of all tests carried out at constant speed gave three peaks for the rear wheel as well as for the front wheel.

Figure 11 gives again the instantaneous resultants of vertical and longitudinal components, plotted at short intervals of the contact time. As mentioned before, the peak force of the longitudinal component always occurs in the latter half of the contact time for acceleration as well as for deceleration. In the case of acceleration and deceleration, therefore, the greatest inclination of the vertical-longitudinal resultant is found just before the tire has rolled over the stud. This is independent of the stud height and of the tire and vehicle characteristics.

Peak Vertical Forces

The effect of the variables, listed at the beginning of this paper, on the measured force components will now be discussed. With so many variables it was not possible to vary each independently, but it is possible to draw some conclusions from the results and to indicate general trends. In most of the discussion on vertical forces which follows, the maximum force in a given trace is referred to; the shape of these traces has already been discussed.

Influence of the Deviation. It was found that the vertical peak force shows a characteristic distribution across the width of the tire, particularly pronounced for car tires and, in this case, clearly not very much influenced by the type of the tire pattern (Figs. 12, 13, 14 and 15). The vertical peak force increases with both positive and negative deviation and then decreases towards the edges of the tire as soon as only part of the stud area is covered by the tire tread. This particular distribution can be attributed to the influence of the side-walls of the tire which increase the stiffness along the edges of the contact area. This assumption is confirmed by an increase in difference between maximum peak force near the edges and the peak force at the tire center line with decrease in inflation pressure. It can also be seen that this difference increases with stud height but not as much as does the force at the tire center line.

The difference in peak force between the maximum and that at the tread center line is from 7 to 11 kg for the car tires and stud height zero, with the exception of the rear wheel of the Chevrolet sedan which showed a difference of about twice as much. At a stud height of 2.5 mm this difference increased to 13 to 24 kg for the car tires, and to 20 to 35 kg using a stud height of 5.5 mm.

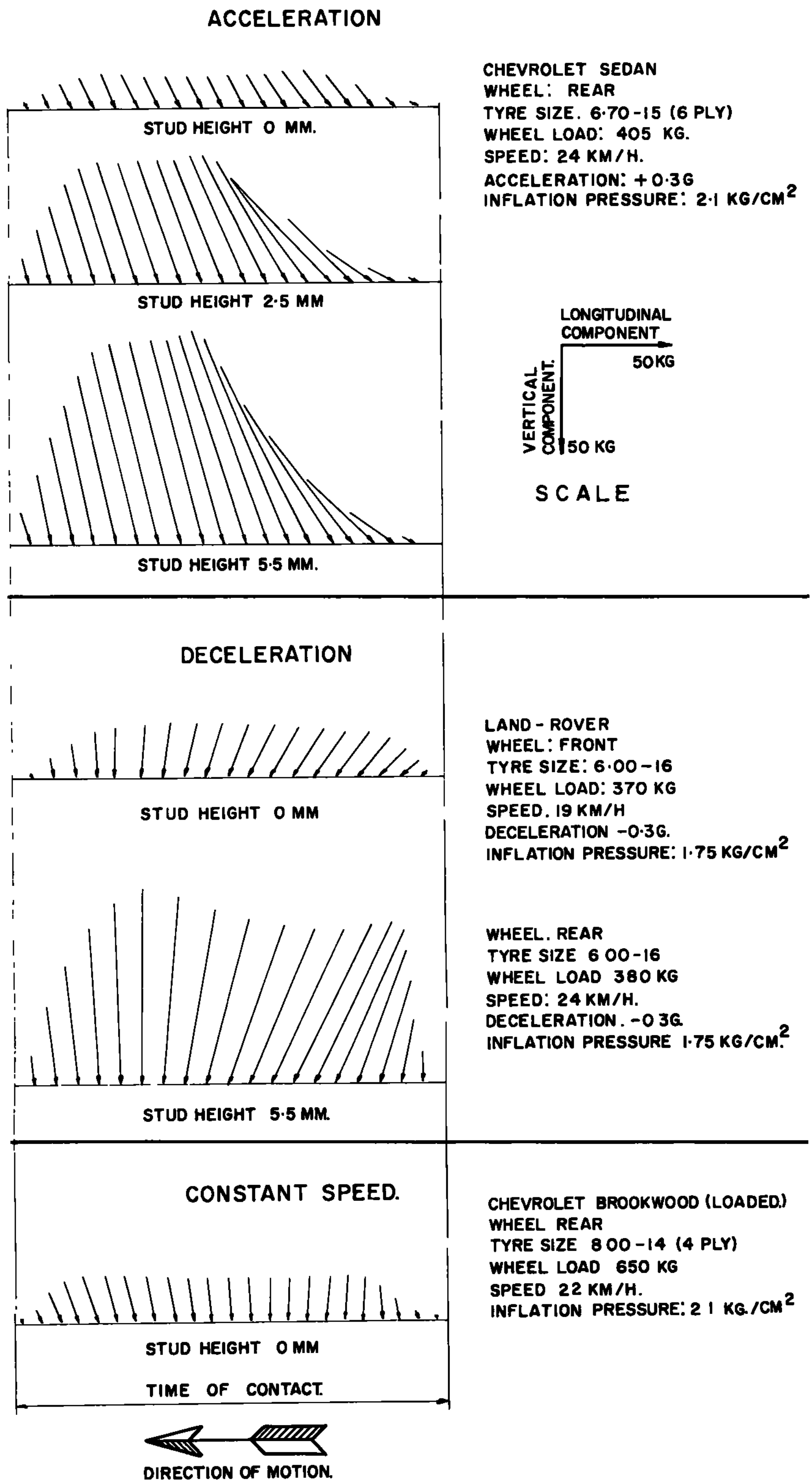


Figure 11. Effect of stud height and condition of motion of vehicle on the resultant forces in a vertical plane close to the major axis of the contact area.

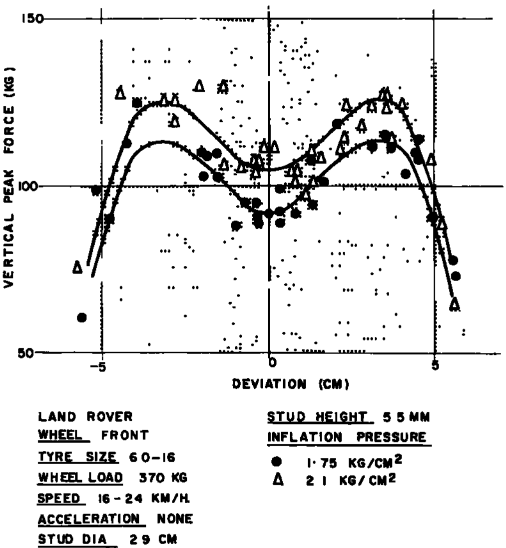


Figure 12. Influence of inflation pressure on the transverse distribution of vertical force at one stud height.

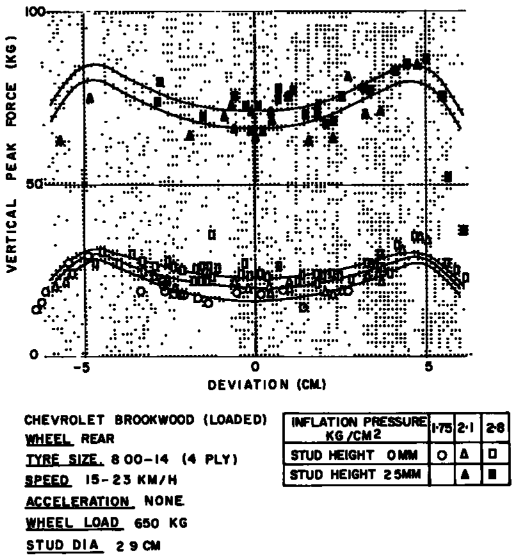


Figure 13. Influence of inflation pressure and stud height on the transverse distribution of vertical force.

The scatter of the test results obtained with the loaded trucks makes it difficult to give precise values, but Figures 16, 17 and 18 show at least the trend of increasing peak forces near the edges of the tire.

Influence of Inflation Pressure, Wheel Load and Tire Type. The influences of these three factors are interrelated. An increase in tire pressure decreases the contact area and, therefore, increases the mean vertical pressure within the contact area for a given wheel load. As shown in Figures 12, 13 and 16 the vertical peak forces,

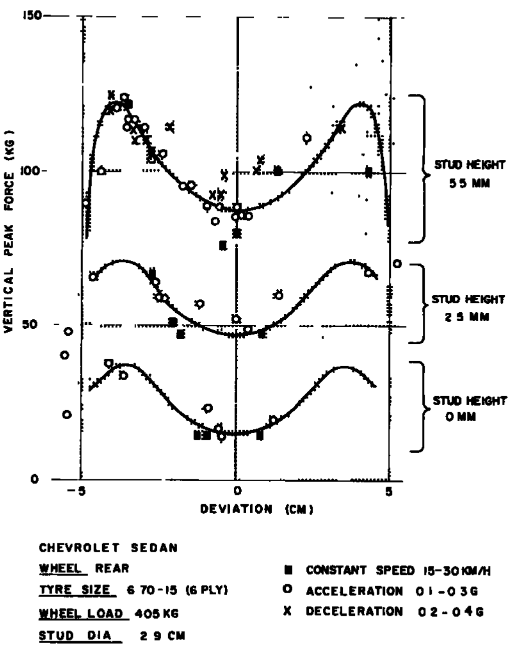


Figure 14. Influence of stud height and condition of motion on the transverse distribution of vertical force.

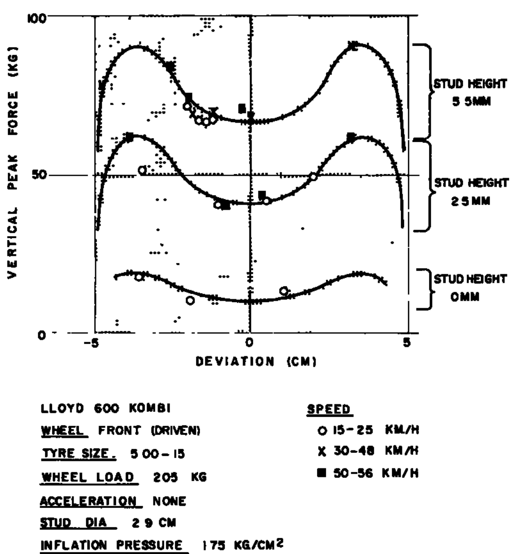


Figure 15. Influence of stud height on the transverse distribution of vertical force at different speeds.

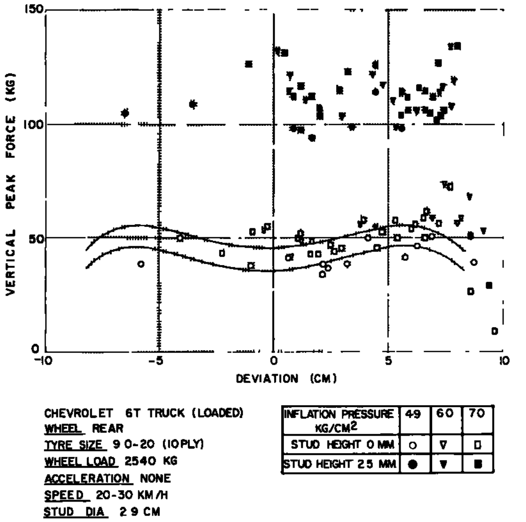


Figure 16. Influence of stud height and inflation pressure on the transverse distribution of vertical force for a heavy wheel load.

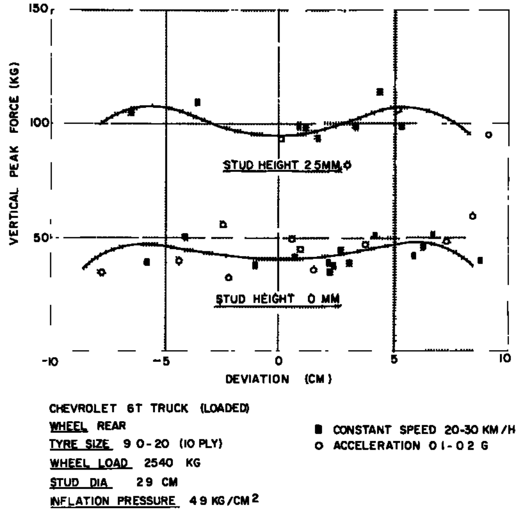


Figure 17. Influence of stud height on the transverse distribution of vertical force for a heavy wheel load at constant speed and accelerating.

however, also increase with inflation pressure. Although the vertical peak force at tread center line might not be expected to increase with a decrease in contact area, plotting vertical peak force at center line against inflation pressure (Fig. 19), in fact shows a definite relation. The points in Figure 19 are taken from tests using different

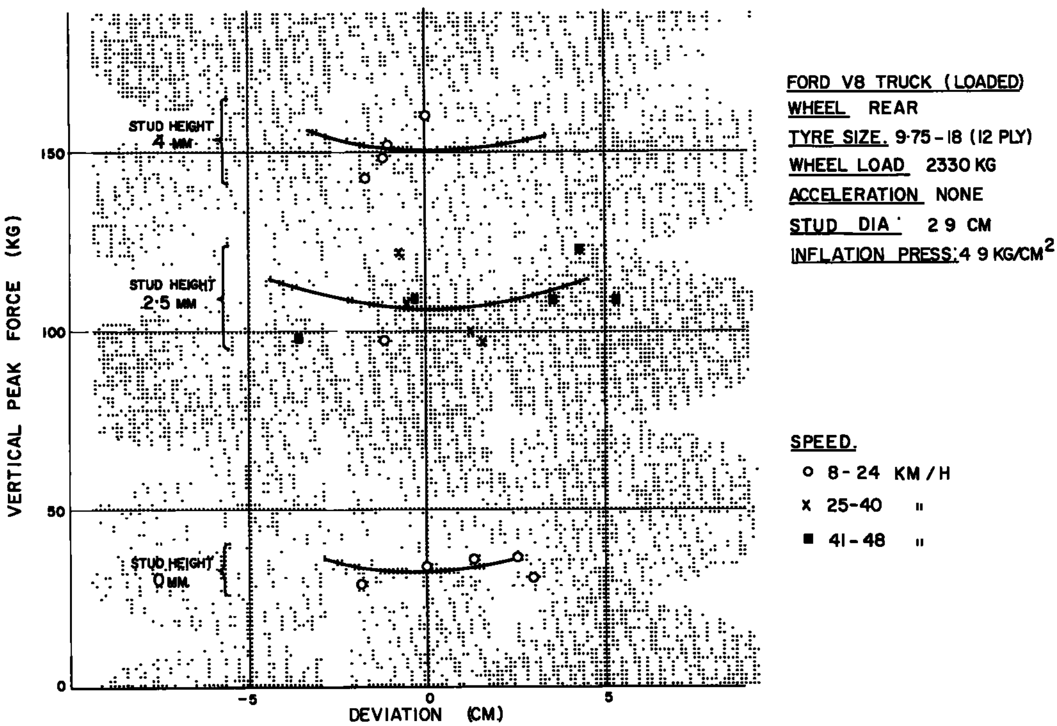
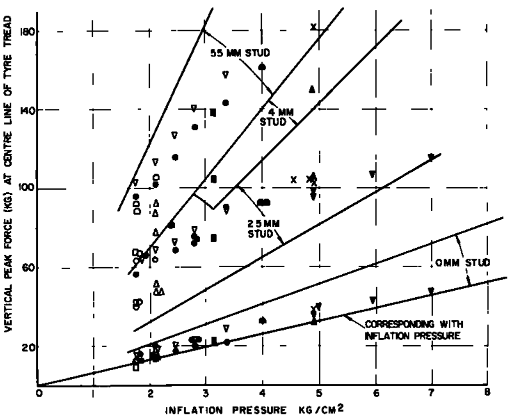


Figure 18. Influence of stud height on the transverse distribution of vertical force for a heavy wheel load at different speeds.



	WHEEL LOAD KG	TYRE SIZE	PLY
○	135	5 0-15	
□	205	5 0-15	
⌂	370-380	6 0-16	
△	405-420	6 70-15	6
▽	480	8 0-14	4

	WHEEL LOAD KG	TYRE SIZE	PLY
●	650	8 0-14	4
■	700	7 0-20	8
◓	680	7 0-20	10
X	1780/2	7 50-20	10
▲	2330	9 75-18	12
▼	2540	9 0-20	10

Figure 19. Relation between vertical force on the tread center line and inflation pressure at various stud heights.

tires and vehicles, front and rear wheels, different wheel loads and traveling speeds. It can therefore be stated that the vertical peak forces at the tread center line depend primarily on the inflation pressure. Increasing the inflation pressure in the 8.00-14 tire, for example, with a wheel load of 480 kg gives an increase in the vertical peak force at tread center line but not in proportion to the pressure increase. Static tests have shown that the contact area does not decrease proportionally with an increase in inflation pressure, but to a smaller extent. A plot (not shown) of vertical peak force against an average pressure (calculated by dividing static wheel load by static contact area as taken from tire prints for different inflation pressures) gave a strictly proportional relationship independent of wheel load. It should be remarked that the 8.00-14 tires at wheel loads of 480 kg and 650 kg (Fig. 19) were not of the same make.

Influence of Speed. The test results have revealed no significant effect of vehicle speed on the magnitude of the recorded vertical peak forces, as shown in Figures 15, 18 and 20. In a recent paper Chiesa points out (10) that the increase in tire stiffness (in kg per sq cm) of a certain car tire at a wheel load of 350 kg and inflated to 2.0 kg per sq cm is only 10 percent if the speed is increased from 30 kph to 75 kph (19 mph to 47 mph).

Influence of Acceleration and Deceleration. Accelerating a vehicle increases the load on the rear wheels and decreases it on the front wheels by an amount de-

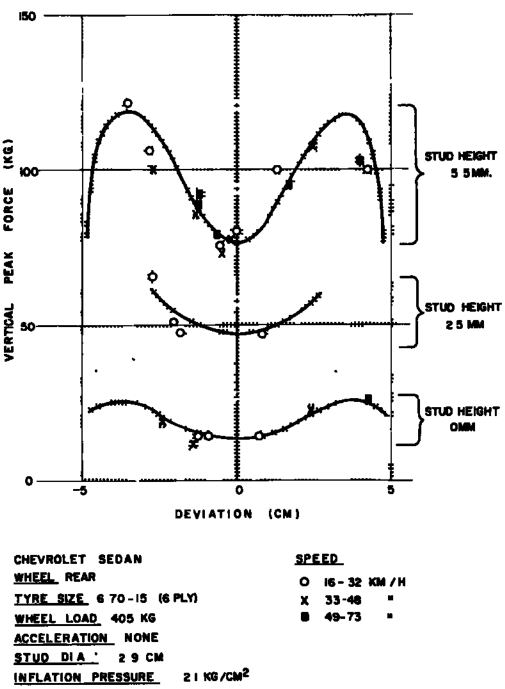


Figure 20. Influence of stud height on the transverse distribution of vertical force at different speeds.

