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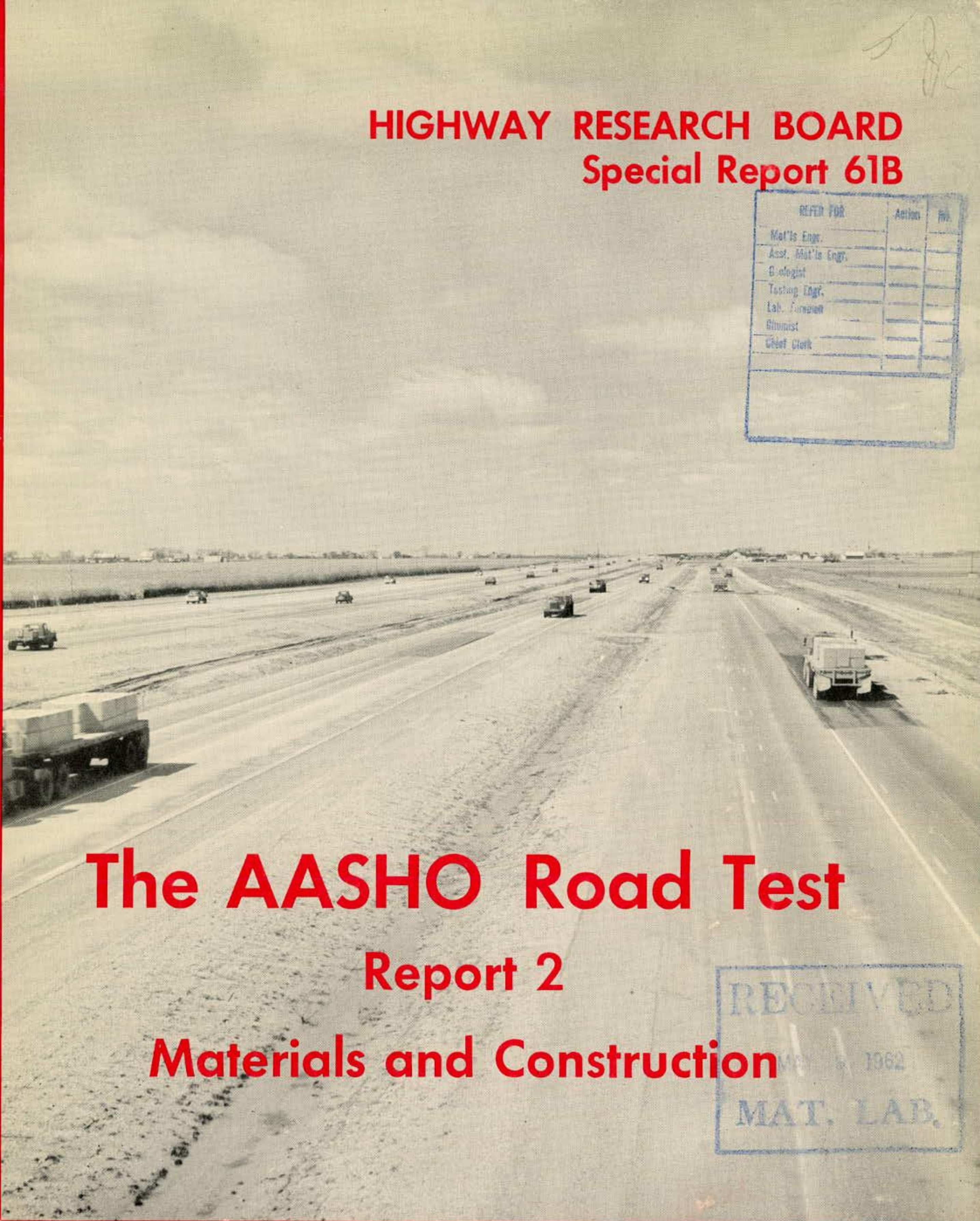
HIGHWAY RESEARCH BOARD
Special Report 61B



The AASHO Road Test
Report 2
Materials and Construction

National Academy of Sciences—
National Research Council

publication 951

The AASHO Road Test

Report 2
Materials and Construction

By the

HIGHWAY RESEARCH BOARD

of the

NAS-NRC Division of Engineering and Industrial Research

Special Report 61B

Publication No. 951
National Academy of Sciences—National Research Council
Washington, D.C.
1962

This is one of a series of reports of work done under a fiscal agreement of June 10, 1955, between the National Academy of Sciences and the Bureau of Public Roads relating to AASHO Road Test Project; and under individual agreements covering Cooperative Highway Research Project (AASHO Road Test) made between the National Academy of Sciences and the several participating state highway departments, members of the American Association of State Highway Officials.

Included in the series are the following reports:

Report	*Subject*	*HRB Special Report No.*
1	History and Description of Project	61A
2	Materials and Construction	61B
3	Traffic Operations and Pavement Maintenance	61C
4	Bridge Research	61D
5	Pavement Research	61E
6	Special Studies	61F
7	Final Summary	61G

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Preface

The AASHO Road Test was conceived and sponsored by the American Association of State Highway Officials as a study of the performance of pavement and bridge structures of known characteristics under moving loads of known magnitude and frequency. It was administered by the Highway Research Board of the National Academy of Sciences—National Research Council, and was considerably larger and more comprehensive than any previous highway research study.

This is the second of a series of major reports on the AASHO Road Test. The first report covered the history and description of the project. This report is a comprehensive description of the materials and construction of the test facilities, and includes summaries of data relating to materials and construction control and the characteristics of the completed pavements and bridge structures. It precedes the research reports on flexible and rigid pavements and test bridges. A brief summarization describing the construction procedures, materials control testing, and as-built characteristics of the test pavements and bridges is included in AASHO Road Test Report 7, the final summary report.

This report is presented in seven chapters. The first is an introduction including a review of the history and description of the project, a summary of the construction, and a description of the test facilities. Subsequent chapters contain information about the materials and construction of the embankment, subbase, base, surfacing, and test bridges.

Although many of the data supporting the observations and conclusions are included in the AASHO Road Test reports, it was not practicable to include all the data. A comprehensive catalog of data files is available from the Highway Research Board. Many of the data systems are in the form of IBM listings that may be purchased from the Board and others may be examined and reproduced at the Board offices in Washington.

Acknowledgments

Personnel from many organizations and industrial firms assisted in carrying out the construction of the test facilities at the AASHO Road Test. It is impractical to list in this report the names of all individuals who participated. However, the efforts of the following are particularly acknowledged:

The Bureau of Public Roads of the U. S. Department of Commerce, for technical advice and services in a great many areas; and E. G. Yemington, formerly Highway Physical Research Engineer, for assistance in establishing the testing procedure for embankment construction.

The Illinois Division of Highways, for its outstanding performance as the host department and, in particular, R. R. Bartelsmeyer, Chief Highway Engineer; John Grayhack, Jr., District Engineer, District 3; and W. E. Chastain, Sr., Engineer of Physical Research, Bureau of Research and Planning. Also the following Illinois personnel for extensive services in many fields, including preparation of the material and construction specifications and material control: H. J. Alton, Assistant Engineer of Design, Bureau of Design; E. D. Antrobus, Assistant District Engineer of Materials, District 3; J. E. Burke, Assistant Engineer of Physical Research, Bureau of Research and Planning; C. E. Cullen, District Engineer of Materials, District 3; O. A. Evans, Assistant District Engineer of Right-of-Way, District 3; Dolph Hoke, District Soils Engineer, District 3; J. D. Lindsay, Engineer of Materials, Bureau of Materials; and L. C. Reime, formerly Assistant Engineer of Construction, Bureau of Construction.

The Bureau of Public Roads and the highway departments of the following States for loan of personnel during construction: Illinois, Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, Ohio, Oklahoma and Wisconsin. A complete listing of personnel serving on committees, panels, project staff, and on loan from the States is given in Appendix F.

The Arctic Construction and Frost Effects Laboratory (ACFEL) of the U. S. Army, New England Division, Corps of Engineers, for the study on frost susceptibility of the A–6 embankment soil.

The Portland Cement Association, The Asphalt Institute and the several States for participation in materials testing programs.

The following contractors and J. F. Healey, Project Manager for the grading and paving, for their cooperation and extraordinary efforts in constructing the test facilities to meet the exacting requirements dictated by the research: S. J. Groves and Sons Company; Arcole Midwest Corporation; Valley Builders, Incorporated; and Rock Roads Construction Company.

The many organizations, agencies, firms, and their personnel, for extended efforts in producing and manufacturing materials and equipment specifically for the construction of the test facilities.

The following organizations, for the services of resident observer consultants: The Asphalt Institute, the Portland Cement Association, the American Trucking Associations, the Canadian Good Roads Association, and the Department of Highways, Province of Ontario, Canada.

Definitions of Terms

The following definitions apply to the terms as used in this report:

1. *A–6 soil embankment.* The upper three feet of embankment on the test tangents constructed of the selected yellow-brown and the gray clay A–6 soils. The characteristics of these soils are given in the report.
2. *Earth subgrade.* The subgraded surface of the top of the embankment.
3. *Subgrade.* Any surface prepared for the placement of the subbase, base course, or surfacing.
4. *Sand-gravel mulch (mulch).* The portion of the subbase material placed over the completed embankments during the time it served as a protective cover. Its purpose was to prevent excessive drying of the embankment soil and to assist in bringing the moisture condition of the earth subgrade to a level near that found under pavements which have been in place for several years. This material was later used in-place as part of the subbase for the test pavements.
5. *Data system numbers.* The data obtained at the Road Test have been assigned four-digit data system numbers for purposes of cataloging and filing. The results of the tests in each data system have been punched on IBM cards or placed on summary sheets for ease of reproduction. The first two digits of each number of the systems that apply to the materials and construction identify the data as follows:

   21-- pertains to flexible pavement tangents
   22-- pertains to rigid pavement tangents
   23-- pertains to both tangents
   24-- pertains to test bridges.

   The numbers of the systems containing the data from which summaries have been prepared are given with the corresponding tables and figures included in the report. A list of data systems pertaining to pavement research is given in Appendix I, AASHO Road Test Report 5; a list pertaining to bridge research is given in Appendix A, AASHO Road Test Report 4.
6. *Mean values.* Mean values included in the tables and figures are arithmetic means.
7. *Standard deviation.* A measure of average scatter from the mean for any set of values. It is a root mean square (rms) of the deviations of the individual values from their mean. Thus, the formula for standard deviation is:

$$s = \sqrt{\frac{(Y_1 - \overline{Y})^2 + (Y_2 - \overline{Y})^2 + \ldots + (Y_N - \overline{Y})^2}{N - 1}}$$

in which

$Y_1, Y^2, \ldots, Y_N$ = the individual values in the set;
$\overline{Y}$ = the mean value of the set; and
N = the number of values in the set.

Contents

THE AASHO ROAD TEST
Report 2
Materials and Construction

Chapter 1

Introduction

This chapter presents an over-all conception of the construction of the test facilities. The first section is a review of information contained in AASHO Road Test Report 1 (HRB Special Report 61A). The second section summarizes the construction aims, administration and operations. It also discusses the weather during construction and summarizes the costs of construction.

1.1 PROJECT REVIEW

1.1.1 Background and Concepts

The AASHO Road Test was conceived and sponsored by the American Association of State Highway Officials as a study of the performance and capabilities of highway pavement and bridge structures of known characteristics under moving loads of known magnitude and frequency. It was intended to develop engineering knowledge at the project that could be used in the design and construction of new highway pavements and bridges, in the preservation and improvement of existing pavements, and in advancing toward the ultimate goal of determining an optimum balance between vehicle operation costs and the cost of the highway.

The project was financed by 49 States, the District of Columbia, the Commonwealth of Puerto Rico, the Bureau of Public Roads of the U. S. Department of Commerce, the Automobile Manufacturers Association, the American Petroleum Institute and the American Institute of Steel Construction. The Department of Defense, through its Army Transportation Corps Road Test Support Activity, furnished the drivers for the test vehicles. Foreign countries and domestic materials and transportation associations furnished resident observers and staff consultants.

The basic concepts of the AASHO Road Test were outlined in 1952 by the Working Committee of the AASHO Committee on Highway Transport. This committee also selected the test site near Ottawa, Ill., about 80 miles southwest of Chicago.

In November 1954, the Executive Committee of the American Association of State Highway Officials approved construction of the test facilities. In February 1955, the Executive Committee of the Highway Research Board, with the approval of its parent organization, the National Academy of Sciences—National Research Council, agreed that the Board would administer and direct the project.

AASHO Road Test Report 1 contains a complete history of the preparation and planning, the factors considered in selecting the test site, and the administrative organization created to direct the project.

A list of data systems pertaining to pavement research is given in Appendix I, Road Test Report 5; a list pertaining to bridge research is given in Appendix A, Road Test Report 4.

The specific objectives of the project placed major emphasis on determining significant relationships between the performance of pavements of various designs and the loading applied to them, on developing a means of

Figure 1. Looking east, Loops 5 and 2 in foreground.

evaluating pavement capabilities, and on determining the significant effects of loading on bridges of known design and characteristics.

The test facilities constructed at Ottawa included both flexible and rigid type pavements as well as steel beam, prestressed concrete beam, and reinforced concrete beam bridges.

Both single and tandem axle vehicles were used in testing the pavements and bridges. Ten different axle arrangement-axle load combinations were selected, and each was assigned to a separate traffic lane. Thus, any single pavement section or test bridge span was subjected to repetitive dynamic loading by vehicles with identical axle arrangements and axle loads.

1.1.2 Layout of Project

The test facilities were constructed as six loops. Four major loops (3 through 6) were constructed for testing with tractor-semitrailer type traffic, one smaller loop (2) for testing with light truck traffic, and one auxiliary loop (1) for testing with static, creep-speed and vibrating loads and for observations of the effects of time and weather with no traffic.

The construction plans referred to the loops as A through F. This designation was changed to Loops 1 through 6 to permit cataloging data with IBM equipment. The numerical designation is used throughout this report. Figure 2, which shows the geographic arrangement of the loops, gives the corresponding loop designations used in the construction plans and specifications.

Each loop was a segment of a four-lane divided highway whose parallel roadways, or tangents, were connected by turnarounds at both ends. As shown in Figure 2, the four major loops (3 through 6) were located along the line of proposed Federal-aid Interstate Route 80 so that upon completion of the test the tangents could be rehabilitated and used as a portion of the new route.

Tangents were 6,800 ft long in Loops 3 through 6; 4,400 ft in Loop 2; and 2,000 ft in Loop 1. On the four major loops, turnarounds had 200-ft radii and were superelevated at a rate of 0.1 ft per ft on the inner lane of the curves and at a rate of 0.2 ft per ft on the outer lane. The turnarounds for Loop 2 were superelevated at the same rates, but available right-of-way permitted only 42-ft radii.

The pavement on the north tangent and east turnaround of each loop was a flexible-type pavement consisting of a subbase, base and asphaltic concrete surfacing. That on the south tangent and west turnaround of each loop was a rigid-type pavement consisting of a subbase and portland cement concrete surfacing. As shown in Figure 3, each tangent was constructed as a succession of pavement sections, called structural sections. Each structural section was separated from adjacent sections by short transition pavements, and each was divided into two identical-pavement test sections by the centerline of the pavement.

The structural sections in each test loop tangent had varied designs. In most sections, design was varied by changing the thickness of the component layers of material. In each

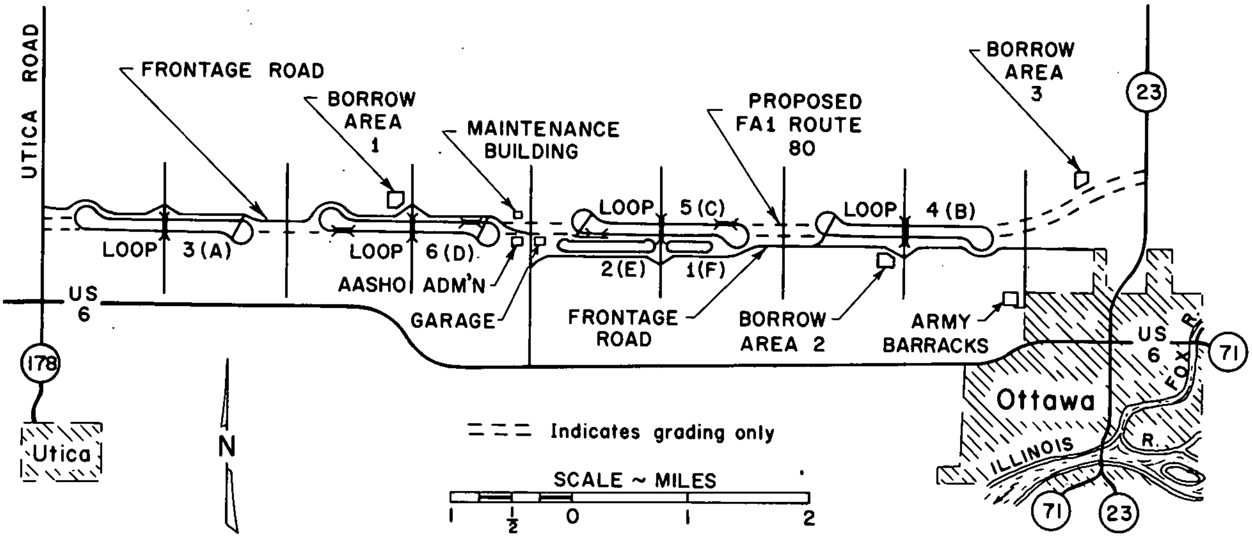


Figure 2. Map of AASHO Road Test.

tangent these sections comprised a complete factorial experiment which included all possible combinations of the selected thicknesses of the layers of material.

A few sections in the tangents of the main loops had base type or shoulder paving as design variables.

In each tangent, pavement thicknesses for the factorial experiment sections were varied about a selected design that was considered adequate for the loading to be applied. Some of the factorial sections were thinner than the selected design, some were at or near it, and some were thicker.

Table 1 shows the combinations of thicknesses of subbase, base and surfacing in the structural sections of flexible pavement in all test loops. Subbase thickness ranged from 0 to 16 in. in 4-in. increments; base thickness ranged from 0 to 9 in. in 3-in. increments; and surfacing thickness ranged from 0 to 6 in. in 1-in. increments.

The surfacing thickness considered as zero was a bituminous surface treatment consisting of two cover coats and a seal coat.

Table 2 shows the combinations of thicknesses of subbase and surfacing in the structural sections of rigid pavement in all test loops. Subbase thickness ranged from 0 to 9 in. in 3-in. increments, while surfacing was constructed in thicknesses of 2½, 3½, 5, 6½, 8, 9½, 11 and 12½ in. The surfacing occurred as both reinforced and non-reinforced for each combination of surfacing-subbase thickness.

Each flexible pavement tangent of the four major loops included sections which had base type as a design variable. In these sections the base course was built as a wedge with thickness decreasing in the direction of traffic. Four base types were used: asphalt treated aggregate, cement treated aggregate, crushed stone and gravel (see Table 1).

Asphaltic concrete paved shoulders were constructed on certain sections on both rigid and flexible pavement tangents of the four major loops.

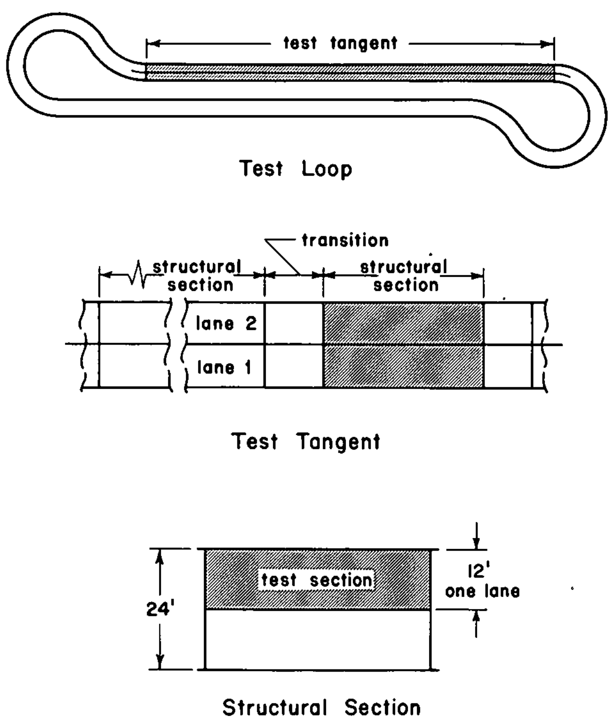


Figure 3. Typical loop layout.

TABLE 1

PAVEMENT DESIGNS, FLEXIBLE TANGENTS

Base	Subbase	Loop 1			Loop 2				Loop 3			Loop 4			Loop 5			Loop 6		
	Surfacing	1	3	5	0	1	2	3	2	3	4	3	4	5	3	4	5	4	5	6
0	0	X	X	X	X	X	X	X	X	X	X									
	4				X	X	X	X	X	X	X	X	X	X						
	8	X	X	X					X	X	X	X	X	X						
	12											X	X	X						
	16	X	X	X																
3	0				X	X	X	X	X	X	X									
	4				X	X	X	X	X	X	X	X	X	X	X	X	X			
	8								X	X	X	X	X	X	X	X	X	X	X	X
	12											X	X	X	X	X	X	X	X	X
	16																	X	X	X
6	0	X	X	X	X	X	X	X	X	X	X									
	4				X	X	X	X	X	X	X	X	X	X	X	X	X			
	8	X	X	X					X	X	X	X	X	X	X	X	X	X	X	X
	12											X	X	X	X	X	X	X	X	X
	16	X	X	X														X	X	X
9	0																			
	4														X	X	X			
	8														X	X	X	X	X	X
	12														X	X	X	X	X	X
	16																	X	X	X
SPECIAL BASE TYPES																				
2-14 (1)	0									X										
2-16 (1)	4											X								
3-19 (1)	8																	X		
2-14 (2)	0									X										
2-16 (2)	4											X								
3-18 (2)	4														X					
2-11 (3)	0									X										
3-16 (3)	4														X					
3-18 (3)	4																	X		
2-10 (4)	4											X								
3-12 (4)	4														X					
3-13 (4)	4																	X		

(1) CRUSHED STONE BASE
(2) GRAVEL BASE
(3) BITUMINOUS TREATED BASE
(4) CEMENT TREATED BASE

TABLE 2

PAVEMENT DESIGNS, RIGID TANGENTS

Reinf.	Surfacing	Loop 1		Loop 2			Loop 3				Loop 4				Loop 5				Loop 6			
	Subbase	0	6	0	3	6	0	3	6	9	0	3	6	9	0	3	6	9	0	3	6	9
REINFORCED	2 1/2	x	x	x	x	x																
	3 1/2			x	x	x		x	x	x												
	5	x	x	x	x	x		x	x	x		x	x	x								
	6 1/2							x	x	x		x	x	x		x	x	x				
	8							x	x	x		x	x	x		x	x	x		x	x	x
	9 1/2	x	x									x	x	x		x	x	x		x	x	x
	11															x	x	x		x	x	x
	12 1/2	x	x																	x	x	x
NON-REINFORCED	2 1/2	x	x	x	x	x																
	3 1/2			x	x	x	x	x	x	x												
	5	x	x	x	x	x		x	x	x	x	x	x	x								
	6 1/2						x	x	x	x		x	x	x	x	x	x	x				
	8							x	x	x	x	x	x	x		x	x	x	x	x	x	x
	9 1/2	x	x									x	x	x	x	x	x	x		x	x	x
	11															x	x	x	x	x	x	x
	12 1/2	x	x																	x	x	x

The original test facilities for the bridge studies consisted of 16 slab-and-beam bridges. The bridges were located in groups of four at the beginning of both tangents in Loops 5 and 6, as shown in Figure 4. Each bridge was a simple-span structure consisting of three identical beams and a reinforced concrete slab. The beams, on a 50-ft span, were wide-flanged rolled-steel sections with or without tension coverplates, precast prestressed concrete I-sections, or reinforced concrete T-beams cast monolithically with the slab. The bridge designs were based on stress levels substantially higher than those used in current practice.

Eight of the original test bridges were constructed with steel beams, four with prestressed concrete beams and four with reinforced concrete beams. The steel beam bridges included both composite and non-composite designs. Both pretensioned and post-tensioned beams were used in the prestressed concrete bridges.

Two additional steel beam bridges were constructed to replace two of the original steel beam structures that failed during the initial stages of the test.

1.2 SUMMARY OF CONSTRUCTION

1.2.1 Construction Aims

Because the principal objective of the test was to determine significant relationships between pavement and bridge behavior and the major variables of design and loading, it was necessary to control all extraneous variables as closely as possible. Thus, the primary construction aim was to achieve exceptional uniformity of the embankment and component layers of the pavement structure throughout all test sections.

A basic concept of the project was that general construction procedures and controls be in accordance with national practice but modified only to the extent necessary to obtain the uniformity required by the research. Normal construction procedures were determined by a nationwide survey conducted in 1953. Answers to questionnaires sent to the state highway departments were summarized by the Working Committee of the AASHO Committee on Highway Transport in a report entitled "Supplement B—Construction and Materials Requirements." This report and a later report entitled "Earthwork Construction Requirements" were used as a guide in preparing the materials and construction specifications for the project.

The revisions in normal procedures resulted in a rigid set of specifications for construction of the test facilities. In some instances the use of equipment and procedures foreign to normal highway construction was required in an effort to obtain uniformity. Thus, the plans and specifications provided for the construction of pilot sections to select the proper construction equipment and to develop acceptable construction procedures for the embankment, subbase, base and surfacing.

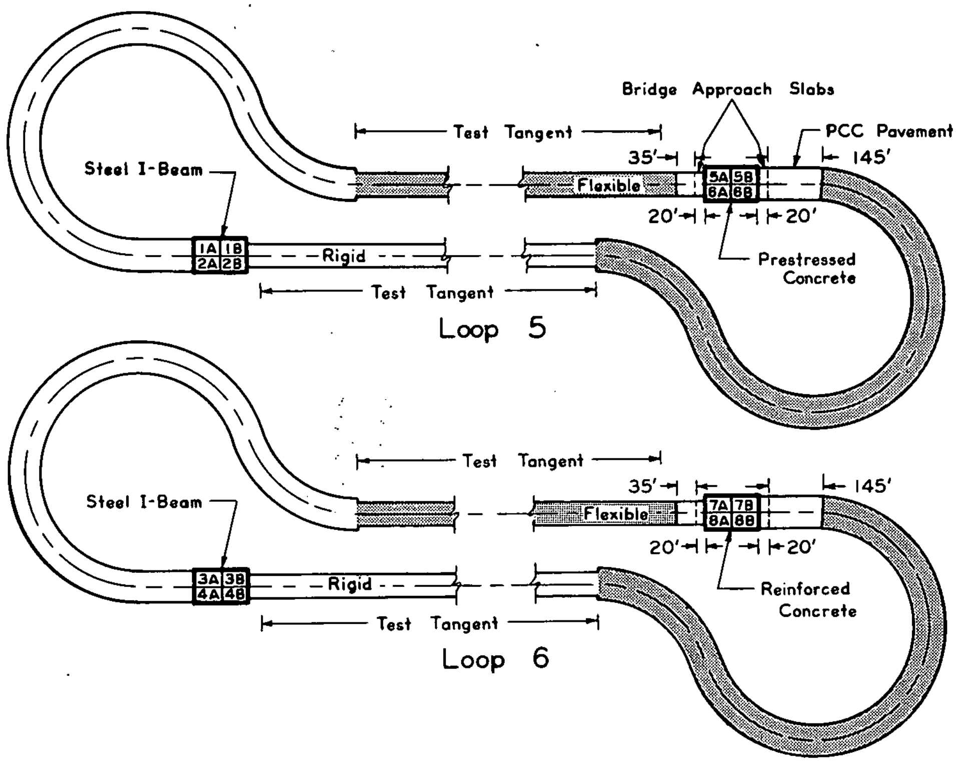


Figure 4. Locations of test bridges.

1.2.2 Administration

AASHO Road Test Report 1 discusses the administrative organization set up by the Highway Research Board for the supervision and direction of the project. The construction was supervised by the Illinois Division of Highways through a permanent task force headed by the engineer of physical research of the Division's Bureau of Research and Planning. Liaison was maintained with both the project director and the chief engineer for research. Engineering personnel for the 10-man task force unit was, for the most part, obtained from several of the Division's district offices and the Bureau of Research and Planning. The Division's top administrative and engineering personnel contributed considerable aid and advice.

Various branches of the project staff provided service to the task force during construction. The Materials Branch conducted all on-site testing for materials and construction control with the exception of that involved in on-site materials acceptance and proportioning. The research branches performed the instrument installations.

The off-site testing of materials for acceptance was performed by the Illinois Division of Highways, principally by personnel from District 3.

An important contribution to the project was made by the highway departments of Iowa, Kansas, Michigan, Minnesota, Missouri, Nebraska, Ohio, Oklahoma, and Wisconsin. These departments loaned key men to take charge of certain phases of the construction work under the general supervision of the Illinois Task Force.

Engineers were loaned to the project by several of the Illinois Highway district offices and central bureaus. Junior engineers employed by the Bureau of Public Roads were assigned to the project for approximately six months each as a part of their training program. Engineering technicians were employed during the summer by Illinois, and technicians were employed locally by the National Academy of Sciences.

Personnel demands for construction of the test facilities were large. A maximum of 110 employees were involved in control of embankment construction with 57 in construction supervision and inspection and 53 in testing for materials and construction control. During the peak of paving operations 149 employees were required for construction control. Ninety-two were utilized for construction supervision and inspection; 12 were stationed at the various material production plants for off-site testing of materials for acceptance; and the remaining 45 were involved in on-site testing for materials and construction control. The key positions were filled with engineers and junior engineers while the majority of the employees were technicians. During the summer, college students were employed as engineering technicians.

1.2.3 Summary of Operations

Completion of construction of the test facilities was scheduled originally for the late summer of 1957. However, several additions and revisions, combined with unfavorable weather conditions, delayed the completion until the fall of 1958.

The Road Test facilities were constructed in three main phases: grading, paving and test bridges. Eight separate contracts were involved (Table 3). The first seven contracts were for the construction of the original facilities. The eighth contract, Section 2B–2, was for construction of the two additional steel beam test bridges.

The contracts for Sections 2A, 2, 2–1, 2B–1 and 2B–2 included the major portion of the work involved in construction of the test pavements and test bridges.

Construction of the test facilities began July 30, 1956. The grading contract (2A) included the embankments for the four major loops and a special studies area. A smaller loop (2), designed for testing with light axle loads, was added to the contract. The special studies area was moved and designed as the auxiliary loop (1). These additions and revisions increased the work load by 20 to 25 percent, and made it necessary to procure an additional borrow area to provide sufficient selected soil for the embankments.

The work performed in 1956 included the construction of culverts, frontage road embankments and embankment for the test loops. Approximately 1¼ million cubic yards of earth were moved, and about one-half of this was placed in the upper 3 ft of the embankment within test loops. The embankments for the four major loops were substantially complete, but approximately 7,500 cu yd remained to be placed in Loops 1 and 2. Fifty percent of the earth work was completed for Loop 2, while less than 1,000 cu yd of selected soil had been placed for Loop 1. Two of the four substructures for the test bridges, which had been added as an extension to Section 2–HB, were in place.

The original plans called for a 6-in. layer of sand-gravel subbase material (sand-gravel mulch) to be placed over the test loop embankments to serve as a protective cover during the winter of 1956-57. However, difficulties in washing the material to meet the exacting specifications delayed production, and there was not enough material produced to cover all test tangent embankments by the close of the 1956 construction season. Therefore, none was placed on the test tangents until the summer of 1957. Processing top lifts of embankment on the test tangents also was deferred until immediately prior to subgrading in the summer of 1957.

Construction was resumed in April 1957, and embankment construction for Loops 1 and 2 continued. Construction of the test bridge superstructures was started in the latter part

TABLE 3

CONTRACTS FOR CONSTRUCTION OF TEST FACILITIES

Construction Section	Description of Work	Contractor	Date of Letting	Date of Award
2HF, 2F–1	Furnishing and fabricating structural steel for four grade separation structures and the steel beam test bridges.	American Bridge Division, United States Steel Corporation	April 24, 1956	May 24, 1956
2–B	Construction of bridge at Clark Run Creek and culvert at Goose Creek	Eric Bolander Construction Company	May 29, 1956	June 11, 1956
2–HB and 2–HB Ext.	Construction of four grade separation structures and substructures for test bridges	S. J. Groves and Sons Co., Arcole Mid-West Corp.	July 6, 1956	July 19, 1956
2A	Grading	S. J. Groves and Sons Co., Arcole Mid-West Corp.	July 6, 1956	July 19, 1956
2B–1	Construction of superstructures for test bridges	Valley Builders, Inc.	March 1, 1957	March 19, 1957
2–1	Construction of flexible pavement	S. J. Groves and Sons Co.	Aug. 2, 1957	Aug. 27, 1957
2	Construction of rigid pavement	S. J. Groves and Sons Co.	Aug. 2, 1957	Aug. 27, 1957
2B–2	Construction of two additional steel I-beam test bridges	Valley Builders, Inc.	March 13, 1959	March 25, 1959

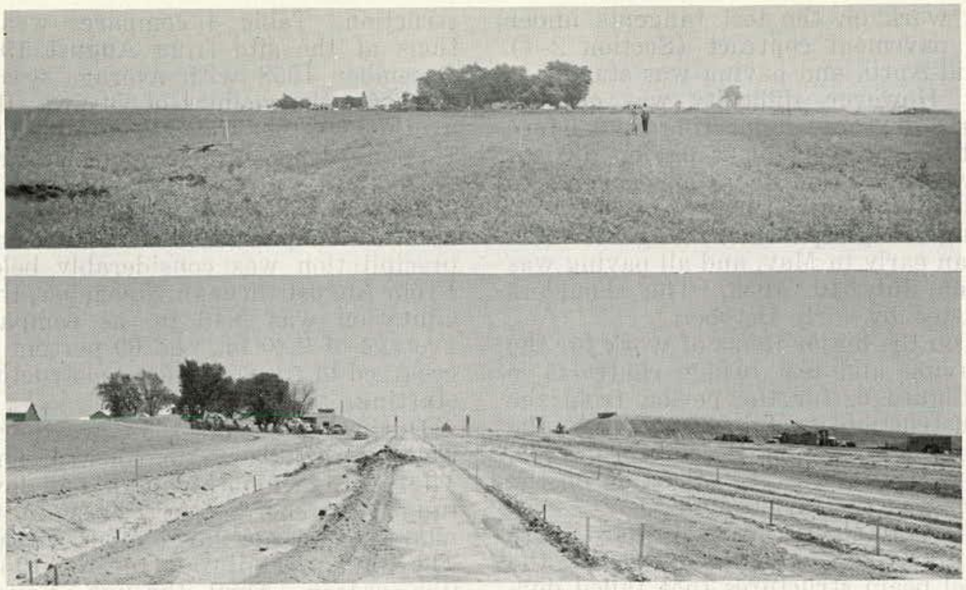


Figure 5. Loop 6 before and during construction.

of May. By the middle of June all embankment soil was in place, and the remaining work on the grading contract included processing top lifts of embankments, subgrading, and placing the sand-gravel mulch on the test tangents.