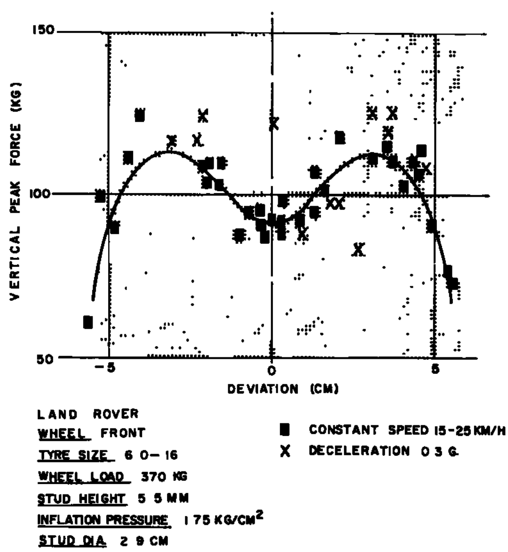


Figure 21. Influence of condition of motion on the transverse distribution of vertical force.

pending on the height of the center of gravity and the wheel base, etc. The results of some tests with various amounts of acceleration and deceleration are given in Figures 14, 17 and 21. Figure 14 shows that the recorded peak force (rear wheel) increases with deceleration despite the fact that the dynamic load on the wheel decreases with deceleration. This is explained by a change in shape of the force-time trace (Fig. 7f and g), arising from tire distortion under the high torque. In the case of the small acceleration of the Chevrolet 6-ton truck (Fig. 17) there is no significant difference in vertical peak forces at constant speed and in acceleration.

Influence of Stud Height Above Surface. Plotting the vertical peak force at the tread center line against stud height (Fig. 22) shows an almost linear relationship for the truck tires. In general, an increase in stud height from 0 to 2.5 mm results in an increase in vertical peak force at tread center line of 2.5 to 3 times in the case of the truck wheels. The relation varies for the Chevrolet Brookwood from 3 to 4.5 (Fig. 23). In-

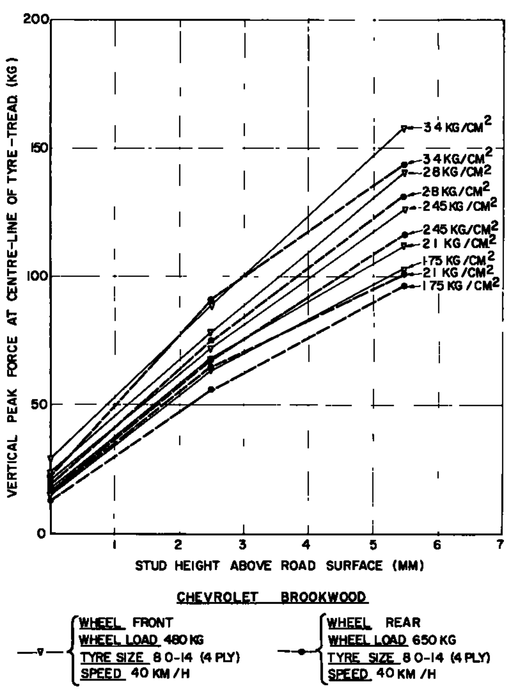
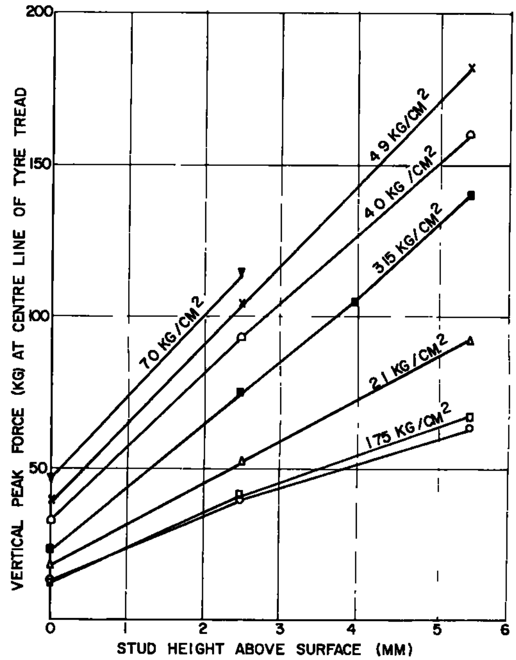


Figure 23. Relation between vertical force and stud height for various inflation pressures.



	WHEEL LOAD KG	TYRE SIZE	PLY
O	135	5 0-15	
□	205	5 0-15	
Δ	420	6 70-15	6
■	700	7 0-20	8
⌂	680	7 0-20	10
X	1780/2	7 50-20	10
▼	2540	9 0-20	10

Figure 22. Relation between vertical force on the tread center line and stud height at various inflation pressures and wheel loads.

creasing the stud height from 0 to 5.5 mm, the truck tires give a ratio of 4.5 to 6, and the car of 5 to 7.5.

Longitudinal Horizontal Forces

Influence of Stud Height. At constant speed the magnitude of the longitudinal forces is so small in comparison to the vertical component that only a small increase in longitudinal peak forces with increasing stud height can be noted. The magnitude of the longitudinal peak forces applied by cars on the stud is of the order of up to 5 kg using zero stud height, up to 10 kg for stud height 2.5 mm and up to 15 kg for stud height 5.5 mm. The range of forces for trucks is up to 10 kg for zero stud height and up to 15 kg for 2.5 mm.

Influence of Acceleration and Deceleration. As can be expected, a variation in

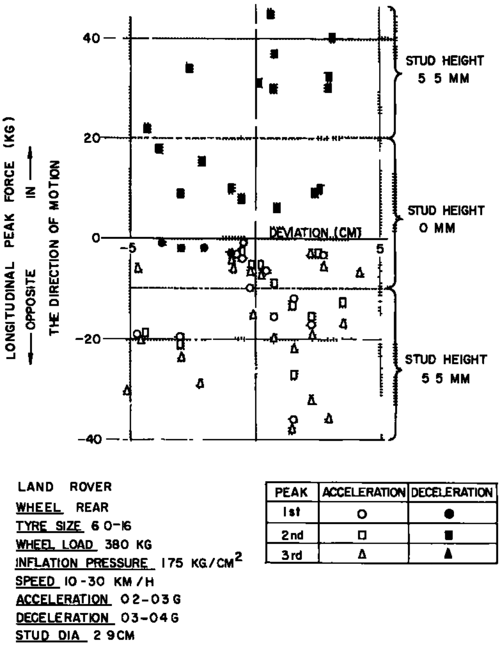


Figure 24. Influence of stud height and condition of motion on the longitudinal horizontal forces.

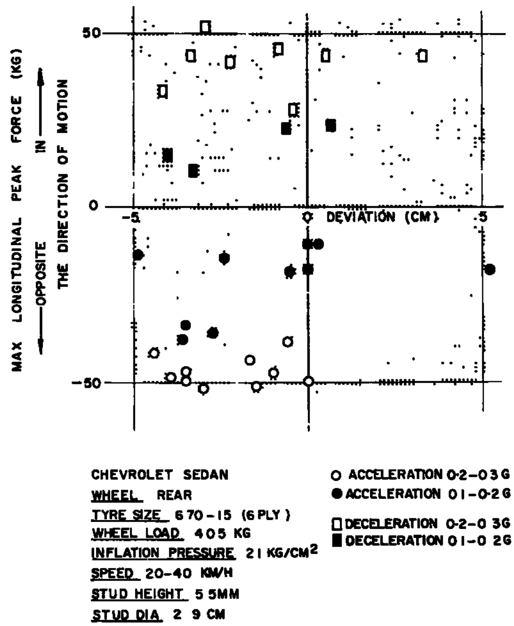


Figure 25. Influence of condition of motion on the longitudinal horizontal forces.

torque applied to a wheel has a big effect on the longitudinal force component. It has been already shown in Figure 7 that the shape of the longitudinal force-time trace varies considerably with the applied torque and also that the direction of the peak longitudinal force changes with a change in the direction of the applied torque.

Taking acceleration first, it can be seen in Figures 24 and 25 that the maximum peak forces (occurring in the latter half of the contact time) are of the order of up to 50 kg for the greatest stud height employed (5. 5 mm) using cars accelerated up to 0. 3 g.

In Figure 26 the maximum longitudinal peak forces are plotted against the degree of

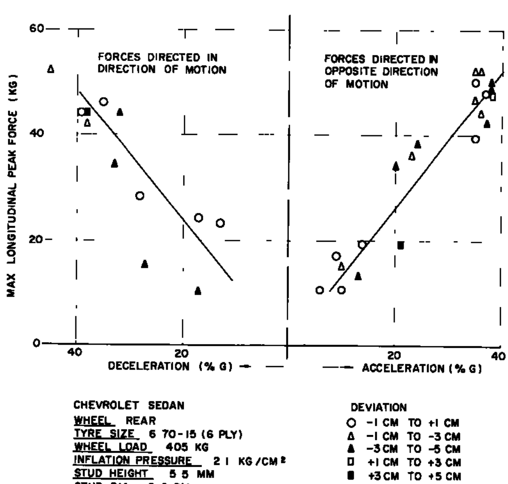


Figure 26. Relation between maximum horizontal longitudinal force and amount of acceleration.

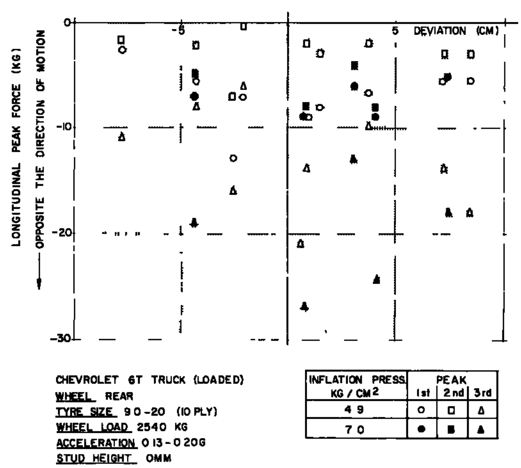


Figure 27. Influence of inflation pressure on the longitudinal horizontal forces for a heavy wheel load in acceleration at zero stud height.

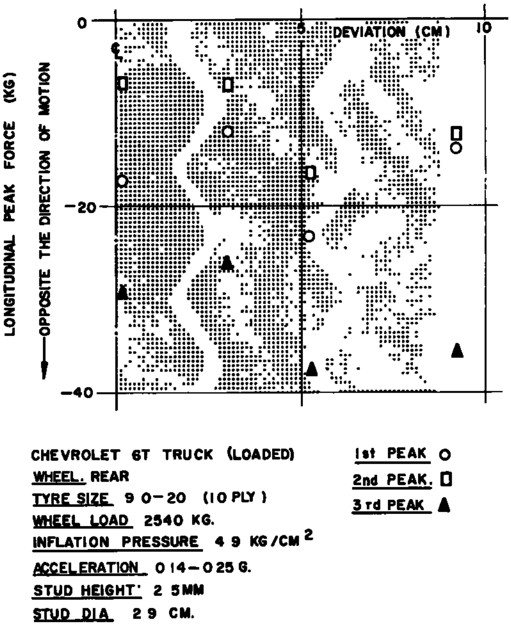


Figure 28. Longitudinal horizontal forces for a heavy wheel load in acceleration at 2.5 mm stud height.

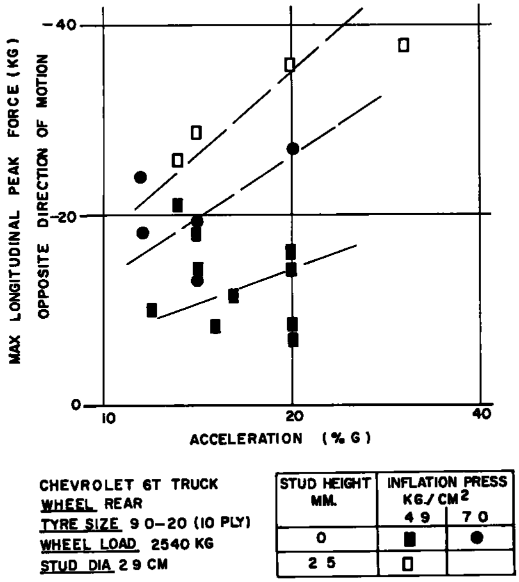


Figure 29. Relation between maximum longitudinal horizontal force and amount of acceleration for a heavy wheel load.

acceleration and deceleration for the rear wheel of a car, using a stud height of 5.5 mm. Figures 27 and 28 show the forces obtained by accelerating the Chevrolet 6-ton truck and the influence of the inflation pressure. Due to the greater driving force required for accelerating greater masses the longitudinal forces exerted by the truck are very much higher than those of the cars. Figure 29 shows the influence of degree of acceleration, with stud height and inflation pressure as variables, on the maximum longitudinal peak force exerted on the stud by a heavy truck tire.

Comparing the results obtained for deceleration and acceleration (Figs. 24 and

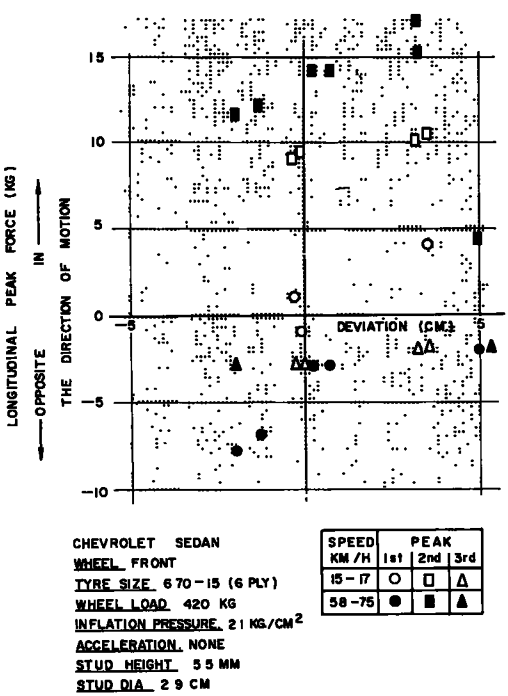


Figure 30. Influence of speed on the longitudinal horizontal forces.

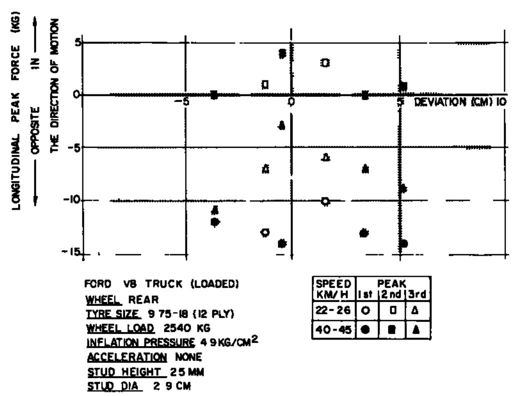
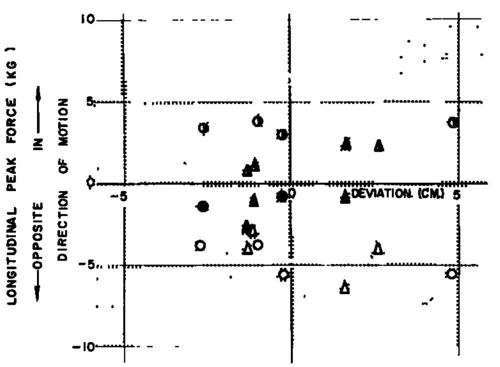


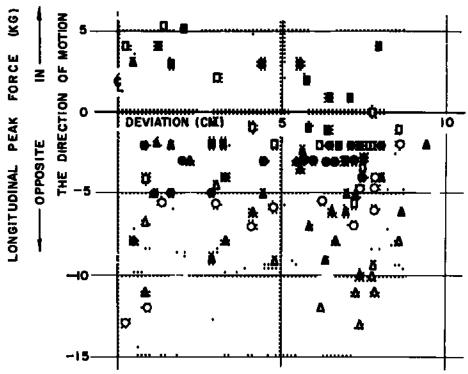
Figure 31. Longitudinal horizontal forces at two different speed ranges.



CHEVROLET BROOKWOOD (LOADED)
WHEEL REAR
TYRE SIZE 8 0-14 (4 PLY)
WHEEL LOAD 650 KG
SPEED 40 KM/H
ACCELERATION NONE
STUD HEIGHT 0 MM
STUD DIA 2 9 CM

INFLATION PRESS KG/CM 2	PEAK 1st	2nd	3rd
21	○	◐	●
28	Δ	◭	▲

Figure 32. Longitudinal horizontal forces at zero stud height and two inflation pressures.



CHEVROLET 6T TRUCK (LOADED)
WHEEL REAR
TYRE SIZE 9 0-20 (10 PLY)
WHEEL LOAD 2540 KG
SPEED 19-27 KM/H
ACCELERATION NONE
STUD HEIGHT 2 5MM
STUD DIA 2 9 CM

INFLATION PRESS KG/CM 2	PEAK 1st	2nd	3rd
5 6	○	□	Δ
7 0	●	■	▲

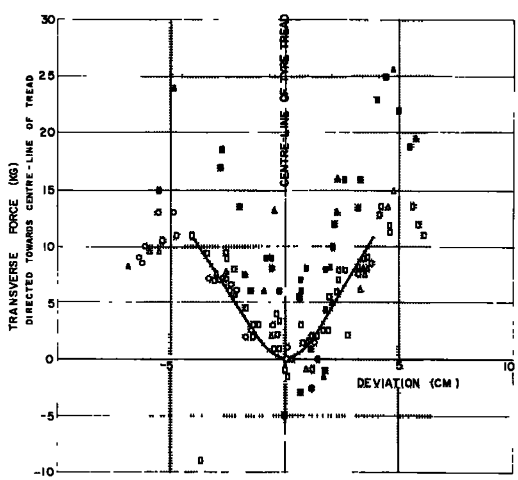
Figure 33. Longitudinal horizontal forces at two inflation pressures.

26), the same order of magnitude of peak forces is found in both cases if the stud height and inflation pressure are kept constant. The forces caused by deceleration, however, seem to be slightly smaller, which may be due to the effect of air resistance.

Influence of Speed. Some test results obtained at different speeds are given in Figures 30 and 31. If the stud is set at a height of 5.5 mm, increase in speed influences, particularly, the peak value in the direction of motion, that is, the second peak. This is illustrated in Figure 30 giving the results for a front wheel. This effect is also seen in Figure 31 giving results with a rear wheel.

Influence of the Deviation. The scatter in the longitudinal forces due to torque variations and tread pattern has not permitted any relationship with the deviation to be found, but it is probably safe to assume a fairly even behavior over most of the tire width with a decrease in longitudinal peak force near the edges of the track.

Influence of Inflation Pressure. The results obtained by using different inflation pressures under constant speed are given in Figures 32 and 33. For the stud flush with the road there is no obvious influence, and even at a greater stud height the effect of variations in torque seems to mask any possible effect of change in inflation pressure.



CHEVROLET BROOKWOOD (LOADED)
WHEEL REAR
TYRE SIZE 8 00-14 (4 PLY)
WHEEL LOAD 650 KG
ACCELERATION NONE
SPEED 15-20 KM/H
STUD DIA 2 9 CM

INFLATION PRESSURE KG/CM²	1 75	21	28
STUD HEIGHT 0MM	○	▲	□
STUD HEIGHT 2 5MM		▲	■

Figure 34. Influence of stud height on the distribution of transverse horizontal force at three inflation pressures.

Transverse Horizontal Forces

The magnitude of the transverse peak forces was also measured and typical results are shown in Figure 34 for the rear wheel of a loaded station wagon, with stud height and inflation pressure as variables.

As already mentioned, the direction of the transverse force is always towards the center line of the tire tread. However, very occasionally a small peak force of short duration in an outwards direction was recorded (the few negative points in Figure 34). This was attributed to irregularities in tread pattern or to small deviations from straight-line travel of the vehicle. It will be seen that the increase in transverse force with increasing deviation follows almost a parabolic curve outwards until only part of the stud surface is covered, when the force then decreases.

The influence of inflation pressure is again negligible on this horizontal component but the stud height has a marked effect, as shown in Figure 34. Some tests to measure this force component under a truck tire showed slightly greater forces than those under a car tire but, due to the greater width of the truck tire, the increase in force with deviation was not so steep. Variation in inflation pressure again produced no effect.

Force-Time Relations

From a study of the shapes of the force-time traces of the three components measured, the following observations have been made about forces generated under or very close to the tread center line.

1. The rise-time of the vertical force is 20 to 25 percent of the total time of contact at zero stud height, 28 to 32 percent at 2.5 mm and 32 to 36 percent at 5.5 mm. The time during which the vertical force decreases almost linearly is of a similar order, thus leaving for the horizontal part of the vertical force-time curve about 50 percent of the contact time at zero stud height, about 35 percent at 2.5 mm and 30 percent at 5.5 mm.

2. The rise-time of the longitudinal horizontal force under constant speed conditions to reach the first peak is slightly shorter than that of the vertical force—about 18 percent at zero stud height and about 28 percent at 2.5 mm and 5.5 mm. The time between the first and second peaks varies between 30 and 60 percent of the contact time, depending on stud height and on whether there are two or three peaks; the curve, however, at zero stud height shows the longest time between the first and second peaks.

The time between the second and third peaks is up to 5 percent less than the rise time to reach the first peak, and the time from the third peak to the end of contact is between 9 and 15 percent of the total time of contact, the higher values occuring at greater stud heights.

3. The transverse force-time curves are shaped like half a sine-wave.

4. In general, in order to express the total resultant force at a point on the road surface to a first approximation in terms of an equivalent frequency of sinusoidal loading, the effective contact length can be taken as a half wave-length. From measurements of the force-time traces for cars the average contact length is about 23 cm at zero stud height. As this figure includes the finite size of the recording stud at both ends of the contact area, 5 cm (say) should be subtracted from this figure, giving a typical effective (point) contact length of 18 cm. At a speed of 10 kph (6.2 mph) this gives an equivalent frequency of about 7.7 cps, or at 80 kph (50 mph) a frequency of 62 cps.

For the trucks used in this work the mean contact length (expressed at a point) was 27 cm which gives an equivalent frequency of 5.1 cps at 10 kph, or 41 cps at 80 kph.

CONCLUSIONS

The discussion given above has dealt with many of the detailed points that have emerged from this work. Many of these points need not be referred to again here. However, it may be useful to summarize a few of the general findings in this investigation. It should be pointed out that the results obtained in this work are based on the recording of forces exerted on an area of 1 sq in. (6.5 sq cm) of a road surface. Somewhat different results might be expected if the stud area were significantly greater or smaller than that used.

The Influence of Various Factors

1. The vertical force component is influenced mainly by the inflation pressure, the deviation, and the stud height.
2. The longitudinal force component is influenced mainly by the torque applied to the wheel and the stud height.
3. The transverse force component is influenced mainly by the stud height and the deviation.
4. Under normal, smooth-running conditions speed, as such, has no significant influence on the forces recorded.

Maximum Vertical Forces

1. For an inflation pressure of 28 psi (2 kg per sq cm) the vertical peak force during the time of contact at tire center line is in the range from 30 to 44 lb on 1 sq in. (2.1 to 3.1 kg per sq cm) at zero stud height, 100 to 155 psi (7 to 11 kg per sq cm) at 2.5 mm stud height, 155 to 240 psi (11 to 17 kg per sq cm) at 5.5 mm stud height. The ranges for an inflation pressure of 70 psi (5 kg per sq cm) are respectively 70 to 88 psi (5 to 6.2 kg per sq cm), 210 to 240 psi (15 to 17 kg per sq cm) and about 400 psi (28 kg per sq cm).
2. The maximum vertical peak forces are exerted near the edges of the track and they are 15 to 24 psi (1 to 1.7 kg per sq cm) greater than those measured at the tread center line at zero stud height.

Maximum Horizontal Forces

1. The longitudinal horizontal force component develops usually three peaks during the time of contact at constant speed. If a greater torque is applied to the wheel, as in accelerating or decelerating, the force-time curve deteriorates to a single-peaked curve. The magnitude of the longitudinal peak forces at constant speed is less than 11 psi (0.8 kg per sq cm) for car tires at zero stud height, up to 22 psi (1.5 kg per sq cm) at 2.5 mm stud height, and up to 33 psi (2.3 kg per sq cm) at 5.5 mm. For trucks these values are up to 22 psi (1.5 kg per sq cm) at zero stud height, and up to 33 psi (2.3 kg per sq cm) at 2.5 mm.
2. Under acceleration or deceleration (up to 0.4 g) longitudinal peak forces of up to 40 psi (2.8 kg per sq cm) at zero stud height were recorded and up to 110 psi (7.7 kg per sq cm) at 5.5 mm stud height.
3. The transverse horizontal component is zero at the center line of the tread and increases up to 33 psi (2.3 kg per sq cm) near the edges of the track at zero stud height, and up to 55 psi (3.9 kg per sq cm) at 2.5 mm. The direction of the transverse component is always towards the center line of the tire tread.

ACKNOWLEDGMENTS

The investigation described in the paper was carried out as part of the program of research of the National Institute for Road Research in Pretoria. The National Physical Research Laboratory in Pretoria designed the stress recorder which was built in the C.S.I.R. Central Workshop. J.B. Basson, T.B. Partridge and A.F. Reyneke assisted with the experimental work. The authors wish to thank P.J. Rigden for his advice and interest and for help in the preparation of the manuscript. The paper is published by permission of the South African Council for Scientific and Industrial Research.

REFERENCES

1. Bobeth, E., "Die Leistungsverluste und die Abfederung von Kraftfahrzeugen." Dissertation Technische Hochschule Dresden, Verlag M. Krayn, Berlin (1913).
2. Gerlach, E., "Messen der Beanspruchung von Strassendecken unter Verkehr." Zeitschrift Verein Deutscher Ingenieure (VDI), 78, No. 10 (1934).
3. Normann, O.K., and Hopkins, R.C., "Weighing Vehicles in Motion." HRB Bull. 50 (1952).

4. Hopkins, R.C., and Boswell, H.H., "A Comparison of Methods Used for Measuring Variations in Loads Transferred Through Vehicle Tires to the Road Surface." Public Roads, 29, No. 10 (1957).
5. Forschungsgesellschaft fuer das Strassenwesen e. V., Versuchstrasse Braunschweig Denkschrift X, Fahrversuche 1933-1935.
6. Martin, H., "Druckverteilung in der Beruehrungsflaeche zwischen Kraftfahrzeugreif und Fahrbahn." Zeitschrift Verein Deutscher Ingenieure (VDI), 80, No. 44 (1936
7. Markwick, A.H.D., and Starks, H.J.H., "Stresses Between Tire and Road." Journal of the Institution of Civil Engineers (London), 16, No. 7 (1940-41).
8. Essers, E., "Fahrzeuggewichte - Bahnkraefte - Strassenschaedigung." Automobiltechnische Zeitschrift (ATZ), 58, No. 1 and 2 (1956).
9. Koessler, P., and Beermann, H.J., "Reihenuntersuchungen ueber die dynamischen Lasten von Strassenfahrzeugen." Automobiltechnische Zeitschrift (ATZ), 60, No. 1 (1958).
10. Chiesa, A., "Verfahren zur Bewertung des Fahrkomforts mit besonderer Beruecksichtigung des Einfusses des Reifendrucks." Automobiltechnishe Zeitschrift (ATZ), 60, No. 3 (1958).

Appendix

The translation of mechanical deflections into electrical signals is achieved by means of two Blumlein capacity bridges which are particularly sensitive to capacity changes. The capacity changes are effected by means of a set of three condenser plates for each of the spring systems as shown in Figure 35. Since the electronic circuits for the horizontal and vertical force measurements are identical, only the one branch (say for vertical forces) need be described.

The outer plates of the two condensers, C_1 and C_2, which make up the two arms of the bridge, are fixed, while the common inner plate can move proportionally to the vertical force and is electrically earthed. Any movement of the center plate which increases C_1 will correspondingly decrease C_2 and vice versa.

The ratio arms of the bridge consist of two inductances, L_1 and L_2, wound on the same ferrite core in such a way that the mutual inductance between the windings is the same as the self-inductance of each. The inductances are further arranged in the bridge so that the effect of the mutual inductance with reference to one is such that it opposes the self-inductance of the other. The bridge is fed with a carrier frequency of 150 kilocycles per sec.

The bridge is initially balanced for the central condenser plate in the undeflected position. A movement of this plate results in a bridge output voltage proportional to the deflection. The force-deflection relationship is actually slightly curved (see Fig. 6) and the magnitude of the output voltage V_0 depends on the input voltage V_1 as well as on the carrier frequency. In practice, V_1 is kept constant and the carrier frequency kept at a value where the change in the response of the bridge is negligible for a small change in frequency.

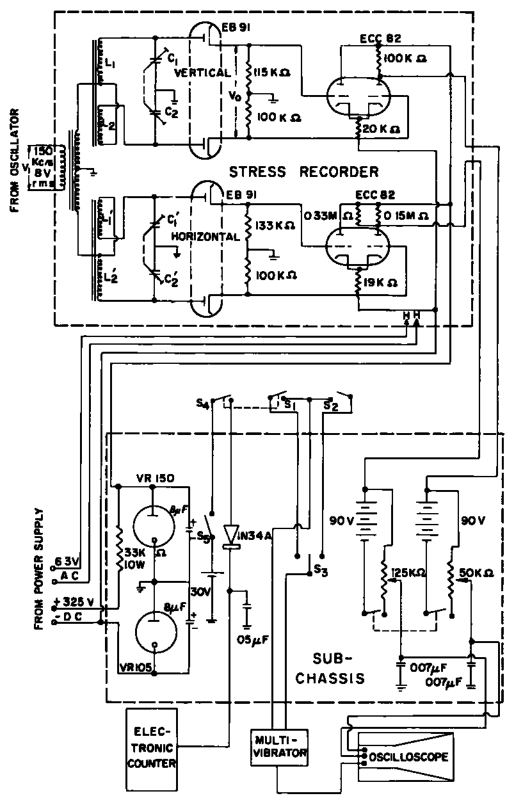


Figure 35. Electrical diagram.

The output voltage V_0 is rectified in a discriminator-type detector and amplified in a direct-coupled amplifier from which it emerges with a superimposed DC voltage. This DC voltage is cancelled by means of a dry battery and potentiometer before the signal is fed to the Y_1 plate of a double beam oscilloscope. A similar signal proportional to the horizontal force is fed to the Y_2 plate.

In order to photograph the traces when the wheel passes over the measuring stud the oscilloscope beam-sweeps are triggered by means of specially modified car dip switches which are actuated by the tire immediately before it reaches the force measuring stud.

The switching system is as follows: S_1 and S_2 are two micro-switches installed on either side of the force-measuring stud to facilitate triggering in either direction. S_3 is a switch fitted on the sub-chassis which can put either the signal from S_1 or from S_2 into circuit, depending on the direction from which measurement is to be made. The triggering pulse is then fed to the multi-vibrator unit where every second pulse is selected to trigger the oscilloscope. This enables either the front or back wheel of the car to be recorded. The selective triggering is achieved by means of neon indicators fitted in the anodes of the flip-flop stages of the multi-vibrator unit. Depending on the setting of the trigger level of the oscilloscope, neon indicator No. 1 will show that the just-triggered tube will be seen on the oscilloscope, that is, the front wheel. Neon indicator No. 2 in turn will indicate that the second pulse will trigger the oscilloscope, that is, the pulse given by the back wheel of the vehicle.

An additional switch, S_4, actuated simultaneously with S_1, is used to provide pulses from both the front and the back wheel. These pulses start and stop a micro-second electronic timer to provide a means of measuring the vehicle speed, using the known distance between the front and rear wheels.

Discussion

H. J. H. STARKS and A. C. WHIFFIN, Road Research Laboratory, Harmondsworth, Middlesex, England—The paper is interesting and provides useful data in a field where little information is available. For this reason, it is regretted that the authors have not given detailed information concerning the properties of the tires which they used and compared their results with those reported by earlier workers.

In Figures 12 to 21 of Bonse and Kühn's paper, the distribution of pressure across the width of the tire contact area is shown usually as a double-humped curve, giving higher pressures near the tire wall than in the center of the contact area. In Figure 1 of Markwick and Starks' paper, to which the authors refer in their introduction, double-humped curves were shown only for tires which were overloaded while curves having a central peak pressure were obtained with normally loaded tires. More recent work at the Road Research Laboratory, in which the vertical compressive stresses are measured within the road surfacing, have confirmed the curves reported by Markwick and Starks. The tire wall might be expected to have an effect with a normally loaded tire in cases where the pressures are recorded by studs which protrude above the road surface, but some of the diagrams shown by Bonse and Kühn refer to studs flush with the road surface. Was there anything special about the tires or treads used by the authors, and were some of them made of synthetic rubber?

Markwick and Starks showed that the mean pressure "p" on a cylindrical plunger of radius "a" projecting by an amount "z" into a semi-infinite elastic solid of elastic modulus "E" and Poisson's ratio "m" was given by the formula:

$$p = \frac{2E}{\pi(1 - m^2)} \quad \frac{z}{a}$$

Although Bonse and Kühn do not refer to this earlier theoretical and experimental work in detail, Figures 12 and 13 of their paper confirm that a linear relation exists between p and z/a. They, however, do not give figures for the hardness of the rubber used in their tires, from which the elasticity might have been inferred, so it is not possible to use their data to investigate the relation between normal pressure and hardness of the tread rubber.

Road engineers are as greatly interested in the forces between tires and fine-textured road surfaces as in those resulting from protruberances of the type represented by the stud heights of 2.5 and 5.5 mm employed by the authors. It is a pity, therefore, that the oscillograms given in Figure 7, showing the vertical and longitudinal forces for several degrees of acceleration and deceleration, refer only to a stud height of 5.5 mm. Admittedly, Figure 8 gives two oscillograms of the vertical and transverse forces for a stud set flush with the road surface, but it deals only with the steady velocity conditions. The oscillograms in Figure 8 are smoother than those in Figure 7 and there is no doubt that some of the irregularities shown in Figure 7 arise from the protruding stud. When Markwick and Starks used a stud set flush with the surface of the road and a tire running freely without change of velocity, they obtained a record of longitudinal pressure of the form shown by the upper trace of Figure 7e. The two areas between these lines and the axis represent the forces working in the forward and backward direction, so that their net difference represents the tractive force. The authors have made no attempt to deduce tractive forces from their measurements, but it would be of considerable interest if they would look into this for it gives much information as to the reliability of the data.

It is very difficult to see precisely what is being measured by a stud 2.9 cm in diameter protruding 5.5 mm above the road surface. It must deflect as a cantilever under the applied force and this will relieve some of the pressure and introduce errors of measurement. Have the authors looked into this? Also, what interpretation do they place on the data obtained with the protruding studs, and how do they propose to apply this information to normal road surfacing problems? In this connection, the information given in Figures 24 to 28 is particularly difficult to understand. Figure 26 shows that the maximum longitudinal force varies markedly with the deceleration or acceleration of the tire, the slopes of the lines being 1.2 kg for a change of deceleration of 1 percent g. Figure 27, however, shows a plot of horizontal forces covering the range 13 to 20 percent g which would alter a force of 10 kg by as much as 8.4 kg.

It is difficult to obtain data from this report concerning the important relation between vertical pressure, inflation pressure and wheel load. In the discussers' experiments, when the wheel load is not sufficiently great to overload the tire, it was found that increase of wheel load merely causes an increase in the tire contact area and little change in the vertical pressure. At constant wheel load, increase of inflation pressure reduces the ratio of peak vertical pressure to inflation pressure from nearly 3 at very low tire pressures to 1.5 at very high pressures, the ratio for the normal working pressure of the tire being about 2.0. This recent work at the Laboratory confirms that done before the war by Markwick and Starks.

Markwick and Starks used a plunger ⅛ in. in diameter with its end normally set flush with the road surface. When their results were assembled together to derive the longitudinal and transverse distributions of pressure, it was found that the pressure fell to zero in those portions of the tire tread where no rubber came into contact with the road surface. Bonse and Kühn use a stud which is 1.14 in. in diameter and, therefore, incapable of showing these effects of the tire tread. The tires they used, however, had large tread patterns and these may have caused some scatter of their results. Have the authors considered this and can they suggest how their results would be affected by the use of plungers of other dimensions and shapes?

R. P. H. BONSE and S. H. KÜHN, Closure—The authors wish to thank Dr. Starks and Dr. Whiffen for their comments on the paper. The reference to Figures 12 to 21 and the relation of these results to those published by Markwick and Starks calls for some clarigication. Results presented in Figures 13, 16, 17 and 18 actually refer to overloaded tires. Figures 12 and 21 concern tests where a stud height of 5.5 mm was used and where double-humped curves could be expected in any case except perhaps for grossly underloaded conditions. Apart from Figure 19, which does not refer to pressure distributions at all, Figures 14, 15 and 20 remain to be considered. It should be pointed out that the results presented in Figure 1 of the paper by Markwick and Starks referred to a 3.00 x 20 motorcycle tire with an inflation pressure of 50 psi (3.5 kg per sq cm), which was considered to be overloaded at 336 lb. Comparing this with a modern motorcycle tire of 3.00 x 19, it is found that this can carry a normal load of 400 lb at an inflation pressure of only 27 psi (1.9 kg per sq cm) (1). The authors doubt whether the results obtained on a pre-war motorcycle tire can be compared with those obtained with modern low-pressure car tires as referred to in Figures 14, 15 and 20. Modern tires are specifically designed to give higher flexibility and it can be assumed that, even under loads of 60 percent (Fig. 15) and 85 percent (Figs. 14 and 20) of the recommended maxima, "overload" conditions will be found comparable with tire behavior described by Markwick and Starks.