Meanwhile, plans and specifications had been prepared for a single contract for the paving. Bids were received on May 17, 1957. The single bid of $6,622,514.01 by the S. J. Groves and Sons Company was rejected, because it exceeded the budgeted funds for this construction.

As a result the Steering Committee, with the aid of the Advisory Panel on Materials and Construction, recommended changes in the plans, specifications and sequence of work which were expected to reduce costs without impairing the research objectives of the project.

The revised plans and specifications provided for two contracts, one for portland cement concrete paving and one for asphaltic concrete paving. All pavement outside the test loops included in the original contract was eliminated unless it was needed in connection with access roads.

The major consideration in reducing costs involved limitations on equipment operation. The original specifications prohibited operation of all but spreading and compaction equipment on any portion of the roadbed after placement of the sand-gravel subbase material over the embankments to serve as the mulch cover.

A test, conducted in June 1957, indicated that hauling equipment could not be operated on this subbase material. Therefore, it was decided to stabilize the subbase material and to permit operation of hauling equipment on the shoulder portions of the roadbed.

The revised specifications called for the addition of a friable, fine-grained soil to stabilize the sand-gravel material. Operation of hauling equipment on the shoulder portions of the roadbed was permitted provided the underlying embankment was not damaged. If damage did occur, the sand-gravel cover was to be removed from the outside shoulder and the hauling equipment operation restricted to this area.

On September 7, 1957, work was started on the two paving contracts. The intention was to complete all pilot construction and paving on turnarounds in 1957, and to pave the test tangents in 1958.

No work was performed in 1957 after mid-November. At this time, the grading contract (Section 2A) was substantially complete. The eight steel I-beam test bridges and the four reinforced concrete beam bridges had been completed. Two of the four prestressed concrete beam bridges were complete and the forms for the decks of the other two were in place. Difficulties in tensioning the post-tensioned beams prevented completion of these two bridges.

The work scheduled for 1957 under the paving contracts was not completed. The pilot construction was well under way, except that only 28 tons of asphaltic concrete had been placed. The portland cement concrete pavement on the west turnarounds was substantially completed for Loops 3 and 6, and 50 percent completed for Loop 5. No paving had been done on any of the flexible pavement turnarounds (east turnarounds).

Construction was resumed on April 1, 1958. Early efforts were directed toward completing the pilot sections, the rigid and flexible pavement turnarounds, and the two prestressed concrete bridges.

The first work on the test tangents under the flexible pavement contract (Section 2–1) began in mid-April, and paving was started in early July. However, difficulty was encountered in spreading and compacting the surface course mixture and continuous paving did not get under way until August. All placement was completed by October 4, 1958.

The paving of rigid pavement tangents (Section 2) began early in May, and all paving was completed on July 10, 1958. The shoulders were completed by early October.

Progress on the major items of work for the grading, paving and test bridge contracts is shown in Figure 6, for the period from the start of construction on July 30, 1956, to the completion of all work on December 3, 1958. The shaded areas indicate construction on the pilot sections and the turnarounds.

Two additional steel I-beam bridges were constructed in 1959 to replace two of the original steel beam structures that failed during the initial stages of the test. The work under this contract, Section 2B–2, was started in April. The new structures were in place and ready to receive test traffic on June 19.

1.2.4 Weather

A weather station was established near the center of the project at the beginning of construction. Table 4 compares weather conditions at the site from August 1956 through December 1958 with average weather conditions for the vicinity of Ottawa, Illinois. The average weather conditions were obtained from U. S. Weather Bureau records over a 10-year period.

During the 1956 construction season the monthly temperatures were near normal while precipitation was considerably below normal. From August through November, the total precipitation was 3.46 in. as compared to the average of 9.20 in., and 69 percent of the total occurred in August when construction was just starting.

During 1957 and 1958, weather conditions were more adverse to construction progress. The average monthly high and low temperatures, in general, were close to the normal monthly high and low temperatures (Table 4). The total precipitation for the 1957 construction season, April through November, was 28.25 in., while the average rainfall for this period was 24.38 in. During the 1958 construction season, April through October, total precipitation was 30.41 in. as compared to the average of 22.58 in.

The greatest monthly precipitation was in July 1958 with 12.53 in. One of the largest 24-hr rainfalls on record for Ottawa and vicin-

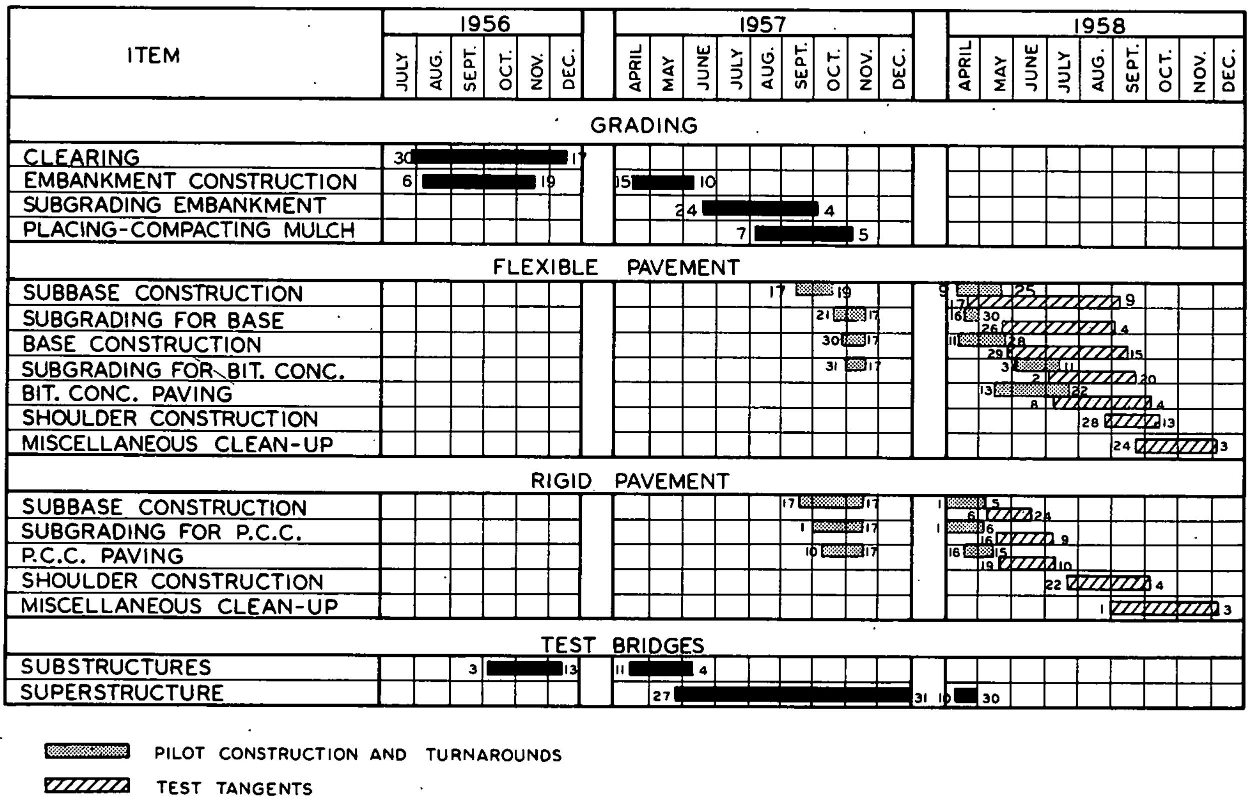


Figure 6. Construction progress chart.

TABLE 4

RECORD OF TEMPERATURE AND PRECIPITATION DURING CONSTRUCTION

(August 8, 1956 to December 31, 1958)

Month	Average Monthly Temp. (°F)				Precip. (in.)		Snow (in.)[2]		Days of Precip.[3]
	Actual		Normal[1]						
	High	Low	High	Low	Actual	Normal	Actual	Normal	
(a) 1956									
Aug.	84	59	85	62	2.39	2.60			5
Sept.	81	47	78	53	0.21	3.20			2
Oct.			67	43	0.40	1.60		0.06	1
Nov.	48	27	50	32	0.46	1.80		2.25	4
Dec.	41	24	38	21	0.65	2.40	4.50	8.02	2
Total					4.11	11.60	4.50	10.33	14
(b) 1957									
Jan.	29	6	36	19	3.12	1.95	9.25	5.50	7
Feb.	42	23	36	22	1.23	1.50	0.24	6.60	3
Mar.	48	28	46	29	1.61	3.30	3.50	3.00	5
Apr.	63	41	60	38	8.50	4.30			12
May	74	50	75	50	4.46	3.80			9
June	83	59	84	62	2.67	4.10			5
July	89	64	88	65	2.76	2.98			8
Aug.	89	61	85	62	2.64	2.60			5
Sept.	77	50	78	53	1.69	3.20			5
Oct.	66	39	67	43	3.11	1.60		0.06	5
Nov.	50	32	50	32	2.42	1.80	Trace	2.25	6
Dec.	42	24	38	21	2.82	2.40	5.50	8.02	4
Total					37.07	33.53	18.49	25.43	74
(c) 1958									
Jan.	31	16	36	19	1.02	1.95	10.25	5.50	2
Feb.	32	11	36	22	0.79	1.50	0.25	6.60	2
Mar.	46	27	46	29	0.17	3.30	0.25	3.00	1
Apr.	65	39	60	38	2.94	4.30			7
May	76	45	75	50	2.41	3.80			6
June	77	54	84	62	6.09	4.10			9
July	83	61	88	65	12.53	2.98			9
Aug.	86	62	85	62	1.61	2.60			5
Sept.	80	55	78	53	2.85	3.20			5
Oct.	70	42	67	43	1.98	1.60		0.06	4
Nov.	55	33	50	32	1.59	1.80	0.50	2.25	7
Dec.	30	10	38	21	0.92	2.40	8.50	8.02	4
Total					34.90	33.53	19.75	25.43	61

[1] Obtained from local U. S. Weather Bureau records over a 10-year period.
[2] To convert snow to precipitation it was assumed that 1 in. of precipitation is equivalent to 10 in. of snow.
[3] 0.005 in. or more.

ity occurred on July 14. At the project site 6.68 in. were measured, most of which occurred as a heavy thunderstorm in the early morning. The official record cited 8¾ in. of rainfall.

The effect of weather on working days during construction is shown in Table 5. The total possible working days included in the table was derived on the basis of six working days per week, exclusive of legal holidays.

During the 1956 construction season only 9 percent of the possible working days was lost due to bad weather, and two-thirds of this occurred in the first month of construction. During the period from August 30 to October 26, the contractor operated 48 consecutive working days without interruption.

Conditions were not as favorable during the construction seasons of 1957 and 1958. In 1957, 31 percent of the possible working days was lost due to rain, and 24 percent was lost in 1958. As shown in Table 5, the longest single period without interruption due to rain was 19 working days in 1957 and 15 working days in 1958.

1.2.5 Construction Costs

The costs of constructing the test facilities, including right-of-way and drainage and traffic structures, were financed in most part from:

(1) A research grant from the Bureau of Public Roads, Department of Commerce;

(2) A joint-State research fund established by 49 States, the District of Columbia, and the Commonweath of Puerto Rico, with the Bureau of Public Roads as fiscal agent; and

(3) To the extent of the normal cost of the four-lane divided highway ultimately to become a part of Interstate Highway 80, from Federal-aid funds matched by the State of Illinois.

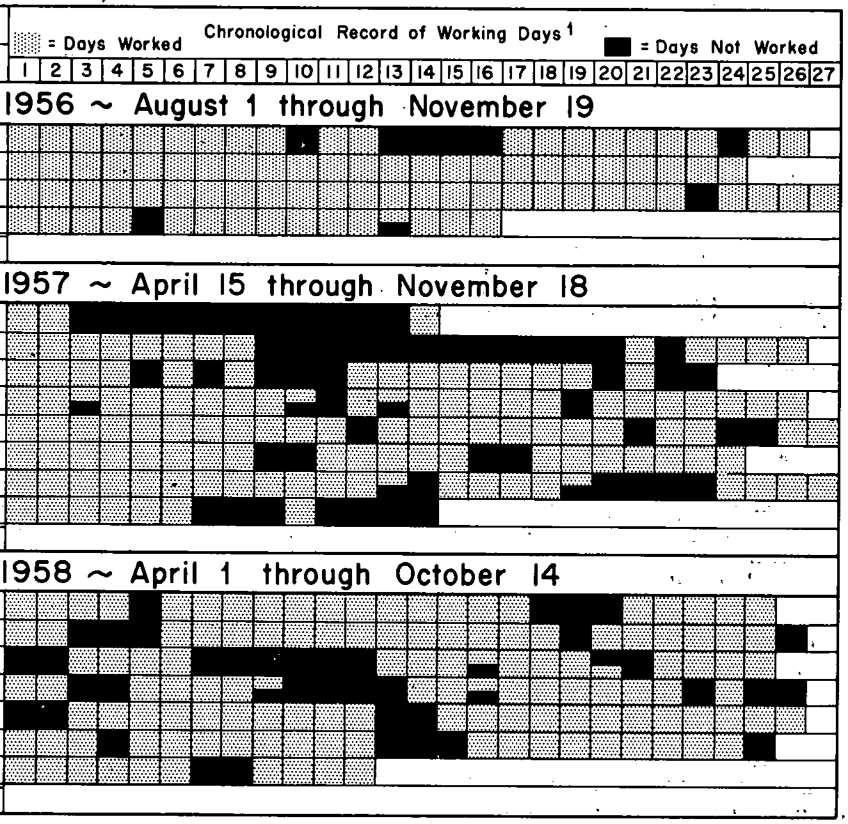
To a lesser extent, construction costs were financed from industry grants contributed by the Automobile Manufacturers Association, the American Petroleum Institute and the American Institute of Steel Construction.

A Bureau of Public Roads—Illinois Division of Highway project agreement covering all costs reimbursable to the State of Illinois was executed on May 9, 1958. Attached to and made a part of the project agreement was a fiscal agreement executed June 10, 1955, (later amended) between the National Academy of Sciences—National Research Council and the Bureau of Public Roads, Department of Com-

TABLE 5

RECORD OF WORKING DAYS

Month	Possible Working Days	Days Worked	Days Not Worked No.	Days Not Worked Percent
1956 ~ August 1 through November 19				
Aug.	26	20	6	23
Sept.	24	24	0	0
Oct.	27	26	1	4
Nov.	16	14.5	1.5	9
Totals	93	84.5	8.5	9
1957 ~ April 15 through November 18				
Apr.	14	3	11	79
May	26	13	13	50
June	23	15	8	35
July	26	22.5	3.5	13
Aug.	27	23	4	15
Sept.	24	20	4	17
Oct.	27	21	6	22
Nov.	14	7	7	50
Totals	181	124.5	56.5	31
1958 ~ April 1 through October 14				
Apr.	25	21	4	16
May	26	21	5	19
June	25	15	10	40
July	26	16	10	38
Aug.	26	22	4	15
Sept.	25	20	5	20
Oct.	12	10	2	17
Totals	165	125	40	24



[1] Consecutive numbering of possible working days in month

merce. This fiscal agreement provided for direct reimbursement by the Bureau of Public Roads to the State of Illinois for all costs of construction incurred by the State.

The project agreement included the following basis of payment for construction and rehabilitation of the test facility:

Item	Cost Items
Federal-aid project:	
Apportioned Federal funds	$3,551,514.99
Illinois State funds	497,349.12
Subtotal	4,048,864.11
Test features:	
B.P.R. administrative funds	6,365,100.00
Joint State funds	1,792,451.02
Subtotal	8,157,551.02
Total reimbursable cost	$12,206,415.13

The normal cost of providing the four-lane divided highway, exclusive of bridges and structures incorporated in the ultimate improvement, was established at $300,000 per mi—$113,000 per mi for grading and $187,000 per mi for paving. Engineering costs incurred by the State of Illinois, in provision of the Interstate route, were set at 6.5 percent of the total construction cost—3.3 percent for preliminary engineering and 3.2 percent for construction engineering. Engineering costs in excess of these percentages were chargeable to research funds. The project agreement also included provision for engineering and construction costs involved in post-research rehabilitation.

The temporary right-of-way required for the loop turnarounds and the additional borrow area required by the research were charged to research funds. All remaining right-of-way and borrow area, the four overhead structures over the test tangents (Section 2HF, 2 HF-Ext. and 2HB), and the structures at Clark Run Creek and Goose Creek (Section 2B) were necessary for the permanent improvement and were paid for by the State with Federal-aid participation.

The remaining contracts involved the construction of the test facilities. Construction Sections 2–A, and 2–1 include the grading and paving for the test pavements, while Sections 2F–1, and 2HB–Extension, 2B–1 and 2B–2 were for construction of the test bridges.