Regarding the question of the relationship between the formula derived by Markwick and Starks and the authors' results shown in Figures 12 and 13, the authors are unable to see any obvious correlation, but assuming that Dr. Whiffen and Dr. Starks meant to refer to Figures 22 and 23, it is agreed that there is a linear relationship between vertical peak force on the stud at the center line of the tires and the stud height, provided that the inflation pressure is kept constant. The authors consider that the vertical force exerted on a projection (or on a smooth surface) at a specific point across the tire width is a function of the height (and probably also of the diameter) of the projection, the stiffness of the tire carcass, the inflation pressure and the wheel load. The magnitude of the vertical force on the stud will furthermore depend on the position across the tire width at which it is measured. For projections of small diameters the tread rubber stiffness may become a major contributing factor, but this aspect was not investigated.

The oscillograms shown in Figures 7 and 8 were included as typical illustrations of the records obtained. The results extracted from a large number of oscillograms taken during the tests were given in the rest of the figures. The authors cannot see that calculation of tractive forces could have verified the feasibility of the data.

The authors decided on a circular stud of 1 sq in. in area with height adjustment instead of the very small stud (projecting 0.01 in.) used by Markwick and Starks because the 2.9-cm diameter stud was robust, would not lodge in the grooves of the tire and so result in a larger scatter of the results, and was only slightly larger in area than typical large stones used in some surface dressing work in South Africa. The beams supporting the stud were designed to deflect only slightly under load, the horizontal deflection being 0.002 in. for a maximum horizontal force of 60 kg and the vertical deflection being 0.002 in. for a maximum vertical force of 200 kg. It is considered that these deflections would not significantly affect the forces measured. Measurements on a projecting stud were included since it is believed that failure of a surface dressing often occurs through the dislodging by traffic of stones that project from the general level of the surface. The results presented in Figures 24 to 28 were included with the object of giving engineers some idea of the order of magnitude of the more severe forces likely to be exerted on projecting stones.

Dr. Whiffen and Dr. Starks should refer to Figure 19 in connection with the difficulty they mention in obtaining information concerning the relation between vertical pressure, inflation pressure and wheel load in the paper. It will be noticed from this figure that the vertical peak force on the stud varies significantly at constant wheel load with the inflation pressure. It is quite possible that the total vertical force over the contact area of the tire may remain substantially constant, but all the results referred to in this paper deal only with forces acting on the stud, and this must be borne in mind in reading the paper. It is also found from Figure 19 that the ratio of vertical peak force on the stud to the inflation pressure is not changed (within the experimental scatter) by an increase in the inflation pressure, regardless of the wheel loads involved, provided the stud is level with the road surface. For conditions where the stud height is increased above the surface this ratio does change with an increase in inflation pressure.

The tire tread pattern definitely resulted in a scatter of the results, as was found from tire prints taken at each determination on the road during the tests. On the question of the effects of different shapes and dimensions of studs, in conjunction with the tread pattern, on the results obtained, the following comments are put forward.

As the cross-section of the plunger is decreased one may expect the stiffness of the tire tread to become more important in determining the unit vertical pressure on it, in other words the effects of inflation pressure and tire carcass stiffness will become less important. This, in turn, may affect the shape of the double-humped curves found in the transverse distribution of vertical forces. Provided there is no slip between tire and road surface the unit horizontal and transverse forces may be expected to be virtually unaffected. When the stud is of small diameter it may fall wholly or partially in the grooves of the tire tread, resulting in a considerable increase in the experimental scatter.

The influence of the stud shape is more difficult to assess but one may again expect that the rubber stiffness will have a more pronounced effect when the shape of the stud tends to a sharp point with a probable increase in the unit pressures at the sharp points, as was found by Markwick and Starks. The observations in regard to flat studs will probably apply to the average unit vertical pressures on shaped studs of equivalent effective diameter. The unit horizontal and transverse forces may again be considered to be unaffected in the absence of slip.

REFERENCE

1. Bosch, K.F.Z., "Technisches Taschenbuch." 11 Auflage, Deutscher Ingenieur Verlag, Düsseldorf (1952).

Re-Evaluation of Kentucky Flexible Pavement Design Criterion

W. B. DRAKE, Associate Director of Research, and
JAMES H. HAVENS, Assistant Director of Research,
Kentucky Department of Highways

Prior to 1948, the criterion in Kentucky for designing the thickness of bituminous pavements was based upon a modified laboratory CBR and the 1942 curves developed by the California Department of Highways. In 1948, the Materials Research Laboratory reported: "An Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Deisgn of Highway Flexible Pavement." Included in that report as a recommended method of thickness design for use in Kentucky was a set of curves based upon an empirical relationship between minimum laboratory CBR and observed pavement performance. These five curves accounted for traffic groups up to 10,000, 000 EWL's. Since that time six additional curves have been included in the design charts for EWL groups up to 320,000,000. These additional curves were determined by extrapolation of the results from the 1948 study. Early in 1957, an evaluation of the design method was undertaken. The basis for this re-evaluation was a statistical comparison of actual pavement performances with the designed life as anticipated or predicted by the design curves currently in use. On this basis, projects were selected, design records assembled, performances surveyed, and the data analyzed. Selected pavements which had been designed by the method developed in the 1948 study were checked for performance by visual survey, by roughness measurements, by measurements of rutting, by measurements of loaded-deflection with the Benkelman Beam, and by opening pavements for observation and sampling. Flexible base types studied included waterbound macadam, bituminous concrete, granular dense-graded aggregate and combinations. Laboratory evaluation on basis of bearing tests were made.

1. The visual survey established a range of performance.
2. Road roughness measurements were related to CBR but no attempt was made to draw design curves from this data since it could be greatly affected by factors not related to structural design.
3. Pavements opened for inspection revealed permanent deformation in the upper layers of the system as well as intrusions of subgrade in waterbound base courses.
4. An alternate method of design based on limiting deflection under load was developed from the Benkelman Beam measurements. Curves drawn from this data indicate a need for a slightly greater thickness than provided by the 1948 curves.

● PAVEMENT DESIGN ENGINEERS are charged with the responsibility of determining the thickness and types of pavement courses necessary to support millions of vehicle-passes, intense loads, and to withstand extreme weather conditions. Most soils are inadequate for direct service of this type; and so pavements of differing thicknesses, depending on the supporting ability of the soil and the amount of anticipated traffic, are needed to distribute the loads and to confine and protect them. Pavement design engineers are, in fact, charged with more far-reaching responsibilities in the sense that thicknesses must be adequate but not excessive. It is this rather tedious balance between economy and pavement-sufficiency that guides the engineers and constitutes the general basis for any thickness-design criterion. Criteria of design are semi-empirical and semi-theoretical. In theory they involve boundary applications of stresses on layered, semi-infinite masses. Often these stresses are either indeterminate or obscure, and therefore, theory must be compensated by empiricisms. Basically, of course, empiricisms are founded on experience and experiment. In this sense, each road that is designed and built is, in part, an experiment or test of the design system used. Thus, a statistical analysis of the performance histories of a large number of pavements with regard to design-parameters, that is, bearing capacity of the soil, traffic, and pavement thickness, should provide a reliable derivation of a design criterion and should likewise reveal any need for modifications or re-adjustments in a criterion so derived and used.

Prior to 1949, the criterion in Kentucky for designing the thicknesses of flexible pavements was based upon a modified laboratory California Bearing Ratio (CBR) and the 1942 curves developed by the California Department of Highways (1). In 1948, the Materials Research Laboratory, in a report on "An Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Design of Highway Flexible Pavements" (2), recommended a similar method of thickness design for use in Kentucky and included a set of five curves based upon empirical relationships between EWL's, minimum laboratory CBR, and the observed performance of Kentucky pavements (Fig. 1)

These five curves accounted for traffic groups up to 10,000,000 EWL's (equivalent 5,000-lb wheel loads, two directions). Since that time, six additional curves have been included in the design charts and cover EWL-groups up to 320,000,000. These additional curves were determined partly by extrapolation of the results from the 1948 study. This series of eleven curves, with some modification in methods of evaluating traffic, has been used by the Department to design flexible pavements during the past ten years. Early in 1957, the Research Division was requested to evaluate the effectiveness of the extrapolated curves as well as the original five curves and to determine

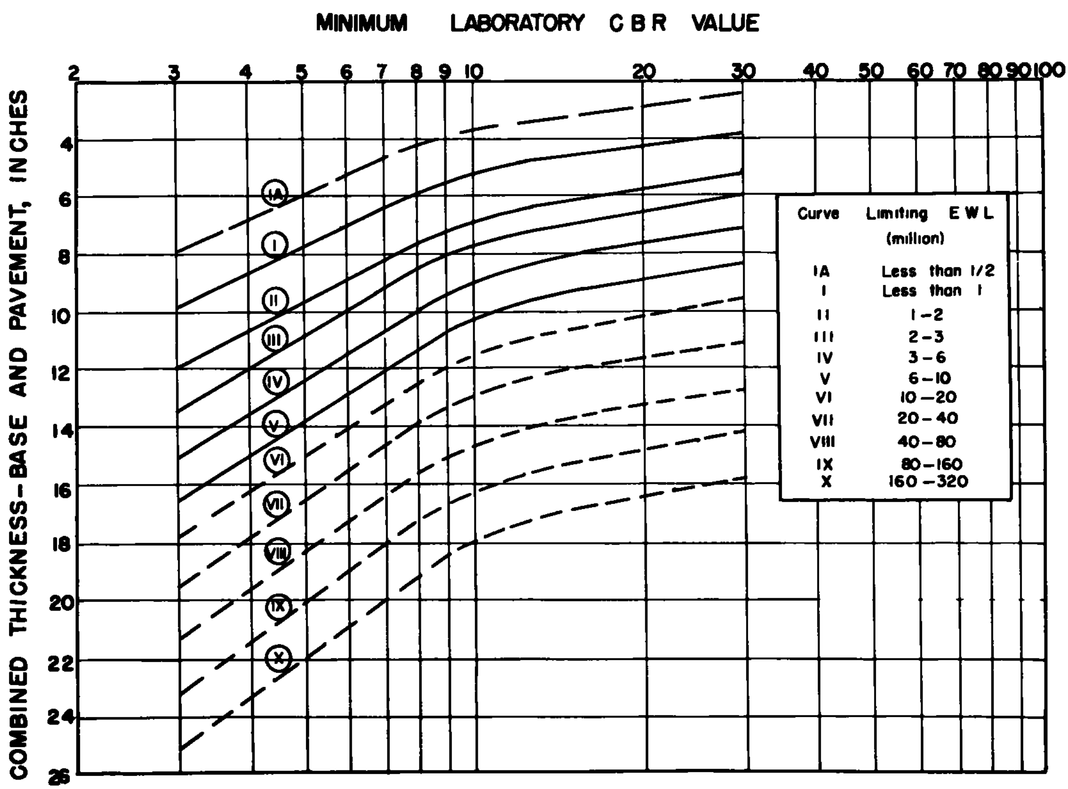


Figure 1. Kentucky flexible pavement design curves. Curves I through V proposed in 1948; IA and VI through X added by extrapolation, 1954.

if the curves should be further revised in any way or if factors heretofore not considered in the design of pavement thicknesses should now be taken into account.

Logically, of course, the basis for this re-evaluation would have to be a statistical study of actual pavement performances, accumulated EWL's, and subgrade CBR's. On this basis, projects were selected, design records assembled, performances surveyed, the data analyzed, and recommendations offered for revising the present design chart.

PRELIMINARY STUDIES

Selection of Projects for Study

The first criterion for selecting the projects to be studied was that the pavement must have been designed by the method recommended in the 1948 study. It was desired that the pavements be of high-type bituminous construction and have been in service as long as one year. The records were studied and a list of all eligible projects, meeting these requirements, was obtained. From a list of some 100 sections of road built since 1948, projects were selected so as to be distributed over the state as well as possible. Most of the major soil and geologic areas of the state were represented, and projects were selected so that available traffic groups were represented. An attempt was also made to select projects so that all of the more common base materials would come under study. Projects one mile or less in length and those not having sustained sufficient traffic were eliminated. Thus, curve revisions and bridge approaches were excluded. Projects involving large areas of salvaged pavement were also excluded. On the basis of these criteria, 70 projects representing 388.7 miles of Kentucky's flexible pavements were selected for study (see Fig. 2). Of these 70 projects, 57 were considered eligible, from the records available, for statistical analysis.

The Fayette-Madison County project and the Johnson-Lawrence County project, pavements studied in the 1948 investigation and not actually designed by the method currently under study, were included so as to provide extended distributions of projects.

Design Data

From the Division of Design, the design thickness of each pavement component for all projects was obtained and recorded. These values were then compared with the values as recorded on the plans and adjusted accordingly. When available, the design EWL and design CBR were also obtained. CBR's for most of the projects were gathered from soil reports on file in the Design and Materials Divisions.

Evaluation of Traffic

Traffic data were obtained from the Division of Planning. For most of the projects, the ADT (Average Daily Traffic, two directions) for each year, from the time the project was completed through 1957, was recorded. Also available were weight data and vehicle classification counts for each year, from and including 1951, for the ten permanent loadometer stations located over Kentucky. With this information, it was possible to calculate the EWL's which had passed over the pavements and thus to study the traffic history of each project.

By comparing the actual EWL value with the designed 10-yr EWL's, a "traffic-age" or "service-age" for the projects could be determined. Thus, if the actual EWL's at any age exceed the anticipated EWL's at that age, this would indicate that traffic has increased more rapidly than anticipated and that the service-age of the pavement exceeds its chronological age.

For computations of EWL's in the 1948 study, the only type of data available for the entire life of all roads studied was average yearly 24-hr traffic counts. Loadometer data were available from 10 permanent loadometer stations for the period between 1942 and 1947. During 1947, by the aid of temporary loadometer stations, loadometer data were obtained for all roads studied. Thus, where applicable, EWL's were computed from actual loadometer data. However, since many of the roads were built before 1942, it was necessary to project the trends in traffic and distribution factors, evident

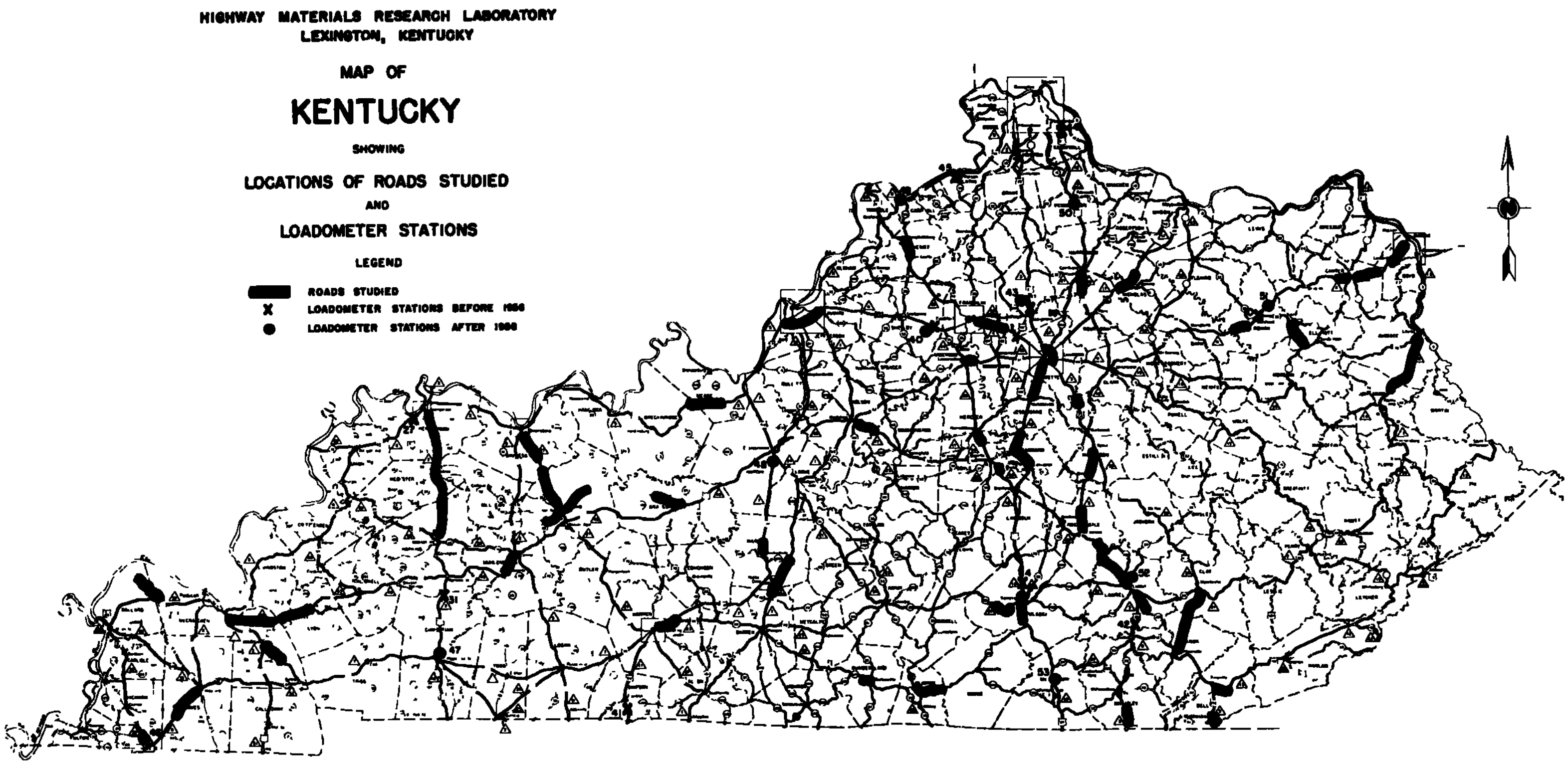


Figure 2. Locations and distribution of projects studied.

in the 1942-1947 data from each of the 10 permanent stations, to a year somewhat beyond the earliest construction date of any road studied. On this basis, the trends of each of the 10 stations were projected backwards to 1934. Then, for the year 1947, a ratio of EWL's to total vehicles per year was calculated for each road and each of the 10 permanent loadometer stations. On the basis of these ratios, similarity between a particular loadometer station and a particular road was established. Thus, the trends in traffic distribution where lacking on a particular road were calculated from a typical

COUNTY ——— ROAD NAME ——— ROUTE NO. ———

PROJECT LIMITS ——— PROJECT NO. ———

LOADOMETER STATION REFERENCE State Average, 1957: Volume Group 3000-3999

(1) Per Cent of Trucks 15.4

(2) Axles per Truck 2.557

(3) Average 24 hour Traffic 3640

(4) Av. 24 hour Truck Traffic = (1) x (3) 561

(5) Av. Yearly increase, 10 yr. period = $\frac{(4)}{2}$ 280

(6) Av. 24 hr. Truck Traffic for 10 yr. period = (4) + (5) 841

(7) Av. Axles per Truck for 10 yr. period = (2) + 0.05 . . . . 2.607

(8) Total Axels in 10 years = (6) x (7) x 10 x 365 = ... 8002,578

(1) Axle Load (tons)	(2) Total Axles	(3) % of Total Axles from L. Sta.	(4) Plus Correct.	(5) Corrected % of total Axles (3) + (4)	(6) Total Axles by Wt. Class (2) x (5)	(7) Calif. Factors	(8) EWL for two directions (6) x (7)
4½-5½	8002,578	5.205	0	5.205	416,534	1	416,534
5½-6½	"	4.732	0	4.732	378,682	2	757,364
6½-7½	"	4.732	1.25	5.982	478,714	4	1,914,856
7½-8½	"	4.574	0.85	5.424	434,060	8	3,472,480
8½-9½	"	4.101	1.50	5.601	448,224	16	7,171,584
9½-10½	"	1.261	0.35	1611	128,922	32	4,125,504
10½-11½	"	0.158	0	0.158	12,644	64	809,216
11½-12½	"	0	0	0	0	128	0
		TOTAL EWL for 10 year period (two directions)					18,667,538

Figure 3. Sample calculation for estimating 10-yr EWL's.

or similar loadometer station. These trends in distribution, when applied to the average yearly 24-hr traffic counts, provided a cumulative total of EWL's which was considered to be the total EWL's on each road since its construction or last resurfacing. The EWL's calculated in this manner were correlated empirically with other design parameters (CBR's, pavement thicknesses, and pavement conditions); and the best fitting curves, so derived, were adopted as the criterion for design.

None of these traffic data was tested for statistical reliability; and since the period involved the war-years, it was suspected that these data were unsuitable for predicting future traffic trends. Alternatively, it was assumed that truck traffic, in percent of existing ADT, would double in 10 years. (An example of the method of estimating 10-yr design EWL's for all roads included in this study is given in Figure 3. In 1954, the method was revised to a 20-yr estimated design EWL basis wherein traffic volume projection factors, vehicle classification factors, and axle and weight distribution factors are used in the computation. Examples of this method are given in Figure 4.) Thus, if it is also assumed that EWL's would increase in direct proportion to the vol-

TRAFFIC VOLUME GROUP 3000+

COUNTY —— ROAD NAME —— ROUTE NO. ——

PROJECT LIMITS —— PROJECT NO. ——

LOADOMETER STATION REFERENCE State Average 1957 Volume Group 3000-3999

(1) Per Cent of Trucks 15.4

(2) Average Axles per Truck 2.557

(3) Average 24 Hour Traffic 3640

(4) Average 24 Hour Truck Traffic = (1) x (3) 561

(5) Average 24 Hour Truck Traffic at End of 10 Year Period = 1.465 x (4) . 822

(6) Average Axles per Truck at End of 10 Year Period = (2) + 0.19 . . . . 2.747

(7) Total Axles in 20 Years = (5) x (6) x 365 x 20 16,483,650

(A) Axle Load (Tons)	(B) Total Axles (7)	(C) % of Total Axles From Load Sta.	(D) Correction	(E) Corrected % of Total Axles (C) + (D)	(F) Total Axles by Weight Class (B) x (E)	(G) EWL Factor	(H) EWL for Two Directions
$4\frac{1}{2}$-$5\frac{1}{2}$	Same as Item (7)	5205	0.09	5295	872,809	1	872,809
$5\frac{1}{2}$-$6\frac{1}{2}$		4.732	0.13	4862	801,435	2	1,602,870
$6\frac{1}{2}$-$7\frac{1}{2}$		4.732	0.27	5.002	824,512	4	3,298,048
$7\frac{1}{2}$-$8\frac{1}{2}$		4.574	0.15	4.724	778,688	8	6,229,504
$8\frac{1}{2}$-$9\frac{1}{2}$		4.101	0.11	4.211	694,127	16	11,106,032
$9\frac{1}{2}$-$10\frac{1}{2}$		1.261	0.05	1 311	216,101	32	6,915,232
$10\frac{1}{2}$-$11\frac{1}{2}$		0.158	0.00	0.158	26,044	64	1,666,816
$11\frac{1}{2}$-$12\frac{1}{2}$		0	0.00	0	0	128	0
TOTAL EWL for 20 year period (two directions)							31,691,311

Figure 4. Sample calculation for estimating 20-yr EWL's.

ume of truck traffic, the accumulation of EWL's at any age throughout the 10-yr period, expressed in percent of the 10-yr estimate, could be described theoretically by:

percent of 10-yr estimated EWL = $6.67x + 0.333x^2$

where x = chronological age in years.

The equation above describes the "theoretical curve," curve No. 1, shown in Figure 5. Curve No. 2, a locus of points determined by the least squares method, represents calculated actual accumulations of EWL's at all ages for all roads which were designed and built according to the 1948 criterion and for which traffic data were sufficiently complete to be included in this re-evaluation study.

While there is wide variance among the data (standard deviation = ± 67.64 percent); the average or trend shows close agreement with the theoretical curve. To this extent, it may be said that actual accumulations of EWL's have closely paralleled the predicted accumulations and that, on the average, "traffic-age" or "service-age" has closely paralleled chronological age. On the other hand, extreme variations in the percentage of accumulated EWL's at a particular chronological age, expressed as the 99.9 percent confidence limit, would be equivalent to ± 3 standard deviations or approximately ± 200 percent. Expressed on a 75 percent confidence limit basis, the extreme deviations, of course, would not exceed ± 1.15 x 67.64 percent. It may be similarly stated, therefore, that 15.9 percent of the roads accumulated traffic at a rate 1.68 times greater than the predicted rate. Likewise, 15.9 percent of the roads would reach 100 percent of their designed traffic-age within 68 percent or less of their designed life-expectancy. To be precise, statistically speaking, the mean square error could have been used rather than the variance since the ratio estimates used involve a slight bias. However, the bias would be negligible in comparison with the variance and can safely be ignored.

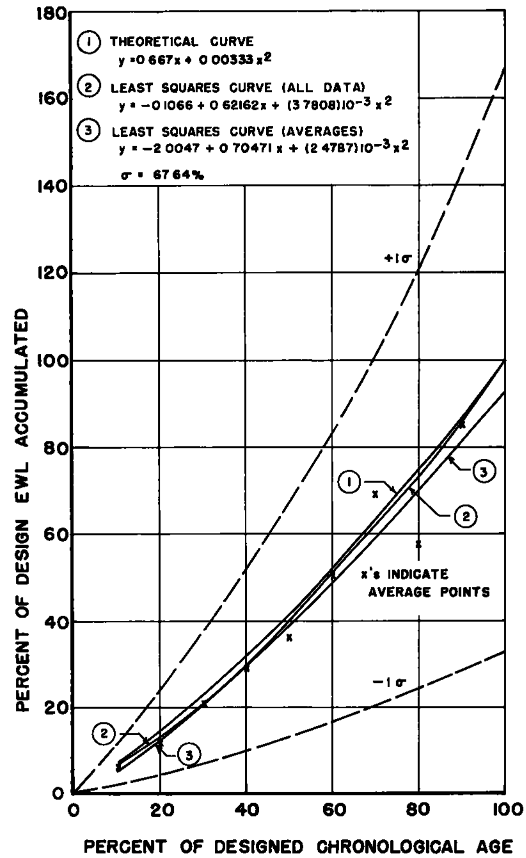


Figure 5. Statistical relationship between percent of designed EWL accumulated and percent of designed chronological age.

Traffic vs Pavement Life

Since the only parameters considered in the present design criterion are CBR's, pavement thicknesses, and EWL's predicted for a chosen number of years in the future, it is implied thereby that a pavement would have a designed life-expectancy comparable to the number of years for which the EWL's were predicted. Hence, the variations evident in actual accumulations of EWL's should have an analogous effect on actual pavement-life statistics. While terminal-life statistics are not available for this study, it may be surmised from variations in traffic alone that the service-life of 68 percent of the roads in this series may vary between 68 and 168 percent of their so-called designed life-expectancy or between 6.8 and 16.8 years.

Actual average life and survivor statistics (3, 4) should provide helpful insight into this aspect of the problem and should also provide a test of the validity of the design system. For instance, if the EWL's were accurately predicted for a 10-yr period and the average life of the pavements proved to be 18 years, it would have to be concluded that the thicknesses were excessive and that the design curves were unrealistic. The design system would seem equally unrealistic, of course, if the EWL's were accurately predicted for 18 or 20 years and the average life from survivor statistics proved to be only 9 or 10 years. Likewise, it can be seen from the present design curves (Fig. 1) that the difference in thickness between a 10-yr design and a 20-yr design, assuming that the 20-yr estimate of EWL's exceeds the 10-yr estimate by a factor of 2, would be about $1\frac{1}{2}$ in.

PERFORMANCE SURVEYS

Visual Inspection

Visual inspections of the various projects were made in the summer of 1957. To aid in evaluating pavement condition, each project was inspected throughout its entire length, and all evidences of distress were noted as to type, extent and location. Conditions recorded included cracking of all kinds—longitudinal, alligator, hairline—and skin and structural patching. Wavy sections, any signs of slides, fill settlement, as well as any adverse or unusual drainage conditions were noted. Numerous measurements of rutting were taken on each project in order to obtain an indication of the extent of permanent deformation in the wheel tracks. In order to reduce the notes taken during the visual inspection to a numerical value, the lengths of wheel track showing longitudinal cracking, alligator cracking, skin patching, and structural patching were summed for each project and tabulated as a percent of the total length of wheel track in the project.

Unfortunately the only traffic groups represented by enough samples to permit a cursory correlation of pavement condition with CBR and thickness were Groups IV and

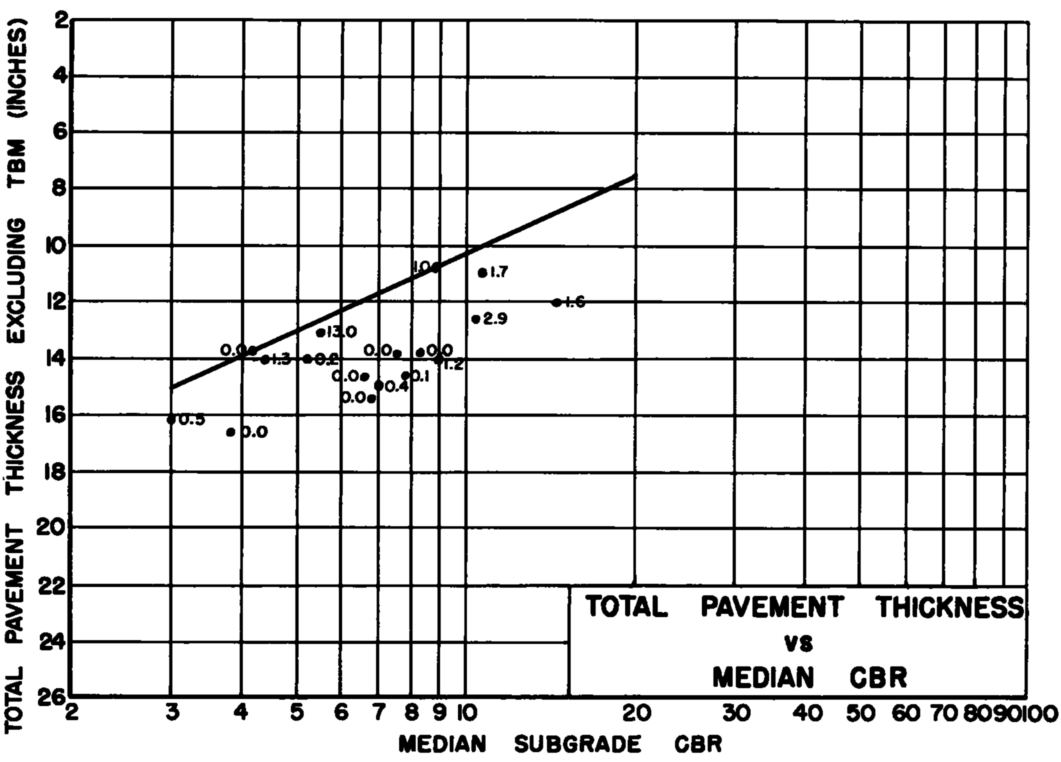


Figure 6. Pavement thicknesses vs median subgrade CBR's for all projects in which the accumulated EWL's fell within the limits of Traffic Group IV, 3 to 6 million.

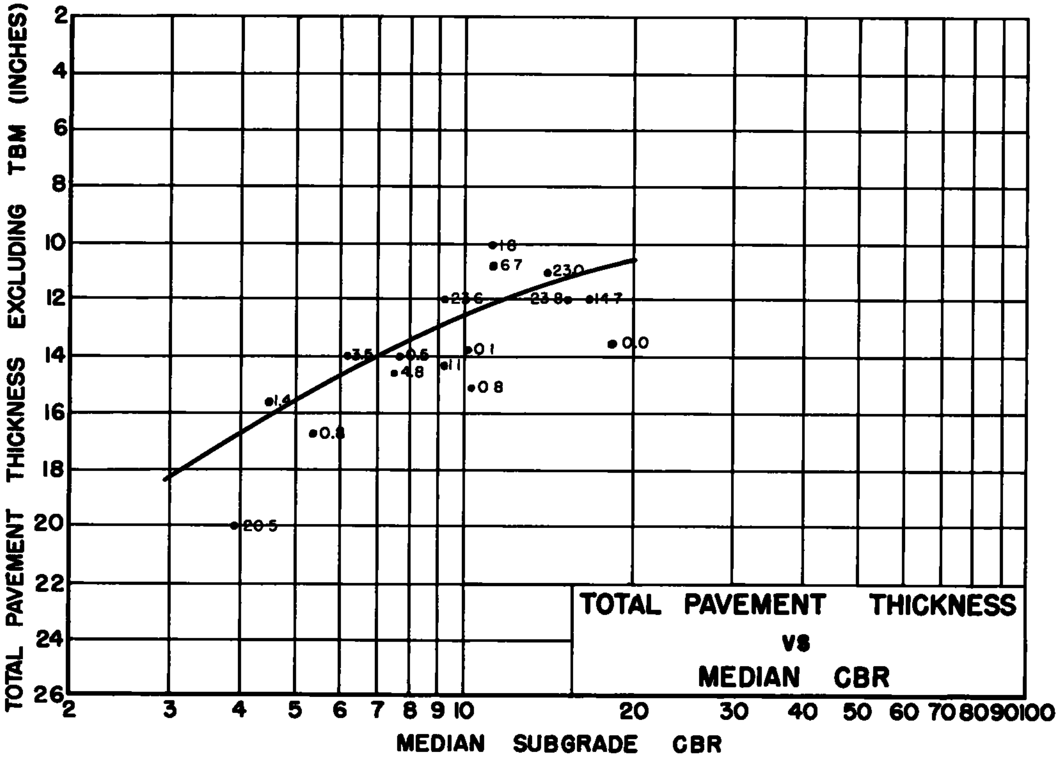


Figure 7. Pavement thicknesses vs median CBR's for all projects in which accumulated EWL's fell within Traffic Group VI.

VI. For projects in traffic Group IV, a plot of thickness vs CBR, with the percent of pavement failed noted by each point, is shown in Figure 6. Here again, there was not a sufficient number of failed pavements to clearly define a relationship, and the straight line represents an approximation of the required thickness assuming that the excessively failed pavement (13 percent) and the two adequate pavements falling on this line are near or below the critical thickness.

Data for projects in Traffic Group VI plotted in the same manner are shown in Figure 7. Here better control for the curve was provided by nearly equal numbers of failed and unfailed pavements in this group. The two excessively failed pavements, shown below the curve at CBR values of approximately 16 and 17, were on the same route (different projects); and performance may have been affected by other factors. Placing these points above the curve would require flattening the curve more sharply at CBR 10.

In Figure 8, the curves in Figures 6 and 7 are shown superimposed upon the original design curves and may indicate a need for slight revision of the design chart.

Rutting

Rutting measurements were made by laying a straightedge transversely across a traffic lane and measuring the maximum deviation (see Fig. 9). This measurement is not entirely rutting in the strict sense because a portion of the deformation may be the result of upheaval between the wheel tracks as illustrated in Figure 10. However, throughout this report the term "rutting" implies the total deviation from a straightedge. Measurements were made at more-or-less random intervals. From these measurements, a simple arithmetic mean of all values was computed for each project.

At first it was thought that rutting was the result of consolidation either in the pavement courses or in the subgrade. Any extreme rutting would then be considered an

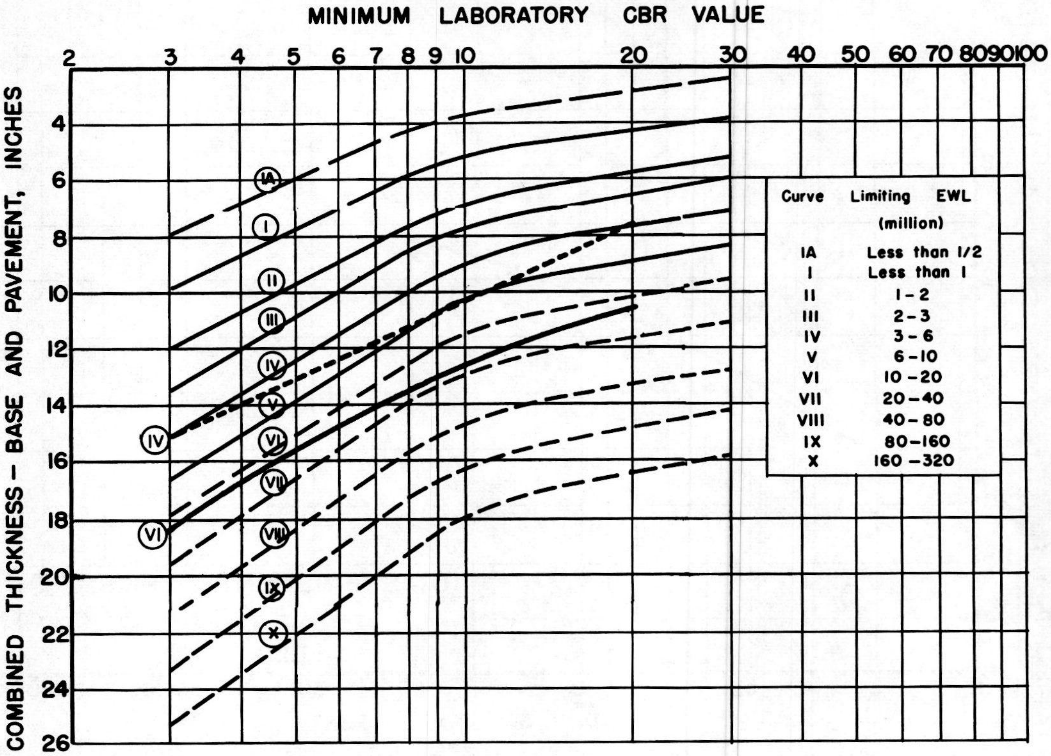


Figure 8. Present Kentucky design chart showing trend lines from Figures 6 and 7 superimposed.