A summary of costs and cost distribution for constructing the test facilities, exclusive of the post-research rehabilitation costs, is given in Table 6. The substantial increase in the grading contract, Section 2A, was due to the addition of Loop 2, moving and expanding Loop 1, the stabilization of the sand-gravel subbase

TABLE 6

SUMMARY OF CONSTRUCTION COSTS (IN DOLLARS)

Contract	Awarded Amount	Revisions		Engineering Costs		Total Cost	Cost Distribution	
		Additions	Deductions	Preliminary	Construction		Research	Illinois (F.A.)
Right-of-way costs					24,355.41	788,526.59	54,030.75	734,495.84
Relocating utilities				12.51	12.13	1,417.92[1]	403.81	—
2–A	1,829,386.06	1,228,295.80	309,020.36	90,705.83	87,957.17	2,927,324.50	2,027,263.98	900,060.52
2	2,865,875.43	209,080.47	186,115.27	95,331.74	92,442.90	3,076,615.27	2,537,534.91	539,080.36
2–1	2,824,124.57	418,189.67	304,575.05	96,945.39	94,007.65	3,128,692.23	2,580,381.01	548,311.22
2F–1	17,331.72	—	360.40	560.05	543.08	18,074.45	18,074.45	—
2HB–Ext.	—	67,543.22	—	2,228.93	2,161.38	71,933.53	71,933.53	—
2B–1	76,967.12	298.22	497.34	2,533.34	2,456.58	81,757.92	81,757.92	—
2B–2	22,449.00	86.70	—	743.68	721.14	24,000.52	24,000.52	—
2HF	89,767.93	—	—	2,962.34	2,872.58	95,602.85	—	95,602.85
2HF–Ext.	—	31,100.74	67.25	1,024.11	993.07	33,050.67	—	33,050.67
2HB	195,613.94	55,852.39	54,484.13	6,500.41	6,303.43	209,786.04	—	209,786.04
2B	83,961.90	780.00	2,770.40	2,705.06	2,623.09	87,299.65	—	87,299.65
Totals	8,005,477.67	2,011,227.21	857,890.20	302,253.39	317,449.61	10,544,082.14[1]	7,395,380.88	3,147,687.15

[1] Includes $1,014.11 paid 100 percent by State of Illinois.

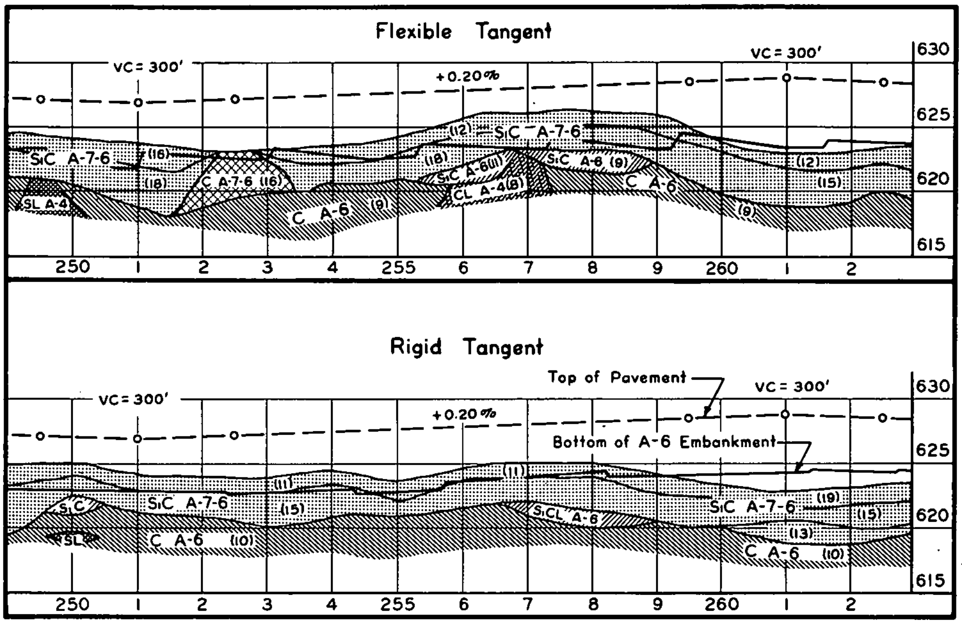


Figure 7. Existing soils profiles, Loop 5.

material, and an increase over the estimated amounts of water and processing required for embankment construction.

1.3 DESCRIPTION OF TEST FACILITIES

1.3.1 Embankment

The upper 3 ft of embankment within test loops was constructed with a selected A–6 soil. The top surface was subgraded to the required lines and grade, and covered with a layer of sand-gravel subbase material (sand-gravel mulch) to prevent loss of moisture. This work was done under the grading contract, Section 2A.

The completed embankments, with the sand-gravel cover, seasoned during the winter of 1957-58. The sand-gravel material was expected to prevent excessive loss of moisture and to retain a portion of the rainfall so that the subgrade condition of new embankment would simulate that normally found in embankments under existing pavements.

One basic requirement of the Road Test was that the embankments for the test pavements consist of a reasonably uniform fine-grained soil susceptible to pumping. Based on a soil report prepared without resort to field sampling, the Working Committee of the AASHO Committee on Highway Transport proposed that the upper 3 ft be constructed with an A–6 soil having a Group Index between 9 and 13, inclusive.

A soil survey was made in the spring of 1955 by the District 3 Bureau of Materials of the Illinois Division of Highways. The purpose of this survey was to classify and determine the physical characteristics of the existing soils along the proposed test tangents and to determine the feasibility of obtaining a selected A-6 soil for the embankments from within the right-of-way limits.

The survey indicated that the soils were reasonably uniform thoughout the area of the project. The soils profiles shown in Figure 7 are typical of most of the area.

The survey also indicated that a sufficient quantity of suitable A-6 soil could not be obtained within the right-of-way. However, borings in several areas adjacent to the right-of-way revealed three areas where the combined estimated quantity of suitable A-6 soil was sufficient for the construction of the test loop embankments.

The borings also revealed that the suitable A-6 soil existed as a layer of yellow-brown underlain by a layer of gray, and preliminary estimates indicated that it was necessary to use both soils.

Tests indicated that the physical characteristics of the two colors of soil were similar. However, because of the nature of the research, the differences were sufficient to warrant special consideration (see Tables 7 and 8). Therefore, the yellow-brown material, which approximated 60 percent of the total available, was used in the center two-thirds of the embankment and the gray material in the remainder.

A typical cross-section of embankment on test tangents is shown in Figure 8. The division between the yellow-brown and gray soils was formed on a 2:1 slope which, when ex-

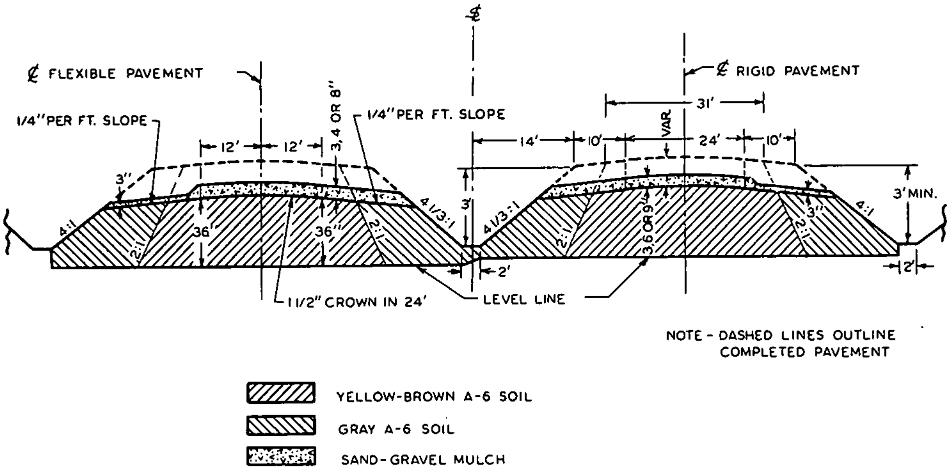


Figure 8. Typical cross-section of A–6 soil embankment with sand-gravel mulch cover.

tended to the surface of the finished shoulders, gave a width of 31 ft. Thus, the actual width of the yellow-brown soil at both the top and bottom of the A-6 embankment depended on the depth of the pavement structure and generally varied from one section to the next.

The embankment on test tangents was constructed for a four-lane divided highway with two 44-ft roadbeds and a 48-ft depressed median. The bottom of the A-6 soil formed a level line, and the thickness of this embankment was 36 in. measured at the edges of pavement. The top surface of the center 24-ft portion of the embankments was finished to a 1½-in. crown, and the shoulder portions were finished to a ½-in. per ft slope.

A typical longitudinal section of the embankment is shown in Figure 9. The top surface of the pavement formed a smooth grade. However, since pavement structural thickness was varied, the relative position of the top and bottom of the A-6 embankment generally varied from section to section as shown.

The pavement grade, which generally varied between plus and minus 0.20 percent, was laid so that the test pavements would be constructed on fill throughout. In some cases the bottom of the A-6 embankment was below the original ground surface, and in other instances it was above. Embankment construction to the bottom of the 3 ft of A-6 soil was performed with the same precision and care as the A-6 soil embankment. However, the selected A-6 soil was not used.

A typical cross-section of embankments within the turnarounds is shown in Figure 10. The top 3 ft of these embankments also consisted of the yellow-brown and gray A-6 soils. The grade lines for the turnarounds were laid so that the elevation of the low shoulder would be somewhat above the natural ground and smooth approaches would be provided for attaining the superelevation.

Subbase material was used for the protective cover over the embankment so that it could be utilized in-place as part of the subbase for

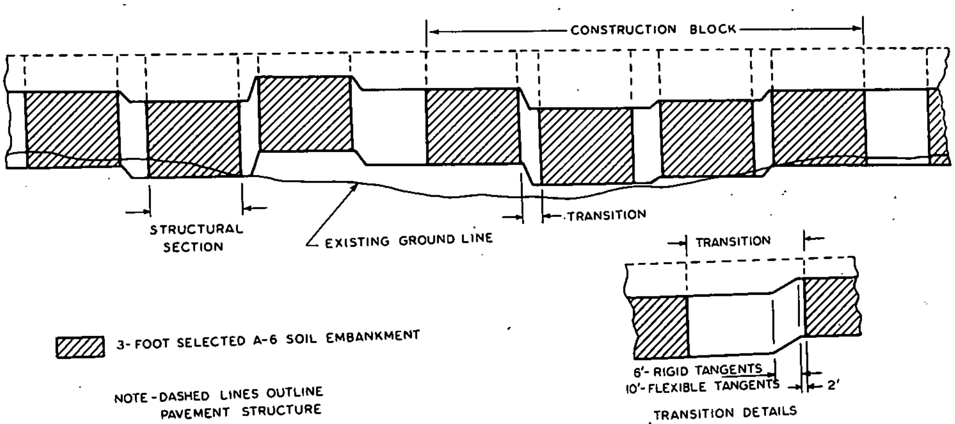


Figure 9. Typical longitudinal section of embankment of test tangents.

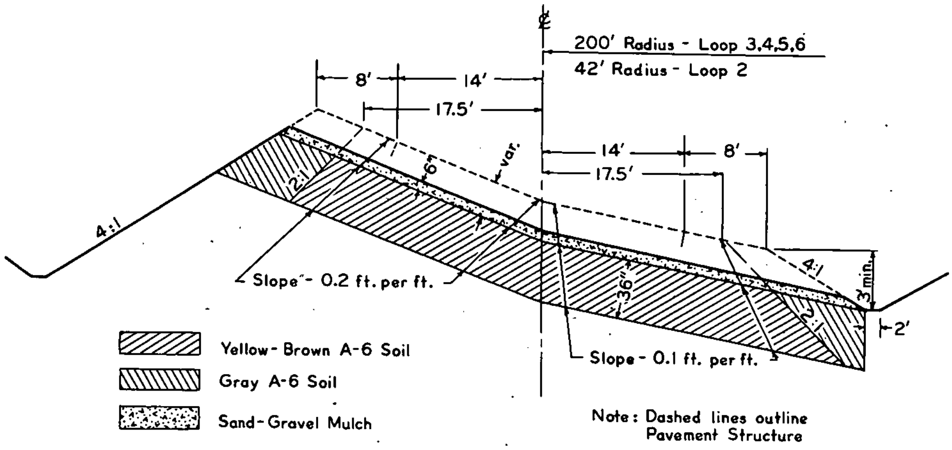


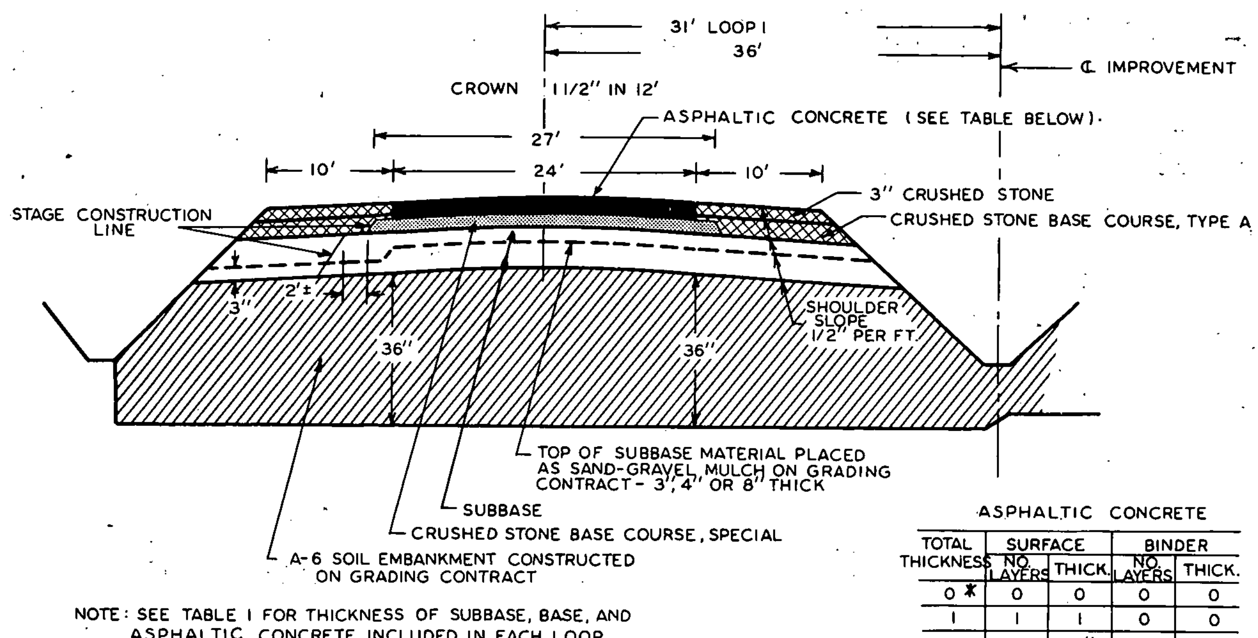
Figure 10. Typical cross-section of embankment on turnarounds with sand-gravel mulch cover.

construction of the flexible and rigid pavements. When used as the protective cover, the material was termed "sand-gravel mulch." This terminology was used in the construction specifications and in many of the construction and materials testing records.

1.3.2 Flexible Pavement

The flexible pavement sections were constructed on the north tangent and east turnaround of each loop under the paving contract designated Section 2-1. Each tangent consisted of a series of structural sections occurring in random order and separated by transition pavements. Thicknesses of subbase, base and surfacing varied from one structural section to the next.

A typical cross-section of the flexible pavements on test tangents is shown in Figure 11. The subbase and the base course were con-



ASPHALTIC CONCRETE

TOTAL THICKNESS	SURFACE NO. LAYERS	SURFACE THICK.	BINDER NO. LAYERS	BINDER THICK.
0 *	0	0	0	0
1	1	1	0	0
2	1	2″	0	0
3	1	1 1/2″	1	1 1/2″
4	1	1 1/2″	2	1 1/4″
5	1	1 1/2″	2	2″, 1 1/2″
6	1	1 1/2″	3	1 1/2″

*A BITUMINOUS SURFACE TREATMENT CONSISTING OF TWO COVER COATS AND A SEAL COAT

Figure 11. Cross-section details of flexible pavements on test tangents.

structured full width from side slope to side slope. The asphaltic concrete surfacing was 24 ft in width and was constructed with a 1½-in. parabolic crown. In most cases, the shoulders were surfaced with 3 in. of crushed stone. Earth shoulders were provided for those sections having a 0- or 1-in. thickness of surfacing placed directly upon the A-6 embankment.

The subbase material was a uniformly graded sand-gravel mixture. Construction of the subbase included processing and compacting the material previously placed as "mulch" as well as placement and compaction of additional material. A discussion of the subbase, including material characteristics and construction procedures, is given in Chapter 3.

The base course for the sections included in the main factorial experiments was constructed of two separate gradations of crushed dolomitic limestone. The portion directly under the surfacing was a special gradation (crushed stone base course, special, in Figure 11). The remainder of the base (crushed stone base course, Type A) was a standard crushed stone material used by the Illinois Division of Highways. The base was constructed in this manner in the interest of economy.