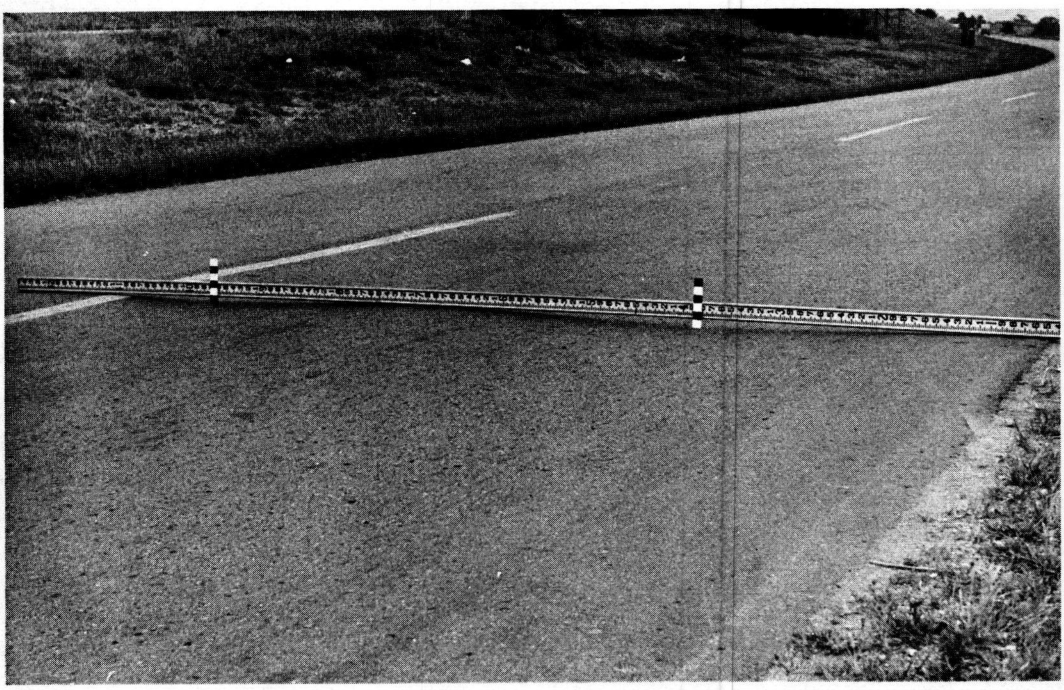


Figure 9. Rutting of pavement within wheel tracks as deviation from a straightedge.

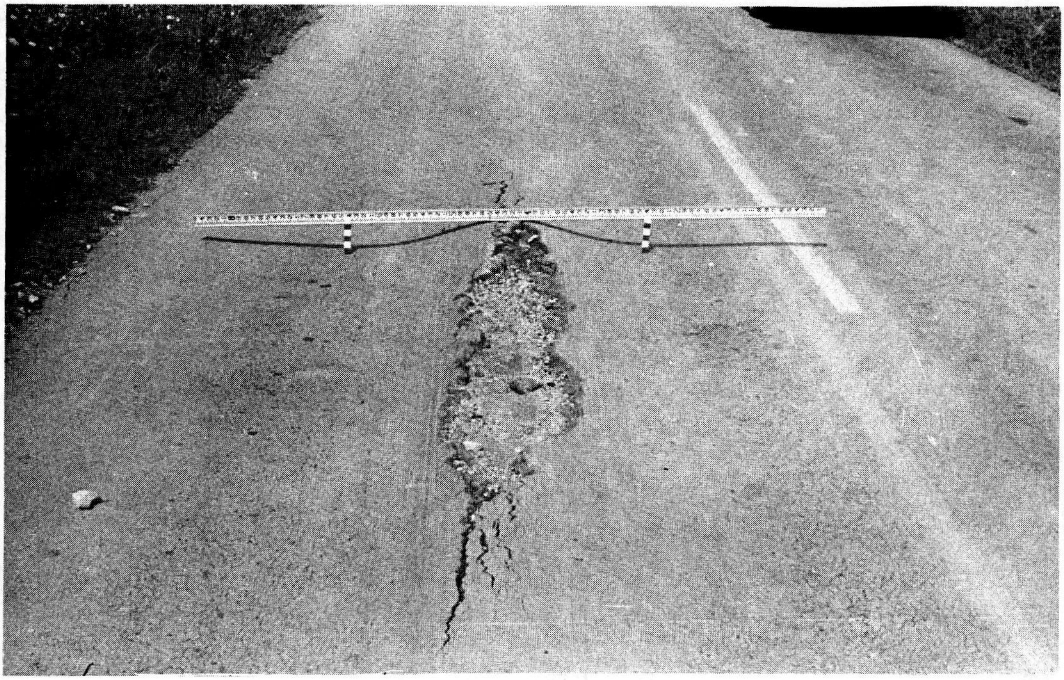


Figure 10. Extreme rutting and upheaval.

advanced stage of failure extending into the subgrade. However, from information obtained by opening selected pavements showing medium to extreme rutting, it was noted that, on the average, only 4 percent of the rutting occurred within the bituminous layers while 72 percent occurred within the granular base courses. These percentages are based upon comparisons of the thicknesses of the layers within and outside the wheel tracks. This indicated that the original thoughts concerning rutting were in error and that waterbound macadam is more highly susceptible to consolidation or movement under traffic than previously suspected. The densities of the WBM obtained while opening the pavements do not indicate any great degree of consolidation in most cases; thus, the deformations must result primarily from particle rearrangement and movement and must be the combined result of upheaval and subsidence.

From what has been said, it might be expected that rutting would increase with total pavement thickness and with traffic. These general trends are also indicated by Figure 11. However, the implied increases in rutting with increased pavement thicknesses are considered to be in the nature of a paradox and should be more properly interpreted as indicating that the

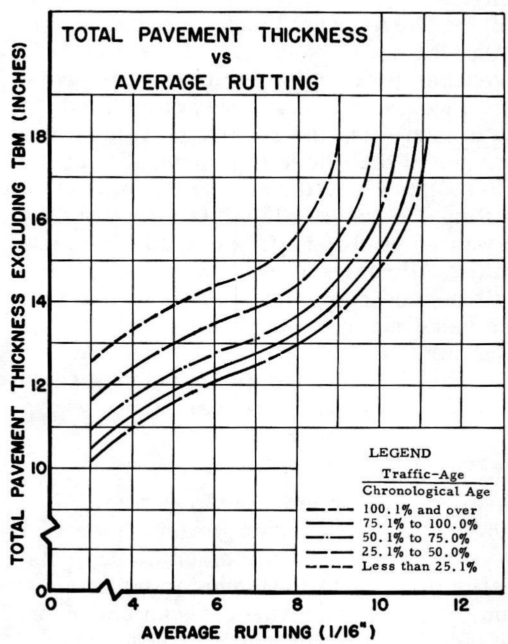


Figure 11. Generalized apparent relationship between thickness and rutting, according to traffic-age.

conditions causing rutting are more critical in the thicknesses designed for high intensities of traffic.

Road Roughness

With information from the field condition survey available, the traffic lane which exhibited the most distress was selected for an evaluation of roughness by the triaxial acceleration method reported in 1955 (5). The only deviation from the reported procedures was in evaluating the roughness records. The following method was used in determining the roughness of a road in terms of change in acceleration, sometimes referred to as "jerk." To obtain average acceleration, a compensating polar planimeter was used to measure the area under the vertical acceleration curve representing the length of pavement under consideration. Since the recorder chart was driven at a pre-set speed of ¼ in. per sec, each inch of chart length representing an elapsed time of 4 sec, and the galvanometer sensitivity pre-set to 2 in. per g, it was possible to resolve the total area beneath the curve into g sec (1 sq in. = 2 g sec); and:

$$\text{Area (in sq in.)} \times 2 \text{ g sec} = \text{Total g sec}$$

$$\frac{\text{Total g sec}}{\text{Total time}} = \text{Avg g}$$

(Total time = 4 x length of chart considered, in inches)

By careful measurement of many charts, the average frequency of the acceleration wave was found to be 5 cycles per in. or $\frac{5}{4}$ cps, giving a period of 0.8 sec per cycle. Since "jerk" is described as da/dt; average "jerk" would be:

$$\frac{\text{Average a}}{\text{Average t}} = \frac{A}{1.6L}$$

t = average period per acceleration cycle, 0.8 sec per cycle.

The vertical acceleration wave was analyzed by dividing the curve into short lengths of particular interest and determining the average "jerk" for that length using the above equation. To obtain an average "jerk" value for the entire project a weighted average was calculated.

In reviewing the roughness values it was noted that there is a general tendency for roughness to increase with increased rutting. However, in certain instances, it was noted that rutting could be rather uniform throughout a project and still result in good riding qualities provided that the vehicle remained in the wheel tracks. The curve in Figure 12 indicates that roughness decreases as the bearing capacity of the subgrade increases.

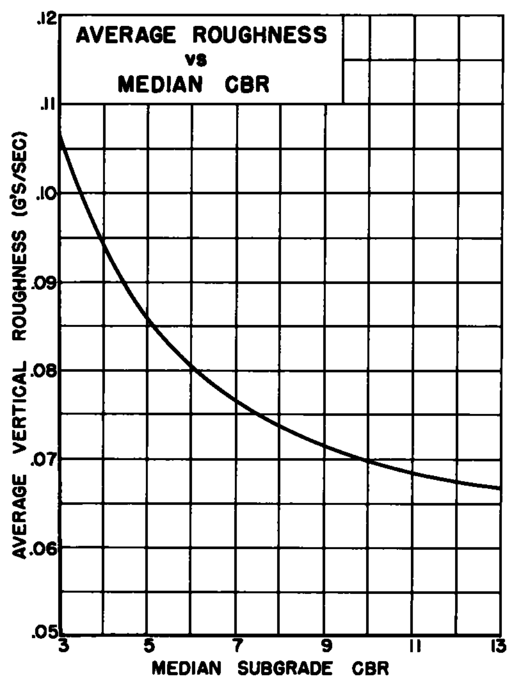


Figure 12. Generalized apparent relationship between average roughness values and median CBR of the subgrades.

Pavement Deflections

In the late summer and early fall of 1957, Benkelman Beam measurements were made at 50 locations on 20 projects. Deflections were measured in both the outside and inside wheel tracks under an 18,000-lb axle load on dual tires. In order to evaluate the seasonal effect, deflection measurements were made again

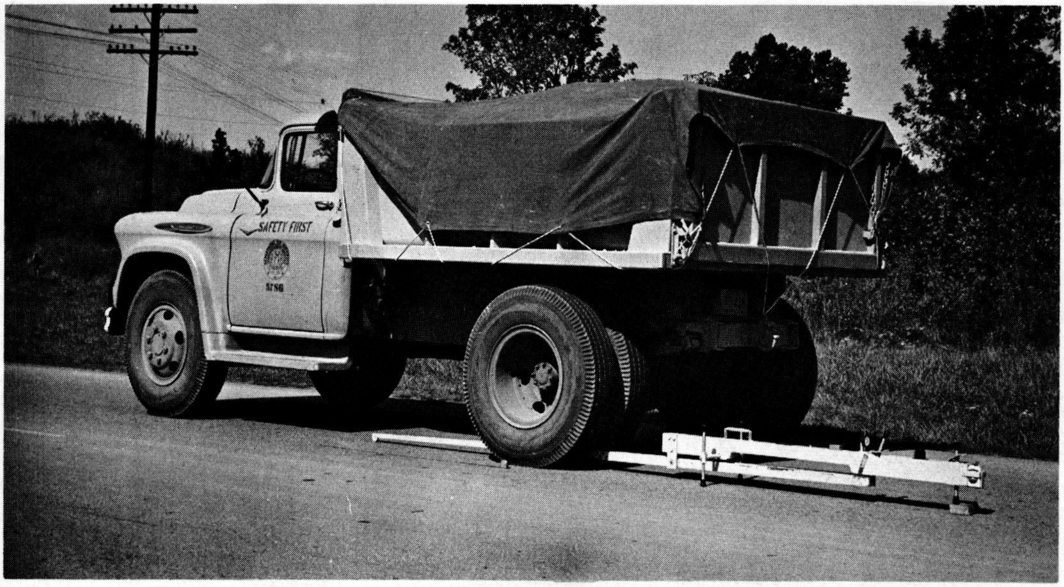


Figure 13. Benkelman Beam in-place for measuring pavement deflection under 18,000-lb single axle.

under the same conditions of loading in the spring of 1958 at the same locations previously visited as well as an additional 18 locations representing 11 other projects.

To obtain deflection readings, the probe beam was placed between the dual tires of the test vehicle so that the foot of the beam rested on the pavement 5 ft ahead of the axle (see Fig. 13). The reference beam then rested on the pavement well back of the influence of the loaded wheels. As the test vehicle moved forward at creep speed, the probe foot deflected with the pavement, and the amount of deflection was read from an Ames dial. At each location, measurements were made until two consecutive readings were in agreement.

Also, in 1958, deflection measurements were made under a tandem axle loading of 32,000 lb at 8 locations on 5 projects. Two of these locations were also loaded with a 36,000-lb tandem axle load and deflection measurements recorded.

Since the length of the probe beam on the Benkelman Beam was designed for obtaining deflection measurements under single axles, modifications in the method of measuring were necessary. The probe beam was placed between the dual tires so that the foot rested on the pavement beneath the front axle (see Fig. 14). As the test vehicle moved ahead, the partial rebound between axles was noted, then the deflection was read as the rear axle passed the probe foot, and finally the complete rebound was read as the loaded vehicle moved well away from the setup.

Most of the measured deflections occurred over about a 3-ft span. Deflection increased slowly until the wheel was within 12 to 18 in. of the probe and then increased rapidly to the maximum. This condition was more pronounced in the thinner pavements. Thick pavements and pavements having one or more courses of bituminous base deflected less and over a wider area of pavement surface. Least deflections were measured on pavements made up of full depth bituminous concrete. For the fall series of measurements, pavements with standard waterbound macadam bases averaged 0.028 in. of deflection. Minimum design pavements having one 3- or $3\frac{1}{2}$-in. thickness of waterbound base averaged 0.048 in. of deflection while pavements having 3 in. or more of bituminous base averaged 0.015-in. deflection.

For the spring 1958 series of measurements, pavements with waterbound bases averaged 0.033-in. deflection, the minimum design pavements with waterbound bases averaged 0.078-in. deflection, and pavements with bituminous bases averaged 0.016 in. This increase in deflection during the spring was expected since the influence of sub-

Figure 14. Benkelman Beam in-place for measuring pavement deflection under 32,000- or 36,000-lb loads on tandem axles.

grade moisture would logically be greater during this period. The three locations measured during both periods on pavements having their total depth made up of bituminous concrete averaged 0.003 in. less deflection for the spring measurements. This decrease might be attributed to the lower temperatures at the time of spring testing.

The average difference between inner wheel track and outer wheel track deflections for the fall measurements was only 0.0003 in.; however, the outer wheel tracks averaged 0.005 in. more deflection than the inner wheel tracks during the spring measurements. This was probably due to greater susceptibility of the outer portion of the subgrade to climatic changes.

Four of the locations measured under 32,000-lb tandem axle loadings were over waterbound bases, one was over combined waterbound and bituminous base, and three over full depth bituminous bases. At creep speed over waterbound base, the tandem wheels acted independently. Rebound between the wheels was about one-half the maximum deflection and maximum deflection was 15.8 percent less than for the 18,000-lb single axle. For the combined waterbound and bituminous bases, rebound was less than one-half the maximum deflection and maximum deflection was 15.3 percent less than for the 18,000-lb single axle. For full depth bituminous bases, the tandem axles acted as a unit with no appreciable rebound between wheels. Maximum deflections for the 36,000-lb tandem and 18,000-lb single axle were equal. The lack of rebound between tandem wheels demonstrates the slab or beam action of bituminous concrete under the test conditions.

Deflections under a 36,000-lb tandem axle loading were measured at two locations over a combined waterbound and dense-graded aggregate base. Rebound between the wheels was more than half the maximum deflection, and the maximum deflection was approximately equal to the maximum deflection under an 18,000-lb single axle.

A plot of deflections according to traffic groups, with all points marked to distinguish between satisfactory or unsatisfactory pavements, is shown in Figure 15. Pavements marked unsatisfactory were showing patching or cracking at or near the point measured. The curve best separating satisfactory and unsatisfactory pavements implies a maximum deflection that can be tolerated by pavements in each traffic group. Deflection values were subsequently interpolated from this curve and plotted semi-logarithmically against the mid-points of the corresponding EWL group. Thus, Figure 16 relates permissible deflections with EWL's. Independently of this apparent relationship, deflections taken in the spring of 1958 were plotted against the corresponding thicknesses of pavements that were adjudged satisfactory (Fig. 17). Spring measurements were used here in order to eliminate seasonal influences, and only satisfactory pavements were used in order to eliminate exaggerated deflections due to failed or weakened pavements. Here, also, a best-fitting curve was drawn, and a relationship between deflections and thicknesses appears to exist. Assuming these two relationships to be valid, to the extent that whatever hidden variables may be involved are either of minor influence or else vary only slightly, thicknesses and EWL's corresponding to the same limiting deflections were interpolated from Figures 16 and 17 and were plotted as shown in Figure 18.

According to Figure 18, pavement thicknesses should be increased in proportion to

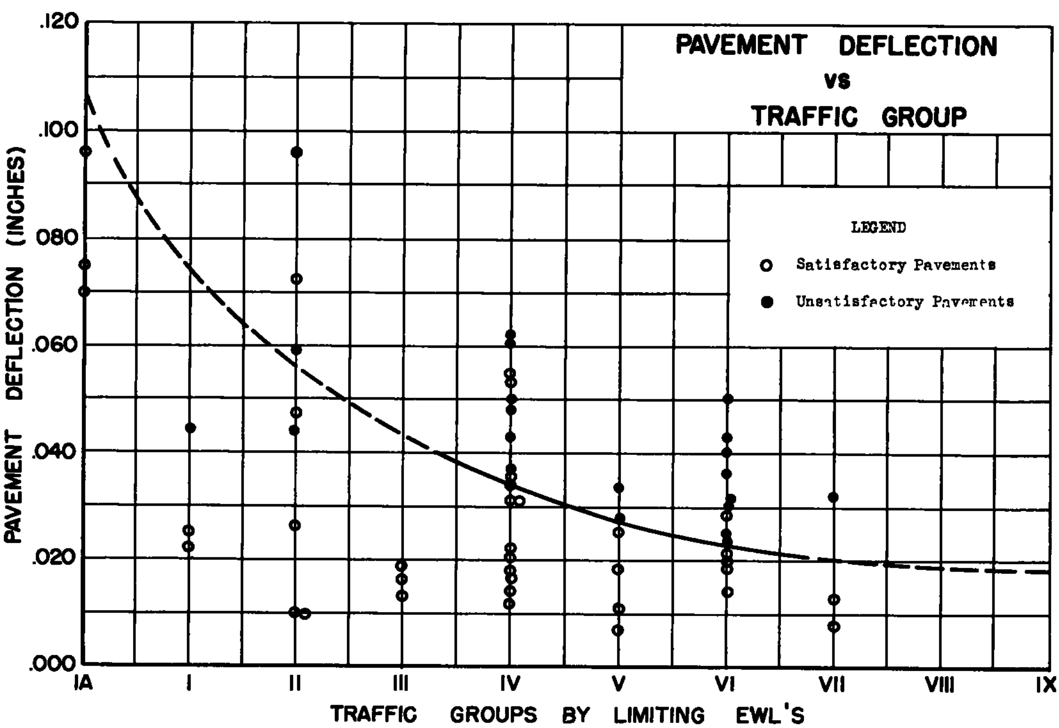


Figure 15. Pavement deflections, 18,000-lb single axle, obtained from both satisfactory and unsatisfactory pavements plotted according to traffic groups corresponding to accumulated EWL's.