Four types of base were involved in a special

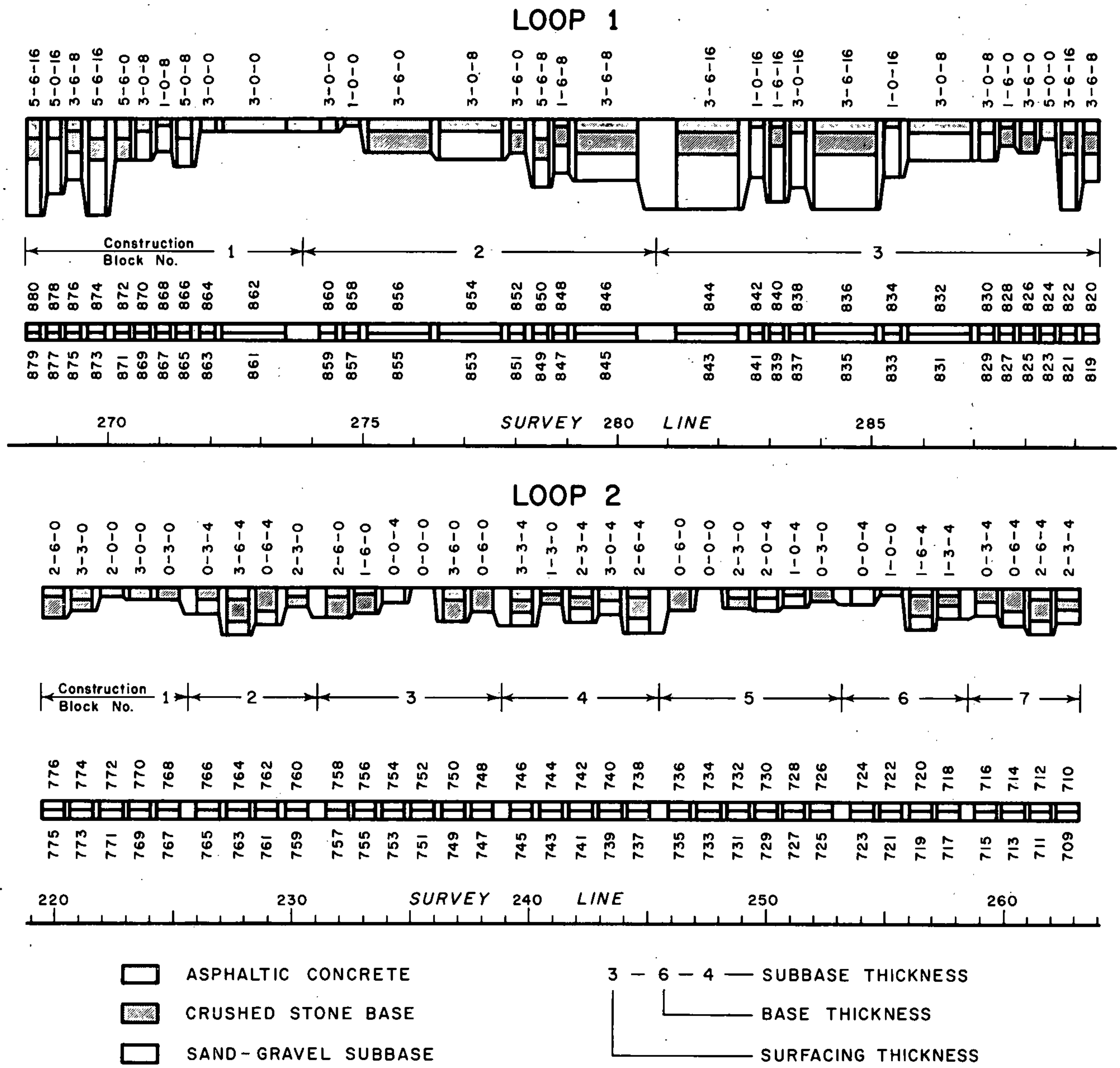


Figure 12. Layout of flexible tangent, Loops 1 and 2.

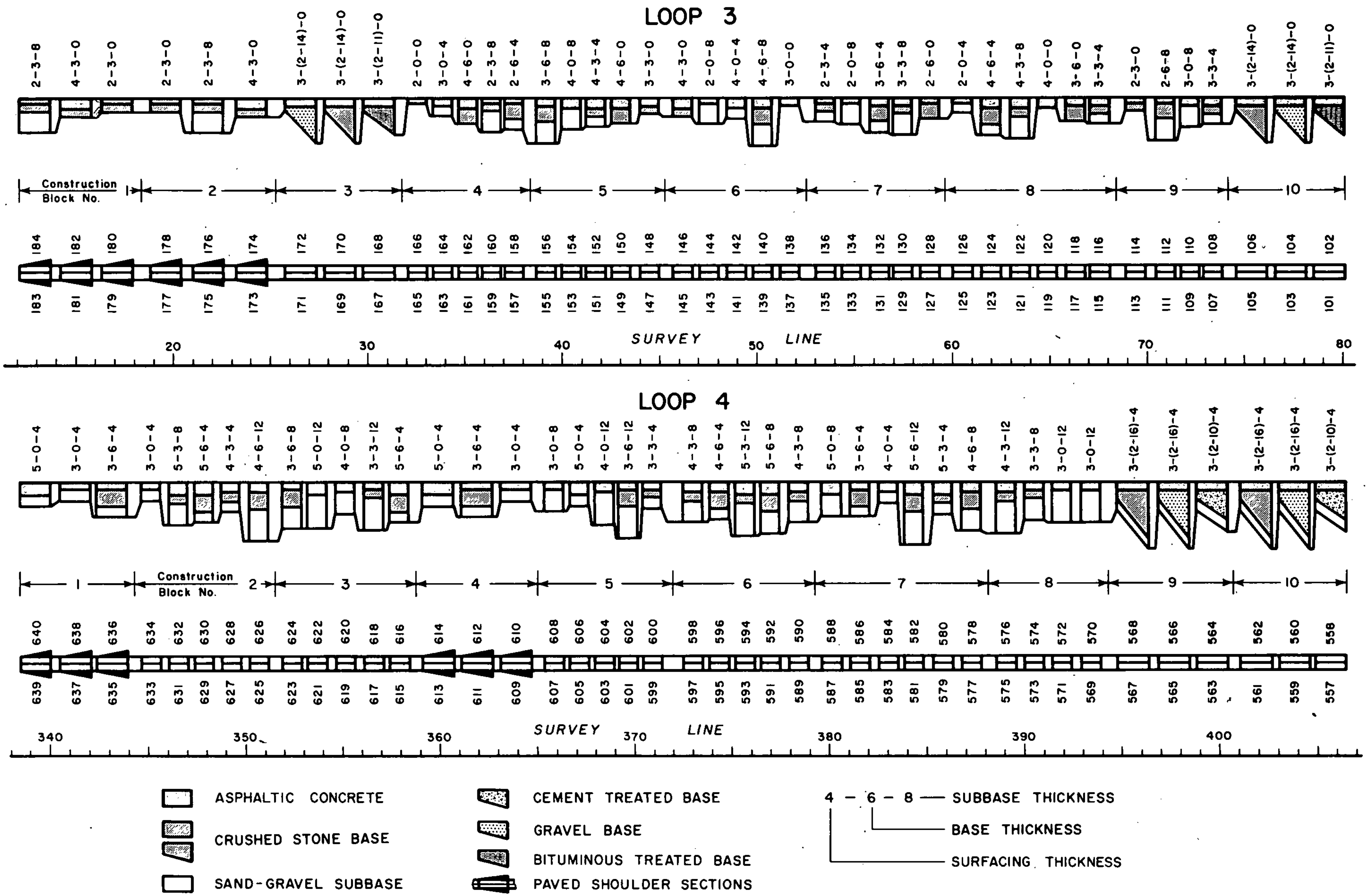


Figure 13. Layout of flexible tangent, Loops 3 and 4.

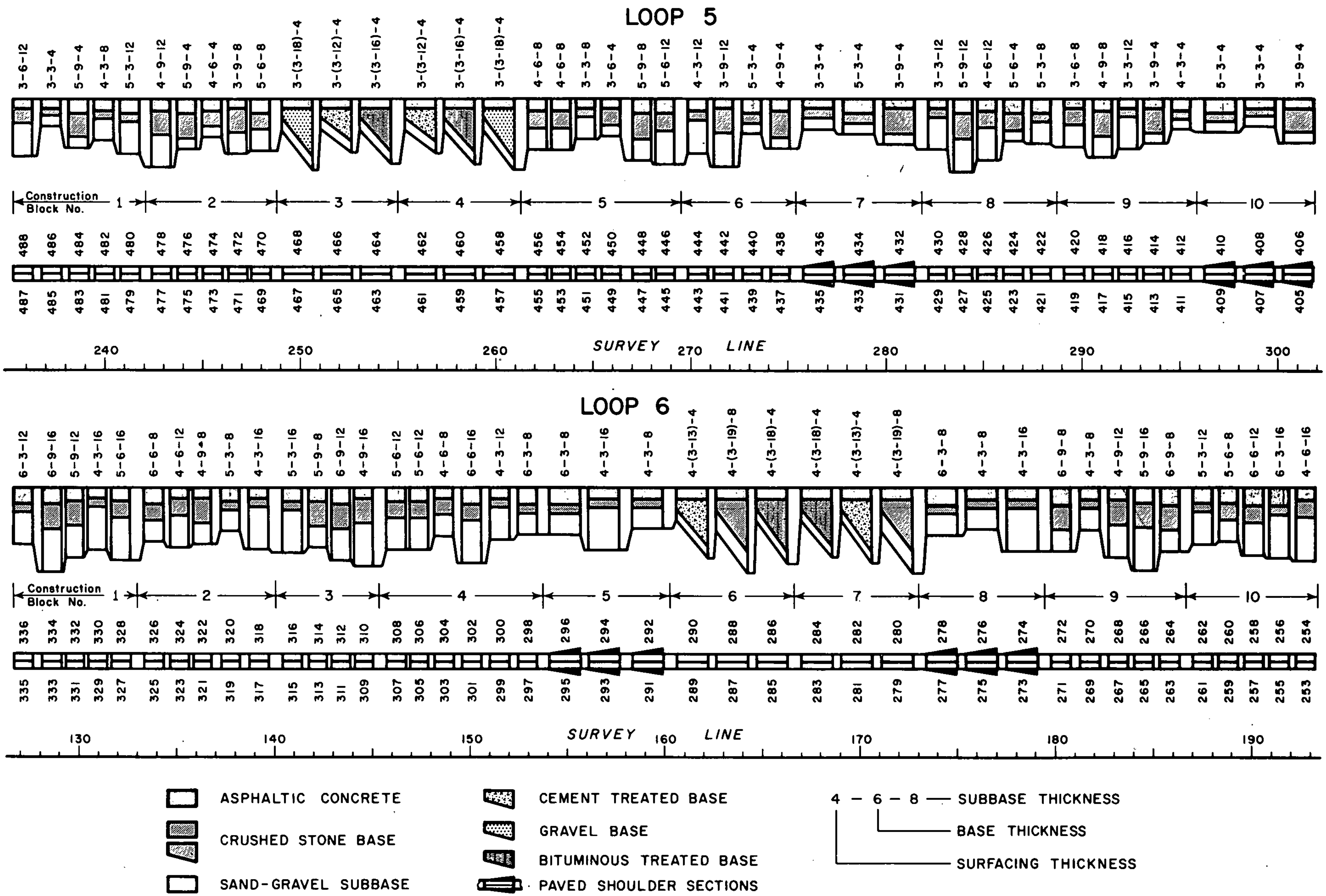


Figure 14. Layout of flexible tangent, Loops 5 and 6.

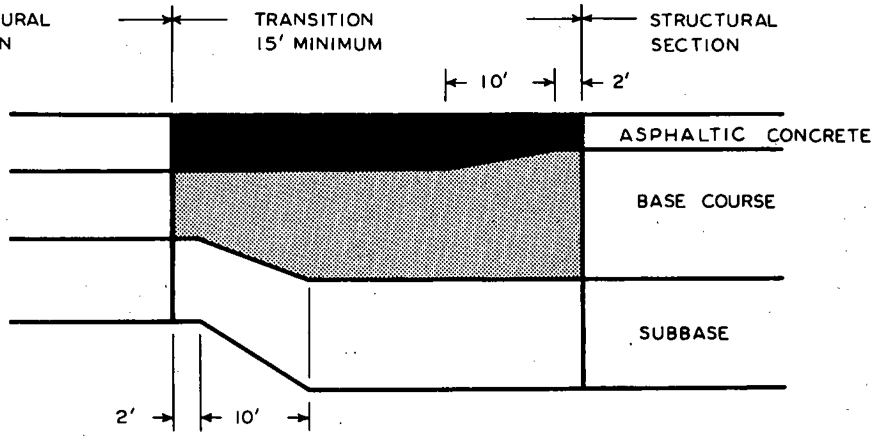


Figure 15. Details of transitions between structural sections.

base-type study included in the four major loops: a cement treated base, a bituminous treated base, an uncrushed gravel base, and a crushed stone base. The crushed stone was the same as that used in sections in the main experiment. Three of the four base types appeared in six structural sections in each loop. In these sections the base course was constructed as a wedge with the thickness decreasing uniformly in the direction of traffic. In each case the particular base-type was used only in the center portion directly under the surfacing. The remainder of the base was Type A crushed stone. The materials and construction for the base courses of the flexible pavements are discussed in Chapter 4.

The asphaltic concrete surfacing consisted of a binder course and a surface course for thicknesses of 3 in. or greater. Only a surface course was used for thicknesses of 1 and 2 in. The number and thicknesses of layers of the binder and surface courses for the various thicknesses of surfacing are given in Figure 11. The sections considered to have a zero thickness of surfacing were those in Loop 2 having only bituminous surface treatment (two cover coats and a seal coat). The surfacing for the flexible pavements is discussed in Chapter 5.

Material of the same gradation as that for Type A crushed stone base was used on the shoulders of most sections. However, six structural sections in each of the four major loops were included in a special paved shoulder study. The shoulders of these sections were paved with asphaltic concrete 3 in. thick (1½-in. surface plus 1½-in. binder) and decreasing in width in the direction of traffic from 8 ft to zero. The remainder of the 10-ft shoulder was surfaced with 3 in. of Type A crushed stone.

Figures 12, 13 and 14 show the layouts of the structural sections in each flexible pavement tangent. Structural sections in Loops 3, 4, 5 and 6 were 100 and 160 ft long. The 100-ft sections were those in the main factorial experiments, and the 160-ft sections were the special base-type wedge sections and the paved shoulder sections. In Loop 2 all sections were 100 ft long. The lengths in Loop 1 were 25 and 125 ft. The 125-ft sections were included in a special subsurface study.

Details of the transitions separating structural sections are shown in Figure 15. The

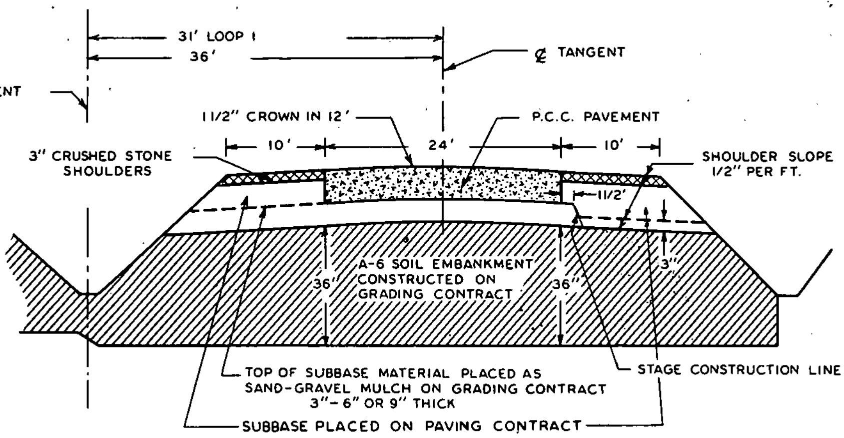
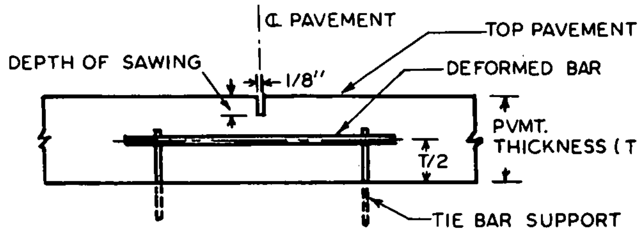


Figure 16. Cross-section details of rigid pavements on test tangents.

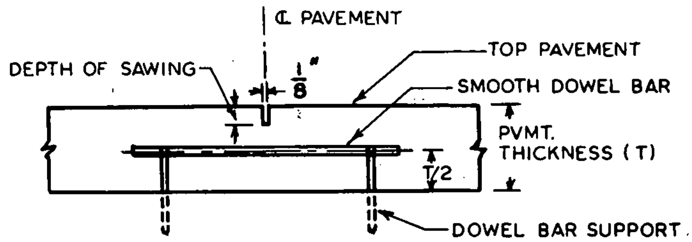
thickness of the surfacing, base and subbase in each section was extended 2 ft into the transition. Tapers from one thickness to the other were made in a distance of 10 ft. For the remaining portion of the transition pavement, the thicknesses of the component layers were selected for maximum pavement strength as shown in the figure.

The sand-gravel mulch was removed from outside shoulders of test tangents and equipment operated on this portion of the embankment during the construction of the subbase, base and surfacing. Figure 11 shows the stage-construction line for the subbase and base. All construction to the right of this line was completed before starting any construction on the outside shoulder.