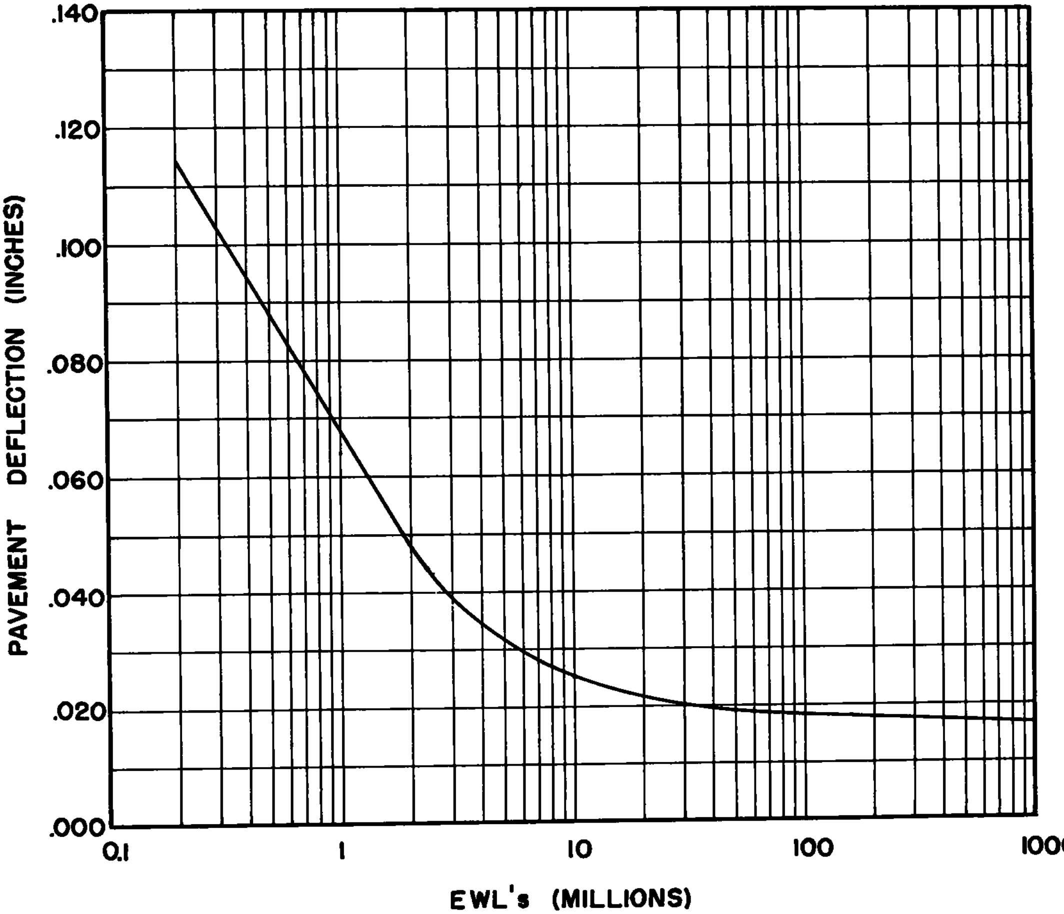
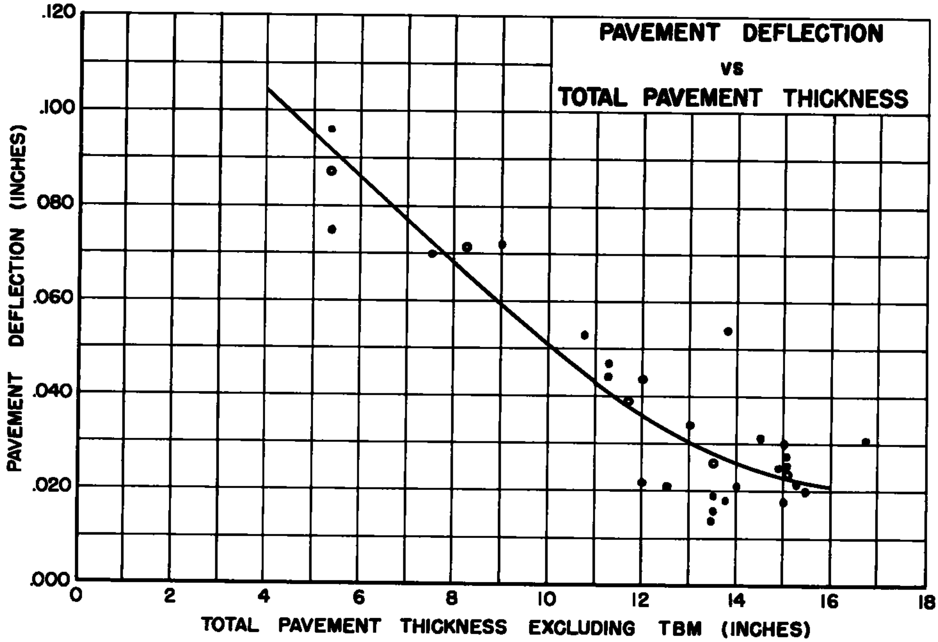


Figure 16. Pavement deflections as obtained in Figure 15, plotted according to the logarithm of the mid-point value of the respective EWL groups.

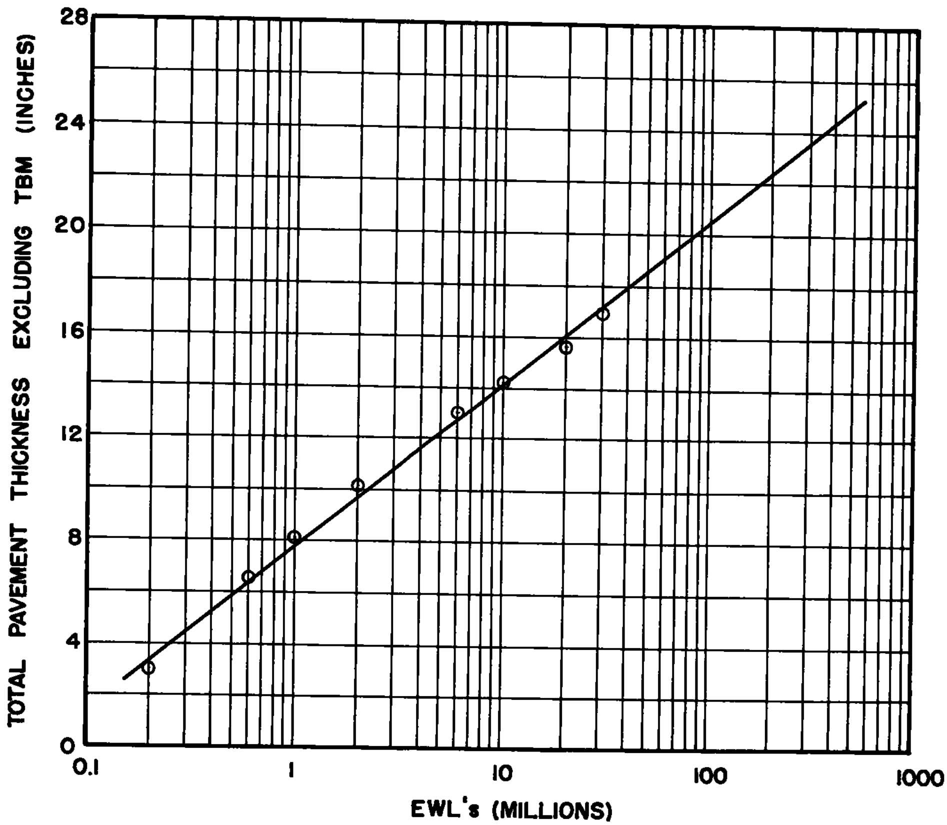
the logarithm of the EWL's. This relationship appears to have been derived more-or-less independently of any parameter describing subgrade support. However, it is rather evident, since each pavement involved in the derivation was originally designed on the basis of a subgrade support parameter, that Figure 17 must reflect a modal or prevailing subgrade CBR. Otherwise, the curve could not have been drawn. Therefore, while the relationship between thickness and log EWL may be of a general nature the plot itself would be significant only with respect to a particular CBR value which, in this case, should be very close to the average or median value of the group of roads involved or of the entire series.

To test the logic employed here, a cursory analysis of the frequency and distributio of project median CBR's was made; and it was found that 90 percent of the CBR values from all data available fell within the range of 3 to 11. Within this range, the arithmetic mean was 7.1, and the average deviation was only 1.7. Thus, the assumption of a strong central tendency in CBR's seemed proper.

Taking 7.1 as the value most likely associated with Figure 18, thicknesses for each of the EWL groups were interpolated from Figure 18 and replotted at CBR 7.1 on the original design chart as shown in Figure 19. Here the points tend to favor somewhat greater thicknesses than were required by the original curves. However, considering the fact that these points were derived on the basis of satisfactory pavements only (Fig. 17), the points would naturally reflect safe design thicknesses but not necessaril



igure 17. Pavement deflections representing only satisfactory pavements plotted according to corresponding pavement thicknesses. The curve implies that deflections of equal or lesser magnitude would be within safe limits.



gure 18. Plot of thicknesses and EWL's interpolated from Figures 16 and 17 for corresponding deflections.

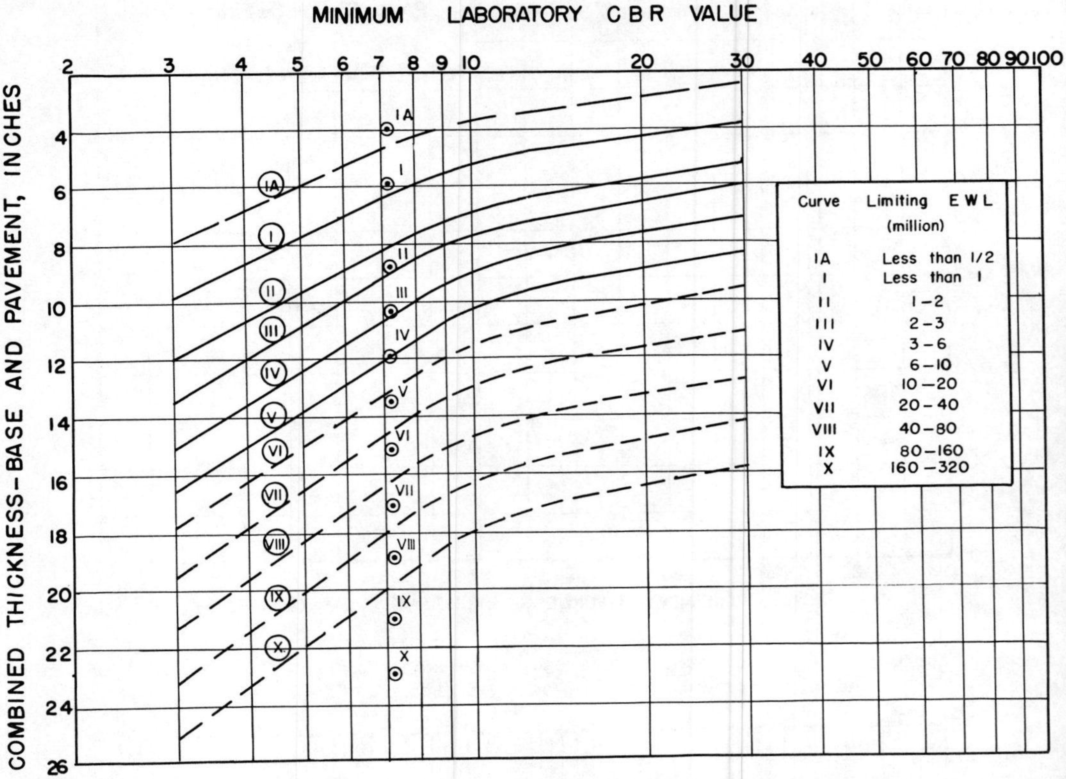


Figure 19. Present design chart showing thicknesses for each median point of the traffic volume groups, taken from Figure 18, plotted at an average CBR of 7.1.

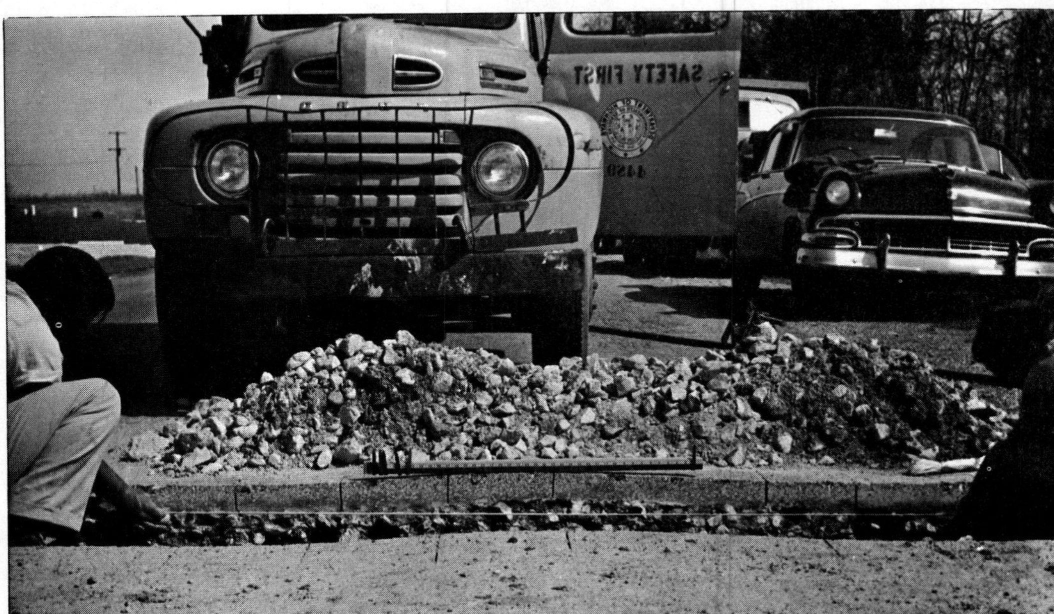


Figure 20. Exposed cross-section of a rutted pavement.

the minimum design thicknesses. In any case, the derivation of these points provides a rather unique independent check upon the original curves as well as the revisions previously indicated in Figure 8.

Pavement Openings

In order to investigate the extent to which rutting, evident at the surface, penetrated the different layers of the pavement, eight locations on seven projects were opened to expose a cross-section to full view. An eighth pavement not originally scheduled for study was opened (in Bullitt County) in order to examine the performance of a different type of granular base material with regard to subgrade infiltration. The Bullitt County base was dense-graded aggregate (DGA).

To open the pavements, a pavement saw with an 18-in. diamond blade was used. An opening approximately 30 in. wide was made across the full width of a traffic lane. The saw was used to cut through the top layers of the pavement while the granular base materials were carefully removed by hand so that the layers could be separated and studied. Samples were obtained from the bituminous layers and returned to the laboratory for density determinations (by weighing in air and in water). These samples were taken from the wheel tracks as well as from between the wheel tracks. In-place density tests were made on the different layers of granular base by the calibrated sand method. Subgrade densities were obtained by both the rubber balloon method and the sand method. Sufficient measurements were made so that the extent of rutting in most of the pavement components could be noted (see section on Rutting).

Disturbed samples from the layers of granular base and from the subgrade were

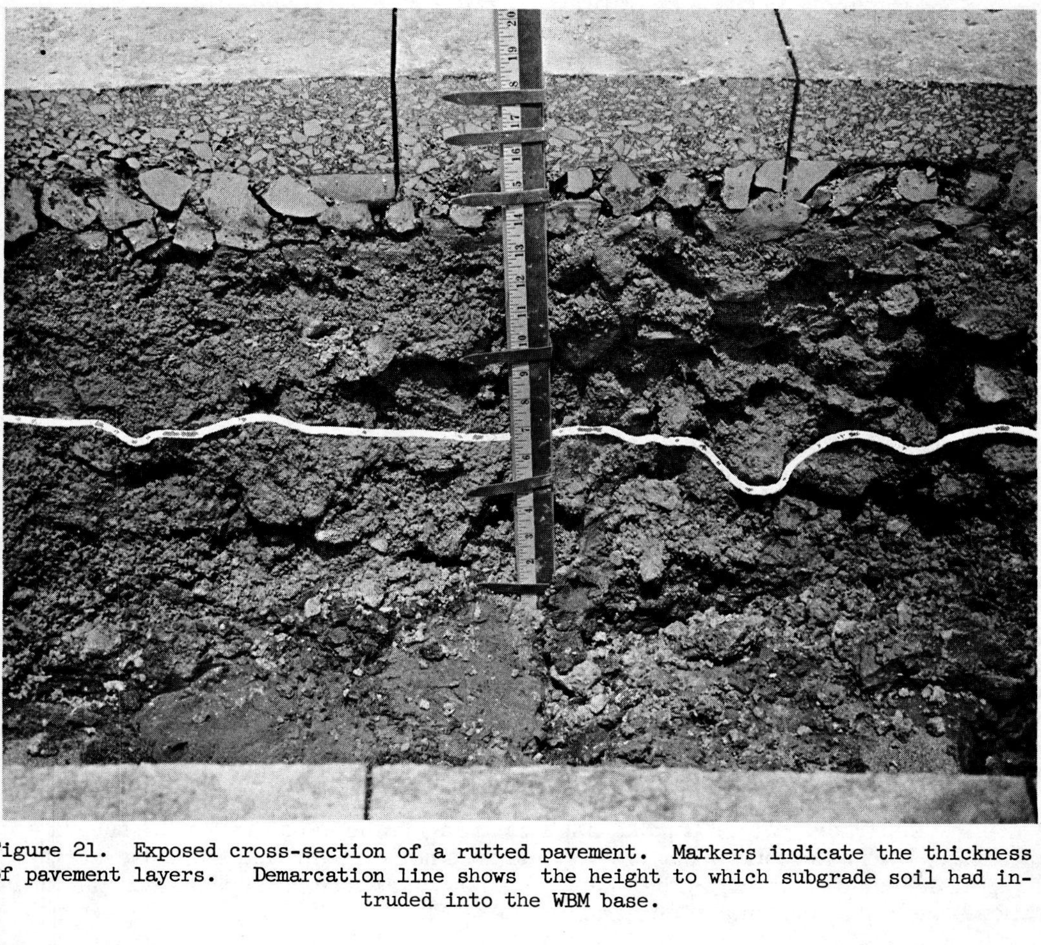


Figure 21. Exposed cross-section of a rutted pavement. Markers indicate the thickness of pavement layers. Demarcation line shows the height to which subgrade soil had intruded into the WBM base.

returned to the laboratory for other testing. It may be noted that no significant difference in density occurred between samples taken from the wheel tracks and those taken between the wheel tracks. This was particularly true of the surface and binder courses but less so of the lower portions of the pavement.