Before constructing the subbase and base course on outside shoulders, this portion of the embankment was reprocessed. One to 2 in. of A-6 soil was spread on the shoulder; the top 6 in. was scarified, pulverized, adjusted for moisture, and recompacted; and the surface was reshaped to the required line and grades.



LONGITUDINAL JOINT ASSEMBLY



TRANSVERSE CONTRACTION JOINT ASSEMBLY

PAVEMENT THICK. (IN.)	DEPTH OF SAWING (IN.)	TRANSVERSE JOINT SMOOTH DOWEL BARS L1 DIA. X LENGTH	LONGITUDINAL JOINT DEFORMED TIE BARS L2 SIZE X LENGTH	PAVEMENT REINFORCEMENT * FABRIC STYLE	FABRIC WEIGHT L3	DEPTH IN PVMT.
2 1/2	3/4	3/8 X 12	NO. 3 X 20"	66-1010	21	1 1/4"
3 1/2	1	1/2 X 12	NO. 3 X 20	66-88	30	1 3/4
5	1 1/4	5/8 X 12	NO. 3 X 20	612-66	32	2
6 1/2	1 1/2	7/8 X 18	NO. 4 X 24	612-44	44	2
8	1 3/4	1 X 18	NO. 4 X 24	612-33	51	2
9 1/2	2	1 1/4 X 18	NO. 5 X 30	612-22	59	2
11	2 1/4	1 3/8 X 18	NO. 5 X 30	612-11	69	2
12 1/2	2 1/2	1 5/8 X 18	NO. 5 X 30	612-00	81	2

* CODE FOR FABRIC STYLE

6 12-4 4
- 4 — GAGE OF TRANSVERSE WIRES
- 4 — GAGE OF LONGITUDINAL WIRES
- 12 — LONGITUDINAL SPACING OF TRANSVERSE WIRES (INCHES)
- 6 — TRANSVERSE SPACING OF LONGITUDINAL WIRES (INCHES)

L1 ALL DOWEL BARS SPACED AT 12" CENTERS

L2 ALL TIE BARS SPACED AT 30" CENTERS L3 WEIGHT IN POUNDS PER 100 SQ. FT.

Figure 17. Portland cement concrete pavement accessories.

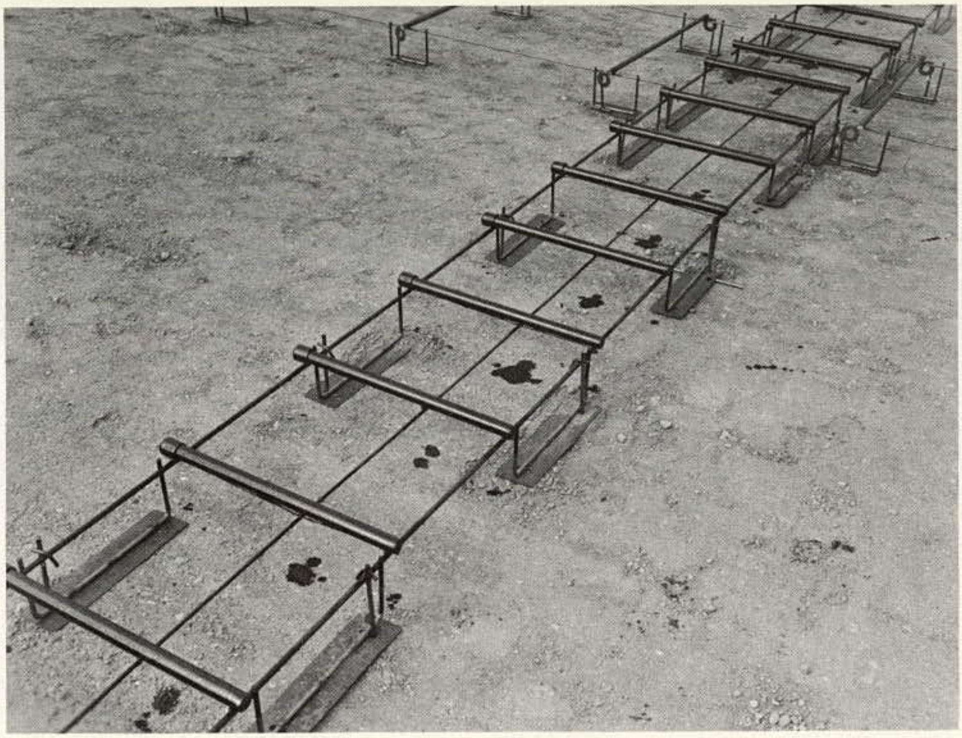


Figure 18. Transverse contraction joint assembly.

1.3.3 Rigid Pavement

The rigid pavement sections were constructed on the south tangent and west turnaround of each loop under the paving contract designated Section 2. Each tangent consisted of a series of structural sections occurring in random order and separated by transition pavements. Thicknesses of subbase and surfacing varied from structural section to structural section.

A typical cross-section of the rigid pavements is shown in Figure 16. The subbase was constructed full width from side slope to side slope. The portland cement concrete surfacing was 24 ft wide and was constructed with a 1½-in. parabolic crown. The 10-ft shoulders were surfaced with 3 in. of crushed stone placed directly on the subbase material or on the embankment soil in sections with no subbase.

The sand-gravel material for the subbase was the same as that used for the flexible pavements. The material characteristics and construction procedures and controls are given in Chapter 3.

The portland cement concrete surfacing included both reinforced and non-reinforced sections. Transverse joints (contraction joints formed by sawing) were spaced at 40 ft in reinforced sections and 15 ft in non-reinforced sections. The lengths of structural sections in Loops 2, 3, 4, 5 and 6 were 120 ft (8 panels) for the non-reinforced sections and 240 ft (6 panels) for the reinforced sections. The lengths of structural sections in Loop 1 were 15 (1 panel) and 120 ft (8 panels) for non-reinforced, and 40 ft (1 panel) for reinforced. The 120-ft sections in Loop 1 were included in a special subsurface study.

Transverse joints in both reinforced and non-reinforced pavement contained load transfer devices, with the size of dowels consistent with pavement thickness. The longitudinal center joint was also a sawed joint containing tie-bar assemblies. The pavement reinforcement was welded wire fabric of weights consistent with pavement thickness. Details of the transverse and longitudinal joints and of the pavement fabric for the various pavement thicknesses are shown in Figure 17.

The surfacing for the rigid pavements is discussed in Chapter 6.

Eight structural sections in each of the four major loops were included in a paved shoulder study. The shoulders of four of these sections were paved with asphaltic concrete, 3 in. thick (1½ in. surface and 1½ in. binder) and 6 ft wide. The remainder of the 10-ft shoulder was surfaced with 3 in. of Type A crushed stone. Figures 20, 21 and 22 show the layouts of structural sections in each rigid pavement tangent.

Details of transitions between structural sections are shown in Figure 23. Except in Loop 1, the thickness of the surfacing and subbase was extended 2 ft into the transition. Tapers from one thickness to the other were made in a dis-

Figure 19. Paving shoulders on rigid tangent.

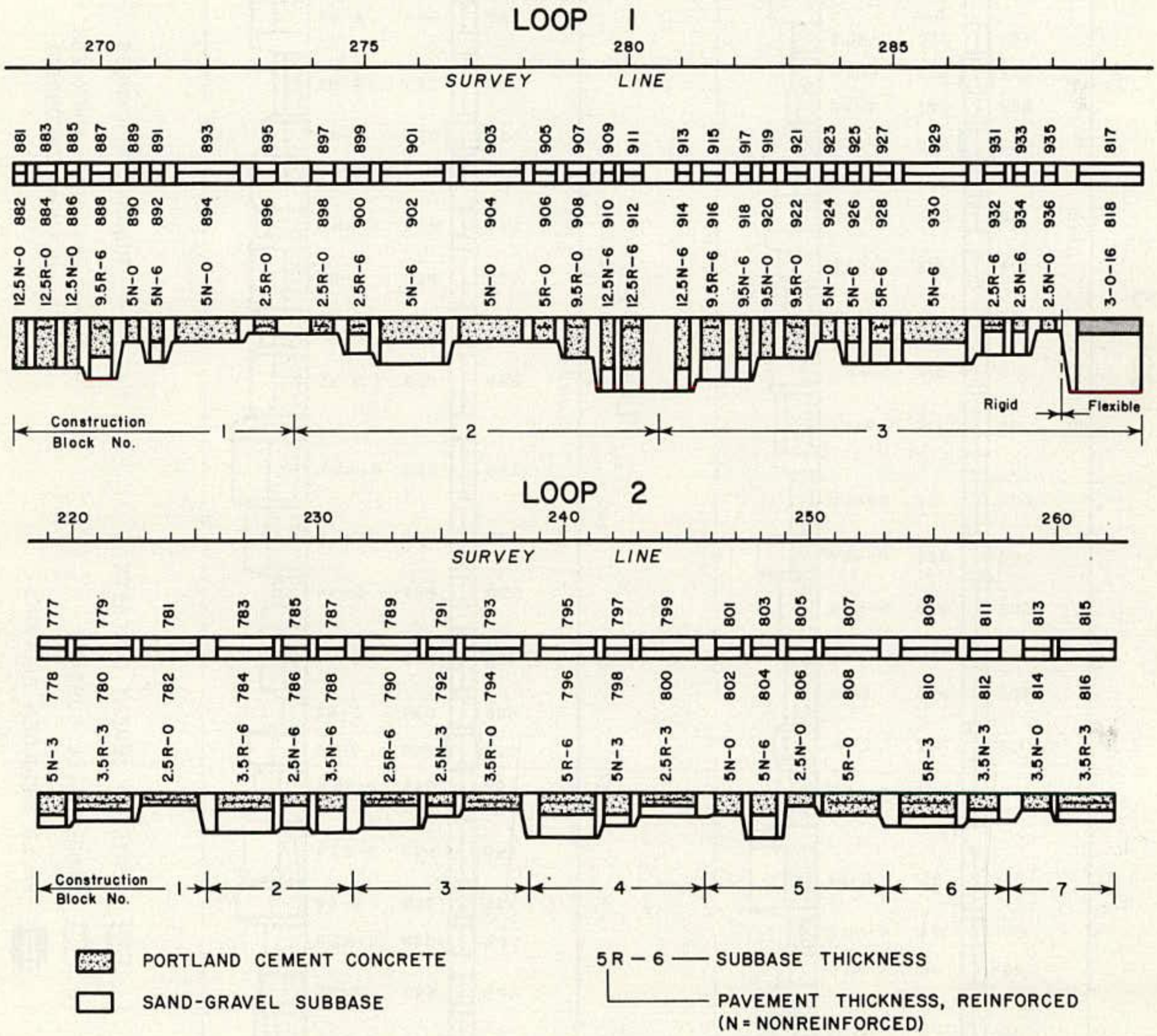


Figure 20. Layout of rigid tangent, Loops 1 and 2.

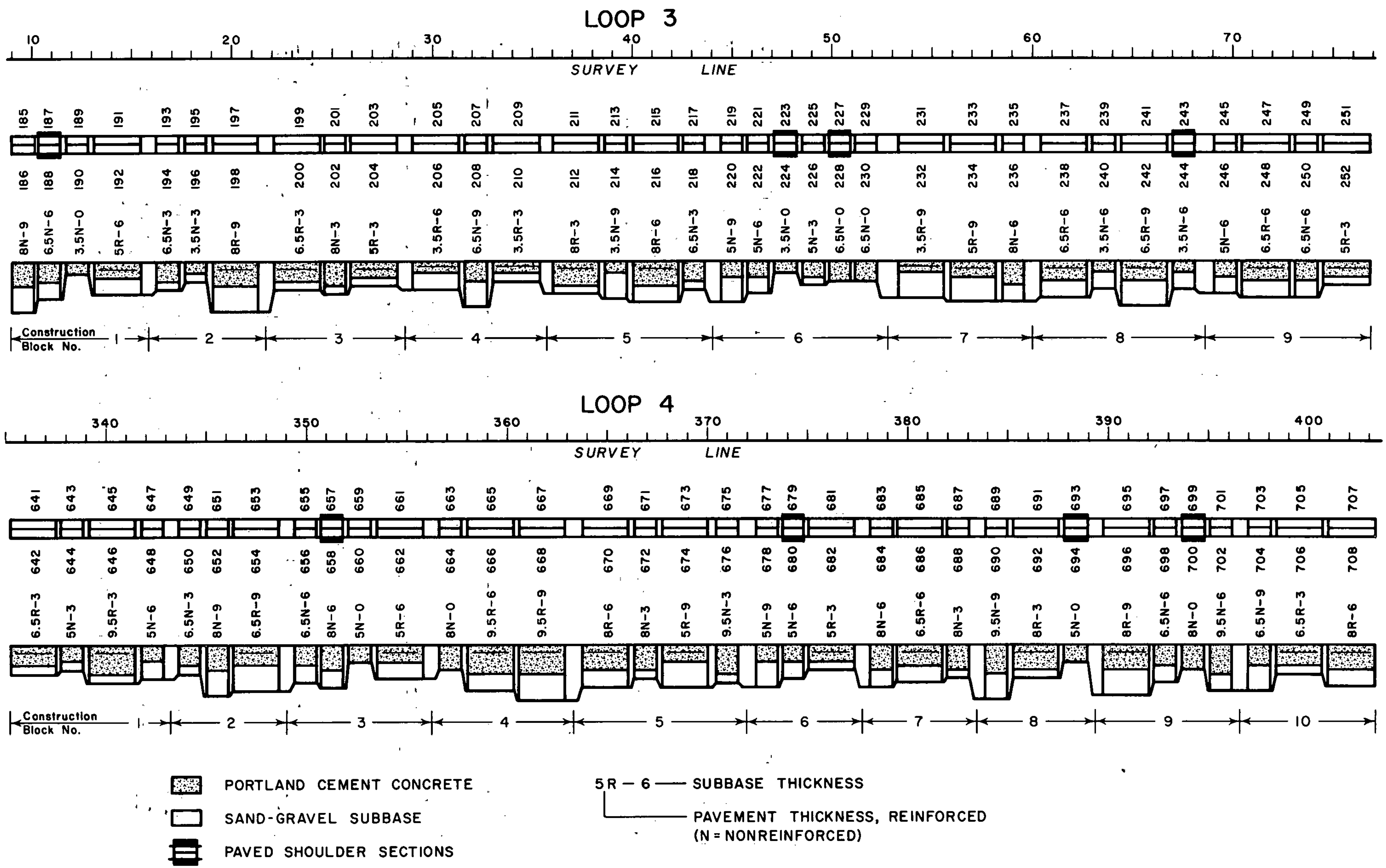


Figure 21. Layout of rigid tangent, Loops 3 and 4.

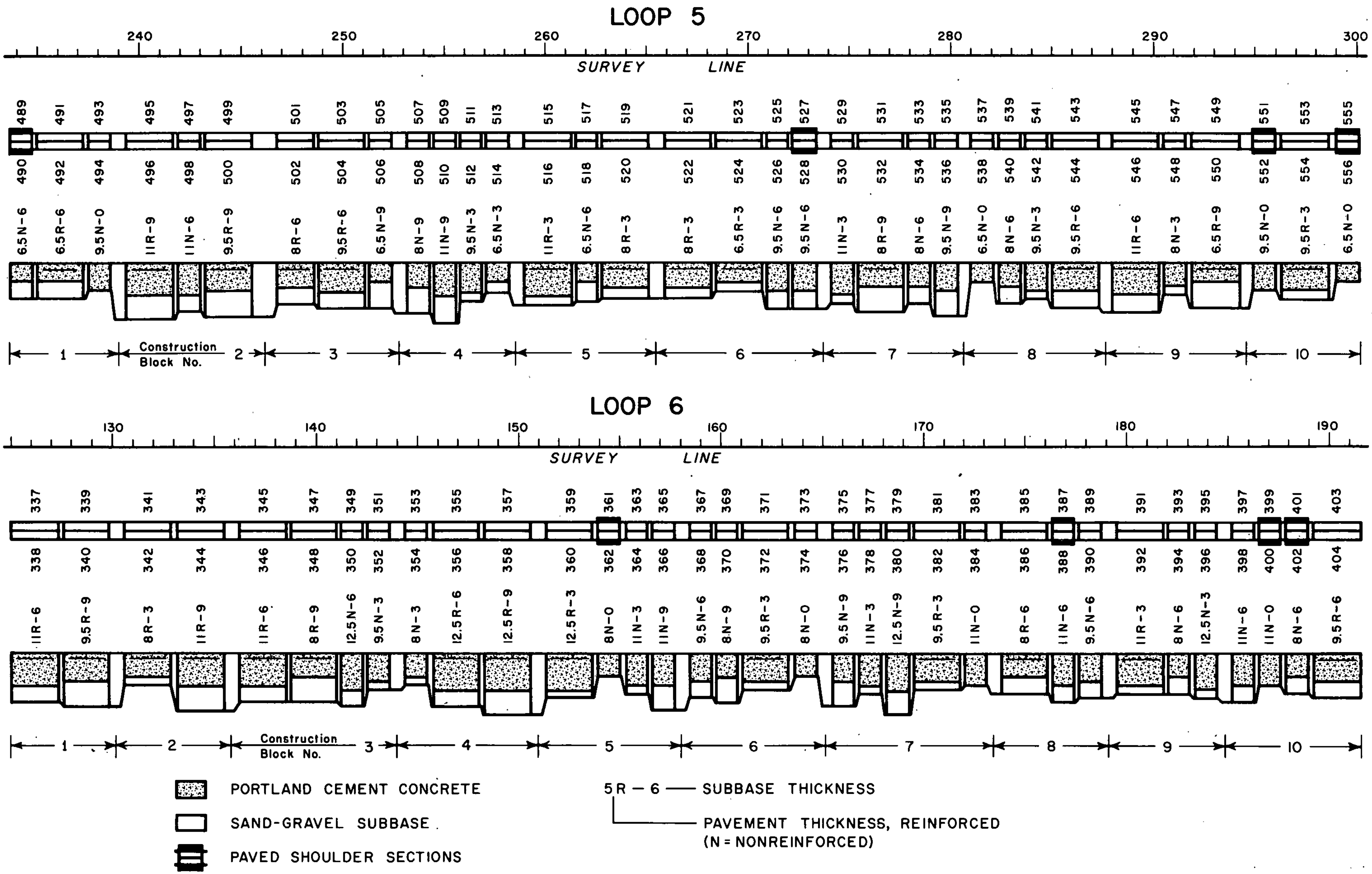


Figure 22. Layout of rigid tangent, Loops 5 and 6.