It was observed that much subgrade material had penetrated the WBM base courses as much as 10 in. in some places (see Figs. 20 and 21). This indicates that the insulation or subbase courses normally used in waterbound base construction in Kentucky has not performed properly and is not fulfilling its intended function, which is to protect the WBM courses from infiltration of soil and subgrade material. Observations made in this investigation indicate that soil in the WBM courses is a result of improper rolling during construction or as a result of traffic action. In those instances where the penetration of soil was rather uniform across the section, infiltration appears to have been caused by construction rolling while the subgrade was wet. In other instances greater penetration within the wheel tracks indicates that the clay or soil was forced up by traffic. Naturally, some loss of strength of the affected WBM courses would be expected; however, the degree of this loss and its equivalent in terms of reduced pavement thickness could not be determined.

SUMMARY

This re-evaluation of the Kentucky flexible pavement design criterion has emphasized some recognized shortcomings of pavement design systems in general and has further clarified some opinions concerning needed revisions in the present flexible pavement design.

Traffic evaluation based upon summations of equivalent-wheel loads does take into account both volumes and weights of traffic. The projected service-life of a flexible pavement designed by this method is dependent upon the accuracy of the traffic projections. The original 10-yr basis of predicting traffic has been revised to a 20-yr basis, and the report indicates that the 20-yr traffic projections may be reasonably valid. The average value for each volume system analyzed is close to the projected traffic value.

The need for an adequate method of rating pavement performance is recognized. The four methods used here are advocated only as being a combination that can be used. The visual rating while probably the oldest and soundest method is usually open to more criticism than some of the others. Visual ratings were the basis for selection of locations for load deflection measurements and pavement openings. Design curves for two traffic volume groups were prepared from the visual performance ratings.

The roughness measurements taken by the triaxial acceleration method, though difficult to analyze on a project basis, undoubtedly have basic significance with regard to over-all pavement adequacy. The data appear to correlate with the visual performance rating.

Load-deflection measurements were used in the analysis of adequate pavement thickness for average subgrade support on various traffic volume groups. Those points indicated a need for revision of thickness.

Pavement openings were used to examine the layered system of selected rutted pavements. The openings permitted the determination of the extent of rutting in each layer of the pavement. The majority of the pavements studied were constructed using waterbound macadam base and 7 of the 9 locations opened were constructed with layers of WBM base. Of the pavements opened, it was noted that 72 percent of rutting was confined to the layers of WBM base while 4 percent was localized in the bituminous courses. Only 24 percent of the rutting penetrated the pavement structure to the subgrade. It appears that one of the greatest shortcomings of WBM type base is its susceptibility to subgrade infiltration. Clay subgrades tend to fill the voids in the base and to lubricate the stone and cause rutting.

Clay subgrade can be forced into the base during construction by extensive rolling over a wet subgrade. Water bonding itself can provide the moisture for the subgrade softening. Where the infiltration of subgrade does not vary through the cross-section and is at the same elevation in the wheel tracks as elsewhere, it appears that the infiltration occurred at the time of construction.

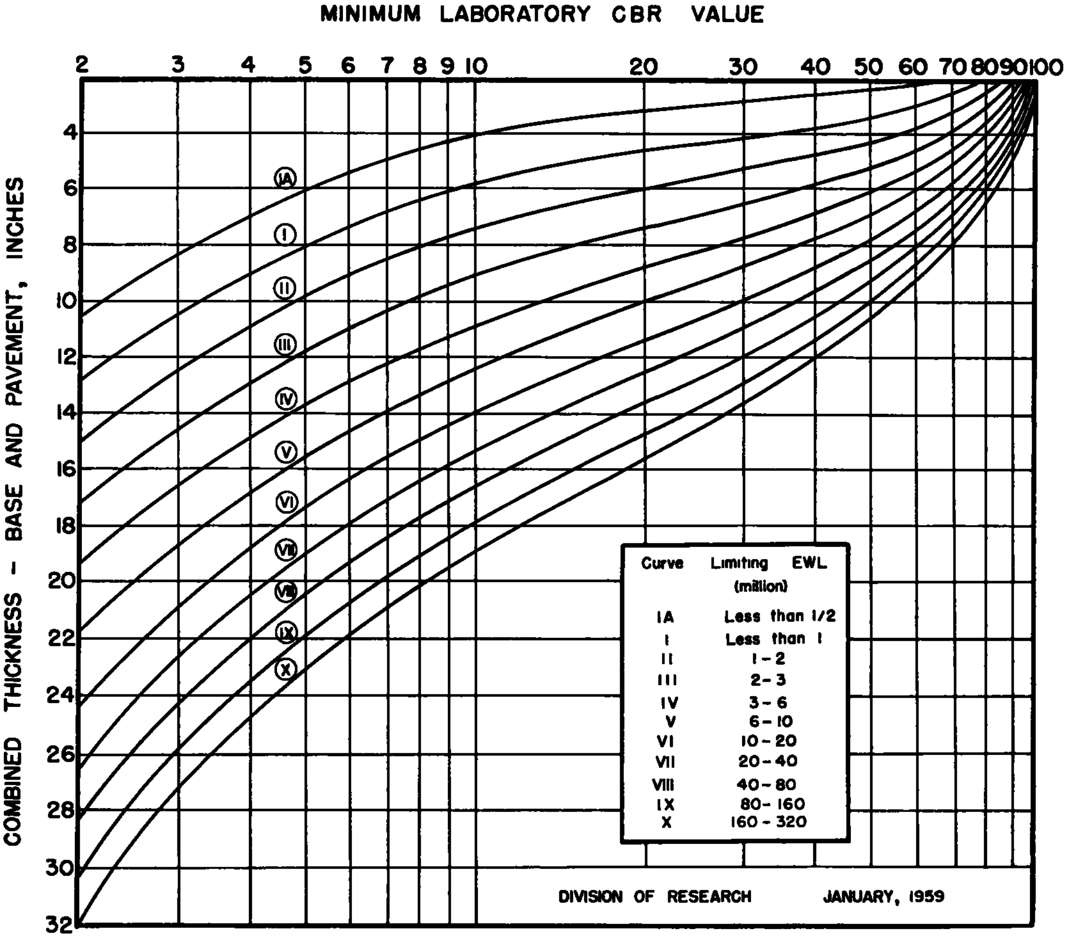


Figure 22. Revised flexible pavement design curves.

Traffic can pump subgrade soil into the voids of WBM. If traffic is the motivating force, the height of infiltration would normally be greater in the wheel tracks. In the majority of the locations opened the infiltration was to a uniform elevation, and it is deduced that the subgrade soil was rolled into the base by construction equipment.

Dense-graded aggregate base is less susceptible to damage from subgrade infiltration and lubrication. Present Kentucky specifications require the moisture to be added to the stone in a plant-mix operation, thereby eliminating the possibility of over-wetting the subgrade at that time. Dense-graded aggregate type base having considerably less voids than the average waterbound macadam is a much better insulation against subgrade infiltration.

The flexible pavement design curves shown in Figure 22 represent the combination of the data from the 1948 study, revisions to 1957, and the results of the various approaches presented in the present investigation. These curves require a somewhat greater total thickness of pavement in the lower CBR range. The curves have been extended to a CBR value of 2, primarily to emphasize the need for subgrade improvement or stabilization of soils with CBR values of less than 3. It is still recommended that soils with CBR of less than 3 not be used for subgrade. The curves have been extended to CBR 100 to permit the use of the curves for subbase or local granular materials. The thicknesses have been reduced for CBR values of over 20.

Present Departmental policies regarding the types of base materials and relative course thicknesses for the various highway systems appear to be sound.

ACKNOWLEDGMENTS

The authors wish to acknowledge the cooperation and assistance of many individuals in the Kentucky Department of Highways: D.H. Bray, State Highway Engineer; A.O. Neiser, Assistant State Highway Engineer, who through his special interest in pavement design requested that the study be made and suggested specific approaches that should be investigated; George D. Aaron, Acting Director, J.R. Pulliam, Engineer of Traffic Surveys, J.J. Constantine, Head of the Statistical Section, Division of Planning J.A. Spears, Director of Maintenance; W.B. Carrington, Director, and S.T. Collier, Assistant Director, Division of Design; and J.A. Bitterman, Director, and O.R. Threlkeld, Soils Engineer, Division of Materials. The cooperation of the District Engineers and their Maintenance Assistants aided materially in the field operations.

Robert C. Deen, Research Engineer and Head of the Soils Section, presently on educational leave of absence, was directly responsible for assembling the design data, the soil testing, and analysis and some phases of the performance survey.

Loren H. Strunk, formerly Research Engineer and Head of the Bituminous Section, supervised the laboratory testing of base materials and conducted certain phases of the performance surveys.

REFERENCES

1. Grumm, Fred J., "Designing Foundation Courses for Highway Pavements and Surfaces." California Highways and Public Works (Nov. 1941 and March 1942).
2. Baker, R.F., and Drake, W.B., "Investigation of Field and Laboratory Methods for Evaluating Subgrade Support in the Design of Highway Flexible Pavement Bulletin No. 13, Engineering Experiment Station, University of Kentucky (Sept. 1949) and Proceedings, Highway Research Board, Vol. 28 (1948).
3. Gornberg, G.D., and Blosser, N.B., "Lives of Highway Surfaces—Half Century Trends." Public Roads, Vol. 29 (June 1956).
4. Carter, Jas. Lee, Jr., "A Study of the Service Life of Pavements in Kentucky." Master's Thesis, College of Engineering, University of Kentucky, unpublished (1955).
5. Gregg, L.E., and Foy, W.S., "Triaxial Acceleration Analysis Applied to the Evaluation of Pavement Riding Qualities." Proceedings, Highway Research Board, Vol. 34 (1955).

Discussion

S.M. FERGUS, Standard Oil Company—Messrs. Drake and Havens have prepared an excellent paper setting forth the design method for flexible pavement presently in use by the Kentucky Department of Highways and have presented a re-evaluation of that method based on a statistical study of actual pavement performance. As a result of the study, it is suggested that the present curves be redrawn to provide slightly greate thicknesses.

This paper should be of great interest to highway engineers everywhere since it provides usable data for design evaluation which takes into consideration not only the axle loads but also the number of axles using the pavement. In this respect it is unfortunat that the curves are designated for Equivalent Wheel Loads (EWL's) only, rather than i terms of particular axle loads.

A study of the data reveals that inherent in the method is the assumption that a computable relation exists between a certain number of coverages with one axle load and a different number of coverages with another axle load. This must be so since different combinations of axle load and vehicle number yield the same value for EWL. It can be shown that this relation is given by the equation

$$N_1 (2)^{P_1} = N_2 (2)^{P_2} \tag{1}$$

where N_1 and N_2 are the numbers of axles carrying respectively, the axle loads (in ton P_1 and P_2. In a given case, assuming that values for P_1, P_2, and N_2 are known, the equivalent number of coverages N_1 can be computed from the expression

$$N_1 = N_2(2)^{P_2 - P_1} \quad (2)$$

As an example, assume it is desired to know what effect one coverage with a 10-ton axle load would have in terms of a greater number N_1 of coverages with a 5-ton axle load. From Eq. 2

$$N_1 = (1)(2)^{10 - 5} = 32$$

Thus it is shown that one 10-ton axle load is assumed to produce as much wear on the pavement as 32 5-ton axle loads.

It is also evident that the California factor (f) used in the Kentucky design system may be computed from the expression

$$f = (2)^{P - 5} \quad (3)$$

and that the EWL values for each class (2 directions) may be computed from

$$EWL = 2,000,000(2)^{P - 5} \quad (4)$$

The relationships between these various values are shown in the following tabulation

Axle Load		California[1] Factor	EWL[2] Millions	EWL Class
Tons = P	Pounds			
2	4,000	1/8	1/4	IA
3	6,000	1/4	1/2	I
4	8,000	1/2	1	II
5	10,000	1	2	III
6	12,000	2	4	IV
7	14,000	4	8	V
8	16,000	8	16	VI
9	18,000	16	32	VII
10	20,000	32	64	VIII
11	22,000	64	128	IX
12	24,000	128	256	X

[1] Computed from Eq. 3.
[2] Computed from Eq. 4.

It is thus shown that in each class the pavement is designed to accommodate 2,000,000 (2 directions) axle loads of the indicated weight or their equivalent in axle loads of other weights. If the Kentucky curves were to be designated by axle loads rather than by EWL's, it would be necessary for design purposes to determine the equivalent design load rather than the total EWL. This could be accomplished by means of the equation

$$N_1(2)^{P_1 - 5} + N_2(2)^{P_2 - 5} + N_3(2)^{P_3 - 5} + \text{etc.} = 2,000,000(2)^{P_d - 5} \quad (5)$$

Assuming that values for N_1, N_2, N_3, etc. and P_1, P_2, P_3, etc. are known, the value for P_d can be determined. As a numerical example, take the case presented in Figure 3 of the subject paper. Taking values for N from column (6) and for P from column (1), the following equation is obtained

$$2,000,000(2)^{P_d - 5} = (416,534)(1) + (378,682)(2) + (478,714)(4)$$
$$(434,060)(8) + (448,224)(16) + (128,922)(32)$$
$$(12,644)(64)$$

$$(2)^{P_d - 5} = \frac{18,667,538}{2,000,000} = 9.33$$

$$(P_d - 5)\log(2) = \log 9.33$$

$$(P_d - 5) = \frac{\log (9.33)}{\log (2)} = 3.22$$

$$P_d = 8.22 \text{ tons} = 16{,}440 \text{ lb}$$

The indicated curve is for an axle load of 16,000 lb or curve VI. This is the same as for an EWL of 18,667,538 as given in Figure 3.

For multiple-lane highways, in which the outer lane traffic is more numerous and heavier than that in the passing lane, it may become desirable to express the vehicle frequency in terms of axles per lane per day. In this case, Eq. 5 would become

$$n_1 (2)^{P_1 - 5} + n_2 (2)^{P_2 - 5} + n_3 (2)^{P_3 - 5} + \text{etc.} = \frac{2{,}000{,}000(2)^{P_d - 5}}{(2)\ (365)\ (y)}$$

where the lower case n's indicate that the frequency is in axles per lane per day and where y indicates the design life of the pavement in years.

Taking y = 10 and using again the values from Figure 3, but dividing each value by 7,300 which is the product of 2 times 365 times 10, the computations are

$$274(2)^{P_d - 5} = (57)(1) + (52)(2) + (66)(4) + (59)(8) + (61)(16) + (18)(32) + (2)(64)$$

$$(2)^{P_d - 5} = \frac{2{,}577}{274} = 9.4$$

$$P_d - 5 = \frac{\text{Log } 9.4}{\text{Log } 2.0} = \frac{0.97313}{0.30103} = 3.23$$

$$P_d = 8.23 \text{ tons} = 16{,}460 \text{ lb}$$

this checks the design axle load computed from Eq. 5.

Where the design life is 20 years, the computations are the same except that the values are all divided by 14,600 which is the product of 2 times 365 times 20. Taking the values from column F of Figure 4, the computations are

$$137(2)^{P_d - 5} = (60)(1) + (55)(2) + (56)(4) + (53)(8) + (48)(16) + (15)(32) + (2)(64)$$

$$(2)^{P_d - 5} = \frac{2{,}194}{137} = 16.0$$

$$P_d - 5 = \frac{\text{Log} 16.0}{\text{Log } 2.0} = \frac{1.20412}{0.30103} = 4.0$$

$$P_d = 9.0 \text{ tons} = 18{,}000 \text{ lb}$$

The design load of 18,000 lb indicates Curve VII which is the same as for an EWL of 31,691,311, the value shown in Figure 4.

It may seem only an academic question whether the basis of design is in terms of EWL's or axle loads but since it is desirable that knowledge accumulated by engineers in one district should be readily understood by those in other districts and since most methods for determining the design thickness of flexible pavements are based on loading considerations, it would seem desirable to include that factor in Kentucky's curves also.

In any case, a vote of approval should go to the authors and to the Kentucky Department of Highways for an important contribution to flexible pavement knowledge.

W. B. DRAKE and JAMES H. HAVENS, Closure—The authors are grateful to Mr. Fergus for an enlightening and scholarly development which appears to enhance the significance of the EWL method of evaluating traffic as well as the axle load method. Since it is now possible to relate each to the other, interesting comparisons may be made among various design criteria, particularly those involving CBR and EWL's or axle loads as the independent parameters.

There is a slight inconsistency in the progression of EWL's used in the Kentucky design chart. However, Mr. Fergus has rounded this out in his calculations; and, for all practical purposes it seems that the progressions of both the California factor and EWL's should be considered as being truly geometrical.

THE NATIONAL ACADEMY OF SCIENCES—NATIONAL RESEARCH COUNCIL is a private, nonprofit organization of scientists, dedicated to the furtherance of science and to its use for the general welfare. The ACADEMY itself was established in 1863 under a congressional charter signed by President Lincoln. Empowered to provide for all activities appropriate to academies of science, it was also required by its charter to act as an adviser to the federal government in scientific matters. This provision accounts for the close ties that have always existed between the ACADEMY and the government, although the ACADEMY is not a governmental agency.

The NATIONAL RESEARCH COUNCIL was established by the ACADEMY in 1916, at the request of President Wilson, to enable scientists generally to associate their efforts with those of the limited membership of the ACADEMY in service to the nation, to society, and to science at home and abroad. Members of the NATIONAL RESEARCH COUNCIL receive their appointments from the president of the ACADEMY. They include representatives nominated by the major scientific and technical societies, representatives of the federal government, and a number of members at large. In addition, several thousand scientists and engineers take part in the activities of the research council through membership on its various boards and committees.

Receiving funds from both public and private sources, by contribution, grant, or contract, the ACADEMY and its RESEARCH COUNCIL thus work to stimulate research and its applications, to survey the broad possibilities of science, to promote effective utilization of the scientific and technical resources of the country, to serve the government, and to further the general interests of science.

The HIGHWAY RESEARCH BOARD was organized November 11, 1920, as an agency of the Division of Engineering and Industrial Research, one of the eight functional divisions of the NATIONAL RESEARCH COUNCIL. The BOARD is a cooperative organization of the highway technologists of America operating under the auspices of the ACADEMY–COUNCIL and with the support of the several highway departments, the Bureau of Public Roads, and many other organizations interested in the development of highway transportation. The purposes of the BOARD are to encourage research and to provide a national clearinghouse and correlation service for research activities and information on highway administration and technology.