PAVEMENT THICKNESS	REINFORCEMENT BAR MAT	
	BAR SIZE Long. & Trans.	SPACING Long. & Trans.
$2\frac{1}{2}$"	No. 2	9"
$3\frac{1}{2}$	2	9
5	2	6
$6\frac{1}{2}$	2	5
8	3	9
$9\frac{1}{2}$	3	8
11	3	6
$12\frac{1}{2}$	3	6

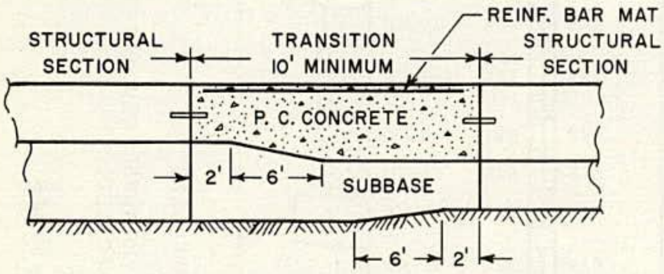


Figure 23. Details of transition pavement between structural sections.

tance of 6 ft. These transition pavements were reinforced with deformed bar mats. The sizes and spacings of the bars for the various thicknesses of pavement are also shown in Figure 19. Contraction joints with load transfer devices were provided at the ends. In Loop 1 the pavements of adjoining sections, including reinforcement if present, were extended into the transitions; the tapers from one thickness to the other occurred in the center 6 ft; and the deformed bar mat reinforcement was not used.

Special transition sections were constructed between flexible and rigid pavements. Details of these transitions are shown in Figure 24.

The sand-gravel material was removed from the outside shoulders of the rigid tangents, and equipment operated on this portion of the embankment during construction of the subbase and surfacing. Figure 16 shows the stage-construction line for the subbase. All construction to the left of this line was completed before starting construction on the outside shoulder. As with the flexible pavement, it was necessary to reprocess the top 6 in. of embankment and reshape the surface on outside shoulders.

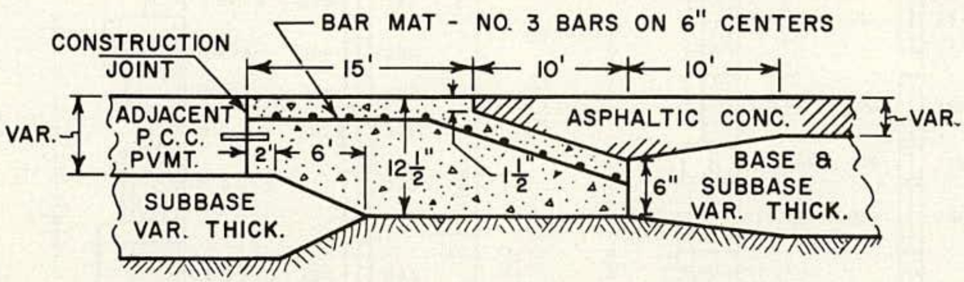


Figure 24. Details of transitions between rigid and flexible pavements.

Figure 25. Group of four test bridges.

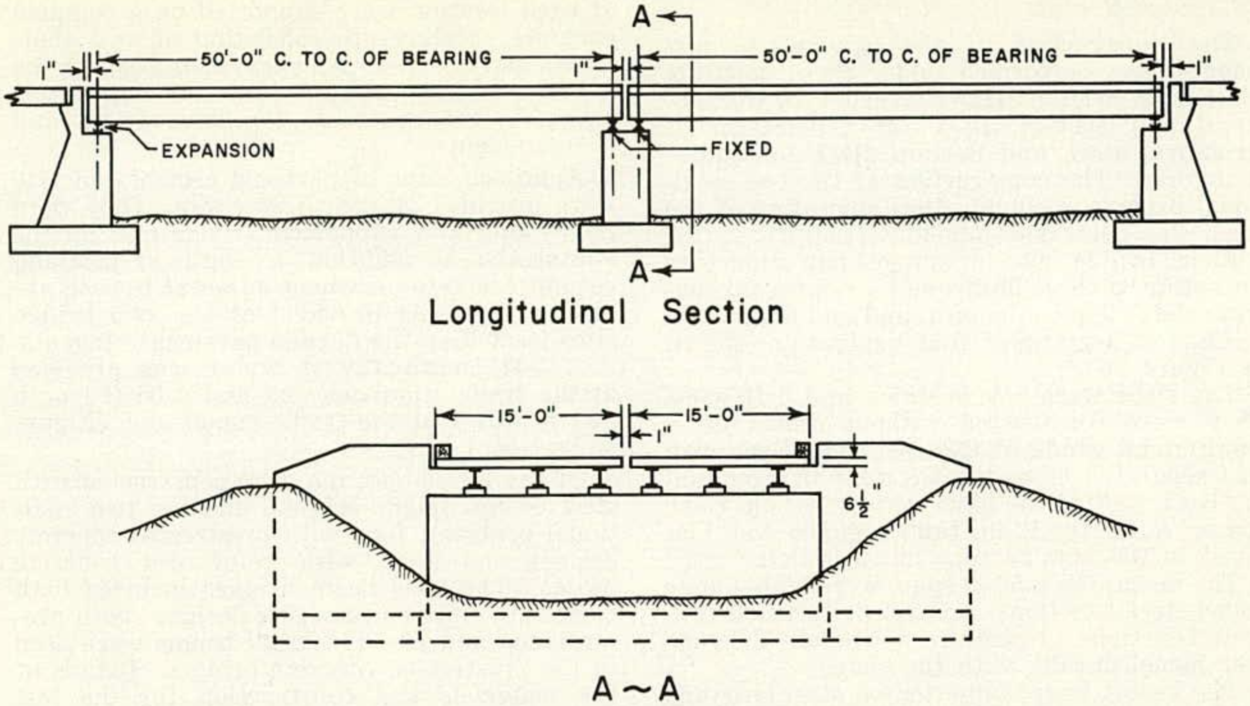


Figure 26. Typical bridge sections.

Figure 27. Underside of bridge.

1.3.4 Test Bridges

The construction of the original 16 test bridges was performed under three separate contracts: Section 2HB–Extension for the substructures, Section 2F–1 for fabrication of structural steel, and Section 2B–1 for superstructures. The construction of the two additional bridges, including the fabrication of the steel, was performed under Section 2B–2.

Each bridge was a simple-span structure consisting of three beams and a reinforced concrete slab. Typical longitudinal and transverse sections of a group of four bridges are shown in Figure 26.

The slabs were 6½ in. thick and 5 ft wide. They were constructed without crown on a longitudinal grade of 0.20 percent. Each slab was separated from the adjacent bridges and the back wall of the abutment by a 1-in. clear space. A 12- by 12-in. timber guard was fastened to the outside edge of each slab.

The beams, on a 50-ft span, were wide-flange rolled-steel I-sections, precast prestressed concrete I-sections, or reinforced concrete T-beams cast monolithically with the slab.

The beams were supported on steel bearings resting on the substructure. The four bridges at each location were supported on a common concrete substructure consisting of two abutments and one pier. An excavation beneath the superstructure was provided for convenient access to the underside for observations and measurements.

Approach slabs of portland cement concrete were provided at each bridge site. They were 20 ft long and supported at one end by the abutments. In addition, a length of portland cement concrete pavement adjacent to each approach slab was provided at the two bridge sites located on the flexible pavement tangents. A 145-ft length (28 ft wide) was provided at the traffic approach end and a 35-ft length (24 ft wide) at the traffic runoff end (Figure 4, Section 1.1.2).

Of the 18 bridges, ten were constructed with steel beams (eight original and the two additional bridges), four with prestressed concrete beams, and four with reinforced concrete beams. The steel beam bridges included both composite and noncomposite designs; both pretensioned and post-tensioned beams were used in the prestressed concrete bridges. Details of the materials and construction for the test bridges are given in Chapter 7.

Chapter 2

Embankment

This chapter describes the construction of the embankments on test tangents and discusses the specification requirements and procedures for both materials and construction control. It also includes summaries of data illustrating materials and construction control and the as-constructed characteristics of the embankments.

2.1 MATERIALS AND MATERIALS CONTROL

In the summer of 1956 the Road Test staff conducted a more thorough investigation of the three borrow areas containing the selected A–6 soils. Borings were made on 100-ft grids, and samples were taken at each 1-ft interval to a maximum depth of about 16 ft.

A typical boring log is shown in Figure 28. The borrow pits contained four distinct layers of material: a 12-in. layer of top soil, a layer of overburden soil, the yellow-brown clay A–6 soil, and the gray clay A–6 soil. There also was a zone of transition from yellow-brown to gray soil with physical characteristics generally the same as the yellow-brown material. This transition soil was used in the lower layers of the center portion of the A–6 soil embankments.

Routine classification tests made on the soil samples included optimum moisture, maximum density, liquid and plastic limits, specific gravity, and gradation and hydrometer analyses. Standard AASHO test procedures were followed.

Table 7 and Figures 31 and 32 summarize the test data for the yellow-brown A–6 soil (including transition soil) for each of the three borrow pits. Similarly, Table 8 and Figures 33 and 34 summarize the test data for the gray A–6 soil. These data were not summarized by borrow pits because of the limited number of tests. The major emphasis on soil testing was placed on the yellow-brown soil because it was used in that portion of the embankment supporting the test pavements.

The specifications provided for three classifications of borrow excavation. Borrow Excavation, Type A, was the 12-in. layer of top soil used to restore the pits to a farmable condition. Borrow Excavation, Type B, was the layer of overburden soil used to construct embankments outside the test loops and embankments within test loops below the 3-ft of A–6 soil embankment. Borrow Excavation, Type C, included the layers of yellow-brown and gray A–6 soils.

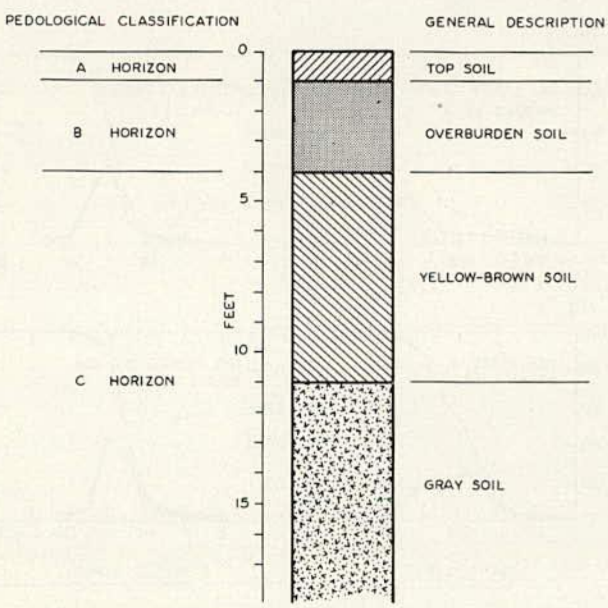


Figure 28. Typical boring log from borrow pit.

Figure 29. Drill rig in borrow pit.

Figure 30. Testing embankment soil.

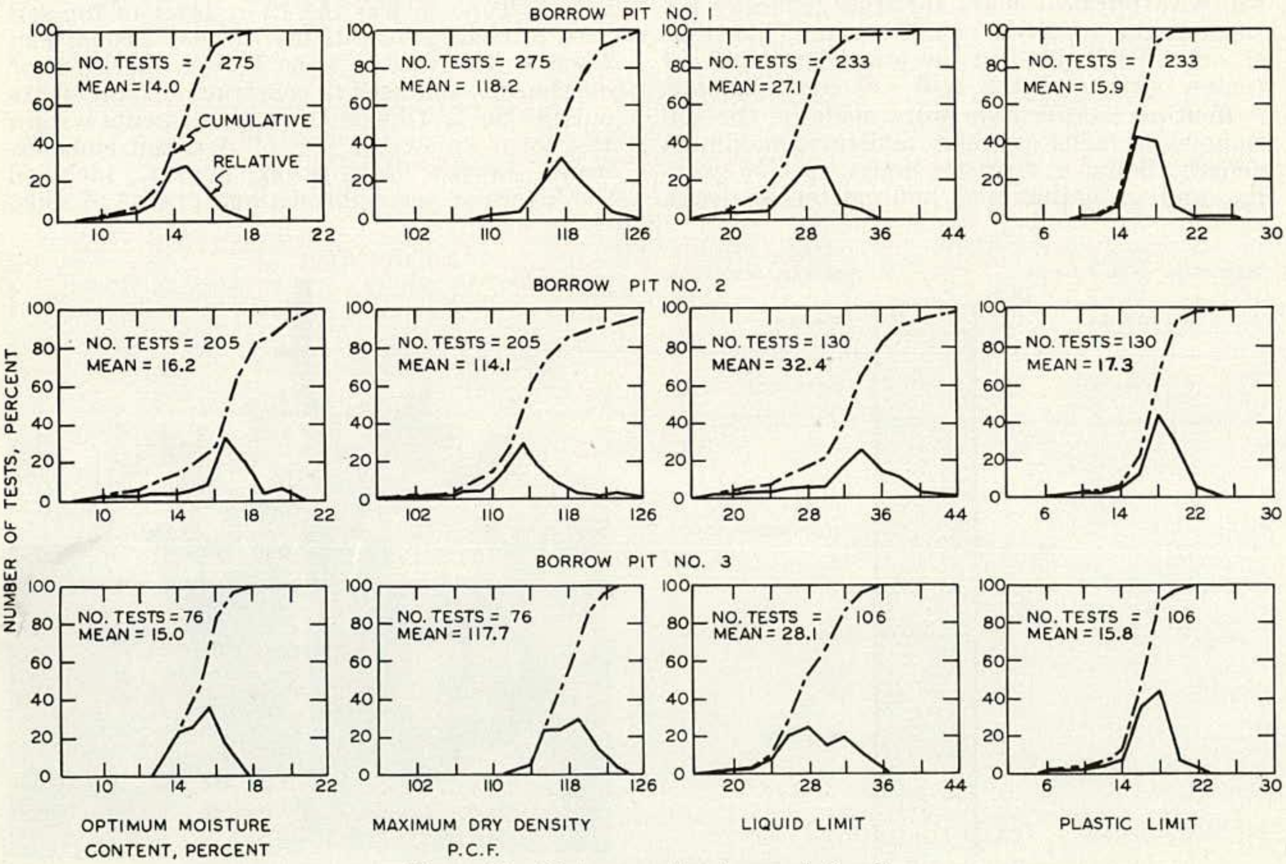


Figure 31. Tests on yellow-brown A-6 soil.

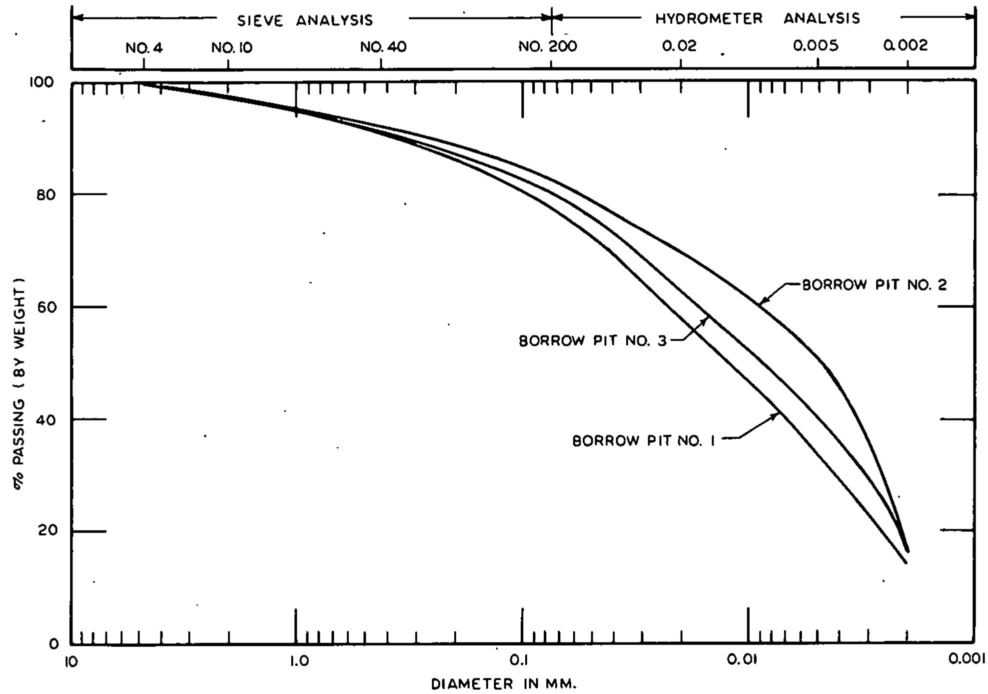


Figure 32. Grain size distribution curves, yellow-brown A–6 soil.

The layers of material were required to be excavated separately, and the material from one layer was not allowed to become mixed with material from any other layer. The intent of the specifications was that access to both the yellow-brown and the gray A–6 soils be maintained at all times during the construction of the top three feet of embankment within the test loops. The contractor was required to arrange his operations accordingly.

Four-wheel and 6-wheel self-propelled scrapers of 18- to 21-cu yd capacity were used to excavate and transport the soils from the borrow pits. The A–6 soils were excavated in 4- to 6-in. layers. Prior to excavation, bulldozers with scarifier teeth mounted along the cutting edge of the blades were used to break up the selected soils into pieces small enough to permit proper spreading on embankments. Interspersed pockets of silt and sand were removed and wasted.

2.2 CONSTRUCTION PROCEDURES

The construction specifications were intended to produce an exceptionally uniform embankment for all test sections. Consequently, the specified procedures and equipment, in some instances, were foreign to standard highway embankment construction.

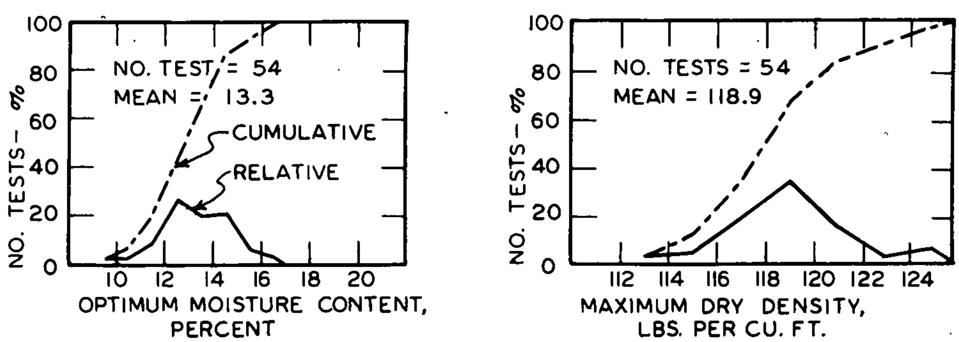


Figure 33. Tests on gray A–6 soil.

The specifications required that embankment be constructed in layers or lifts and that each layer be compacted at a moisture content within plus or minus two percentage points of optimum and to a density between 95 and 100 percent of maximum dry density determined by the Standard AASHO Procedure T99–49. The 1953 survey of state highway practices determined that the specified minimum density requirement was representative of national practice. The upper limit was established on the basis that it would be sufficiently restrictive for the research requirements, and that closer control probably was not possible.

Rotary speed mixers were required for in-place processing of the A–6 soil. Basically, the mixers consisted of a pump, a pressure spray bar and a rotary tiller mounted on a pneumatic-tired tractor. A hood was provided over the tiller to confine the material. The spray bar was placed in front of the hood so that the water was sprayed on the soil as it was being

TABLE 7

PHYSICAL CHARACTERISTICS OF YELLOW-BROWN A–6 SOIL
(Obtained from Data System 2330)

Soil Characteristic		Borrow Pit	Number of Tests	Sample Mean	Standard Deviation	Weighted Average[1]
Optimum moisture content (AASHO Designation: T99–49)		1	275	14.0	1.7	
		2	205	16.3	3.7	15.1
		3	76	15.0	1.0	
Maximum dry density (AASHO Designation: T99–49)		1	275	118.2	3.0	
		2	205	114.1	5.4	116.4
		3	76	117.7	2.5	
Liquid limit (AASHO Designation: T89–54)		1	233	27.1	3.6	
		2	130	32.4	5.0	29.4
		3	106	28.1	3.3	
Plastic limit (AASHO Designation: T90–54)		1	233	15.9	1.6	
		2	130	17.3	2.2	16.5
		3	106	15.8	2.1	
Plasticity index (AASHO Designation: T–91–54)		1	233	11.3	3.0	
		2	130	15.0	3.9	13.6
		3	106	12.0	2.7	
Specific gravity (AASHO Designation: T100–54)		1	63	2.71	0.032	
		2	59	2.72	0.045	2.71
		3	48	2.69	0.023	
Grain size determination (AASHO Designation: T88–54)	No. 4	1	57	99.3		99.0
		2	53	98.9		
		3	16	98.4		
	No. 10	1	57	96.4		96.8
		2	53	97.1		
		3	16	97.2		
	No. 40	1	57	90.2		91.0
		2	53	91.7		
		3	16	91.5		
	No. 60	1	57	85.8		87.7
		2	53	89.2		
		3	16	88.9		
	No. 200	1	57	77.9	5.7	80.6
		2	53	83.9	5.2	
		3	16	79.6	2.9	
	0.02 mm	1	43	58.6		62.8
		2	30	67.9		
		3	14	61.2		
	0.05 mm	1	43	33.8	8.6	42.3
		2	30	51.7	6.2	
		3	14	41.4	4.3	
	0.002 mm	1	43	13.6	7.3	15.3
		2	30	16.2	12.0	
		3	14	17.4	8.0	

[1] Approximate percentage of yellow-brown soil used from each borrow pit: Borrow Pit No. 1, 43 percent; No. 2, 41 percent; No. 3, 16 percent.

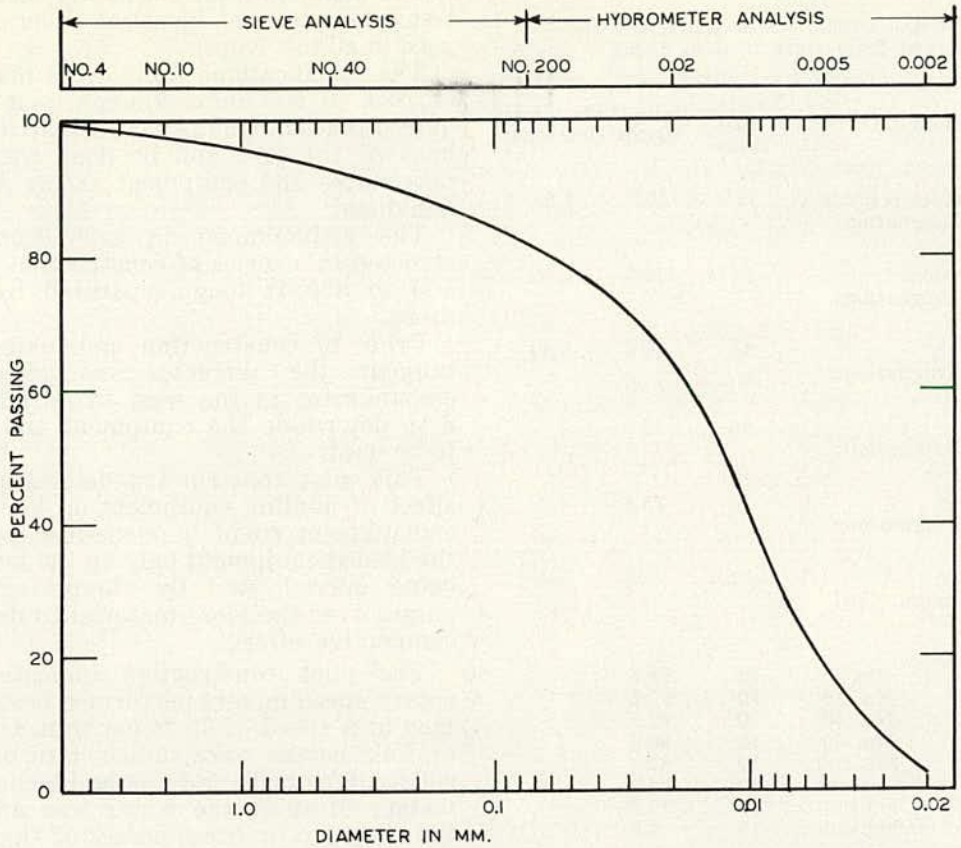


Figure 34. Grain size distribution curve, gray A–6 soil.

pulverized and mixed. The water was supplied by 1,000-gal tanks mounted on farm wagons and pulled by the mixers.

The mixers pulverized each lift of soil and blended it with the required amount of water. It was specified that the depth of mixing include the top inch of the previous lift in order to produce a homogeneous mass. The mixers also were required to operate on each side of the junction between yellow-brown and gray soil to avoid intermixing the two soils.

The specifications stipulated the use of conventional compaction equipment. However, the choice of roller or combination of rollers could

Figure 35. Restoring borrow pit to farmable condition.

TABLE 8

Physical Characteristics of Gray A–6 Soil
(Obtained from Data System 2330)

Soil Characteristic		Number of Tests	Sample Means	Standard Deviation
Optimum moisture content (AASHO Designation: T99–49)		54	13.3	1.89
Maximum dry density (AASHO Designation: T99–49)		54	118.9	2.92
Liquid limit (AASHO Designation: T89–54)		36	30 8	5.99
Plastic limit (AASHO Designation: T90–54)		36	15.1	1.75
Plasticity index (AASHO Designation: T91–54)		36	15.8	4.95
Specific gravity (AASHO Designation: T100–54)		10	2.75	—[1]
Grain size determination (AASHO Designation: T88–54)	No. 4	10	99.4	
	No. 10	10	97.2	
	No. 40	10	92.5	
	No. 60	10	90.0	
	No. 200	10	81.9	
	0.02 mm	10	64.0	
	0.005 mm	10	17.3	
	0.002 mm	10	2.0	

[1] Not calculated.

not be changed after beginning construction on test tangents, and identical rollers were to be used in all test loops.

The specifications limited the maximum size of rock in the embankments to 2-in., and required that embankment construction below the level of the A–6 soil be done with the same procedures and equipment as the A–6 soil embankment.

The embankment in each loop was constructed in a series of construction blocks, each 500 to 800 ft long, separated by transition areas.

Prior to constructing embankment on test tangents, the contractor constructed a block of embankment in the west turnaround of Loop 6 to determine the equipment and procedures to be used.

This pilot construction determined that the effect of hauling equipment on lower layers of embankment could be controlled by operating the loaded equipment only on the loose material being spread, and by staggering successive passes over the loose material to distribute the compactive effort.

The pilot construction indicated that the rotary speed mixers performed best when operated at a speed of 75 ft per min. Usually three or four passes were sufficient to pulverize the soil satisfactorily and mix and blend it with the water. Most of the water was added during the first two or three passes of the mixer, and the final pass was primarily for tilling the soil without adding water. The best results were

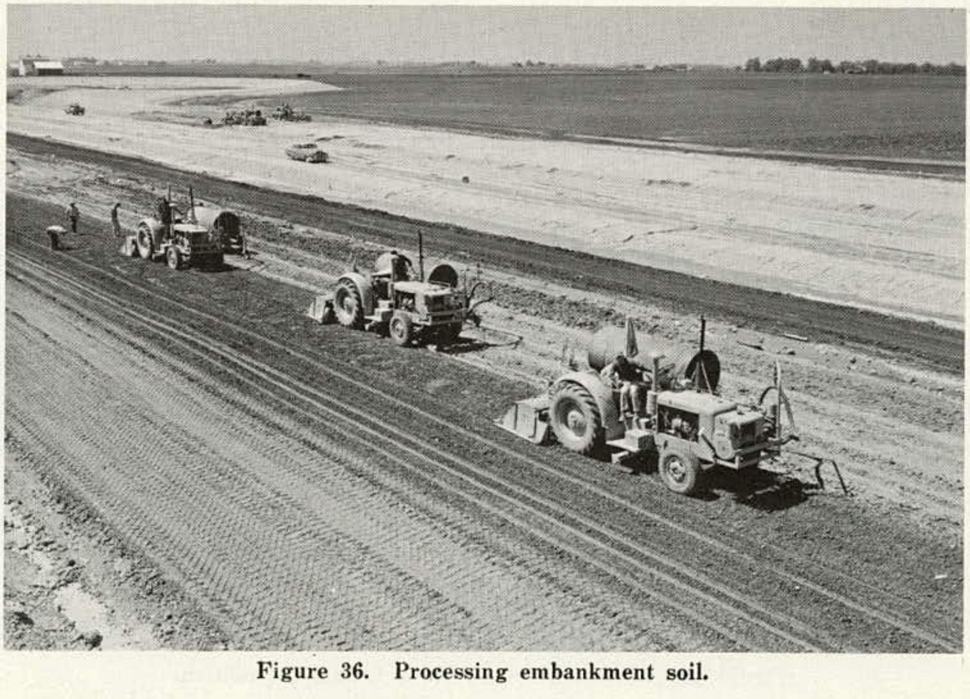


Figure 36. Processing embankment soil.

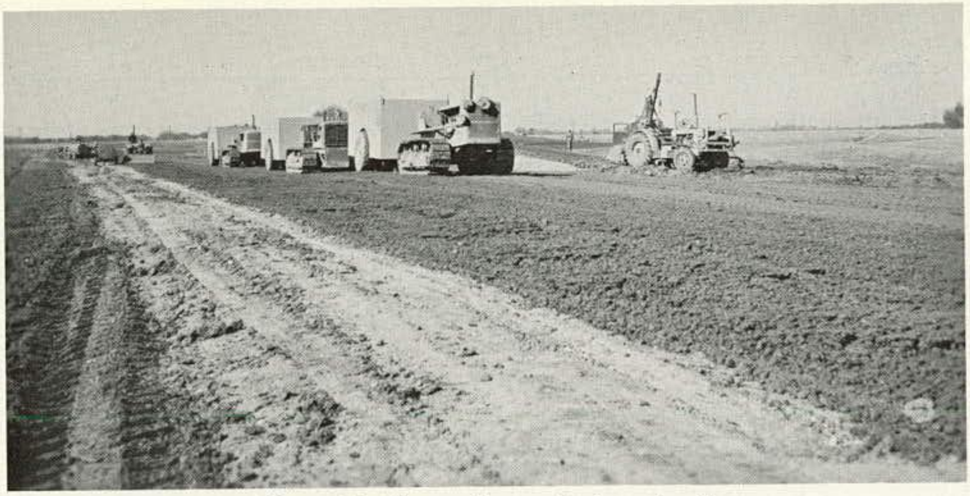


Figure 37. Compacting embankment soil.

obtained when the amount of water added on any one pass did not exceed 30 gal per min (approximately 2 gal per sq yd).

The experimental work with compaction equipment involved sheepsfoot and pneumatic-tired rollers. The absence of any precompaction in the pulverized soil caused the sheepsfoot roller to churn and aerate the soil without compacting it. The entire layer of soil being compacted tended to stick to the drum of the roller and be picked up. The best results were obtained with pneumatic-tired rollers. The rollers had four 18:00x25 24-ply tires on one axle and were pulled with track-type equipment. The inflation pressure of the tires was 70 psi, and the rollers were loaded to a gross weight of 38,000 lb. Generally, four passes of the roller were required to produce the specified density. Compaction was started along one edge of the embankment, and the roller was moved over approximately 1 ft on each succeeding pass. This pattern gave uniform coverage on all but the outer 6 ft, and was easy for the operator to follow as well as for the inspector to check.

As the 1956 construction season progressed it became necessary to reduce the weight of the rollers from 38,000 lb to between 33,000 and 34,000 lb. These reductions were caused by the embankment's becoming more rigid as it was built up and the heavier loads producing over-compaction. The moisture content of the underlying material was also an important factor influencing the required weight and number of passes of the rollers. During the fall of 1956 when the roller weights were being reduced, the subsoil was drying. In the spring of 1957 when the subsoil was wet, the rollers were operated at 33,000 to 34,000 lb but extra passes of the rollers were often required.

The embankment material was deposited in layers which, when compacted, were approximately 4 in. thick. Each layer, or lift, was tested and required to be within specification before construction of the next layer was started. Hauling equipment was required to enter and leave a block of embankment in the transitions provided at the ends. All turning movements were made in the transitions.

The limits of the yellow-brown material were established for depositing and spreading each layer of A–6 soil. Alternate layers were placed in opposite directions with the intention of balancing any effects of the earth-moving equipment on the density of underlying layers.

Each layer of soil was leveled by blade graders to the proper crown and grade, processed by the rotary speed mixers, and immediately compacted. The sequence of operations is shown in Figure 38.

Water was added through pressure spray bars directly to the soil as it was being pulverized. Technicians were easily trained to adjust the moisture content of the soil to within the desired limits by "feel." A technician followed each team of rotary speed mixers and controlled the rate at which water was added.

Frequent elevation checks were made on each block of embankment as it was constructed. For the upper 3 ft, grade stakes were set at the 1-ft level, the 2-ft level, and just prior to placing the final lift.

The final lift was shaped to the proper crown and grade, but was not processed and compacted until just prior to subgrading. During the 1956 season, this unprocessed layer was sprinkled at frequent intervals to preserve the moisture content of the underlying layers.

Embankment construction proceeded from west to east on all loops except Loop 3. Construction on Loop 3 began at the east end and proceeded westward due to lack of entry on its western-most property at the start of construction.

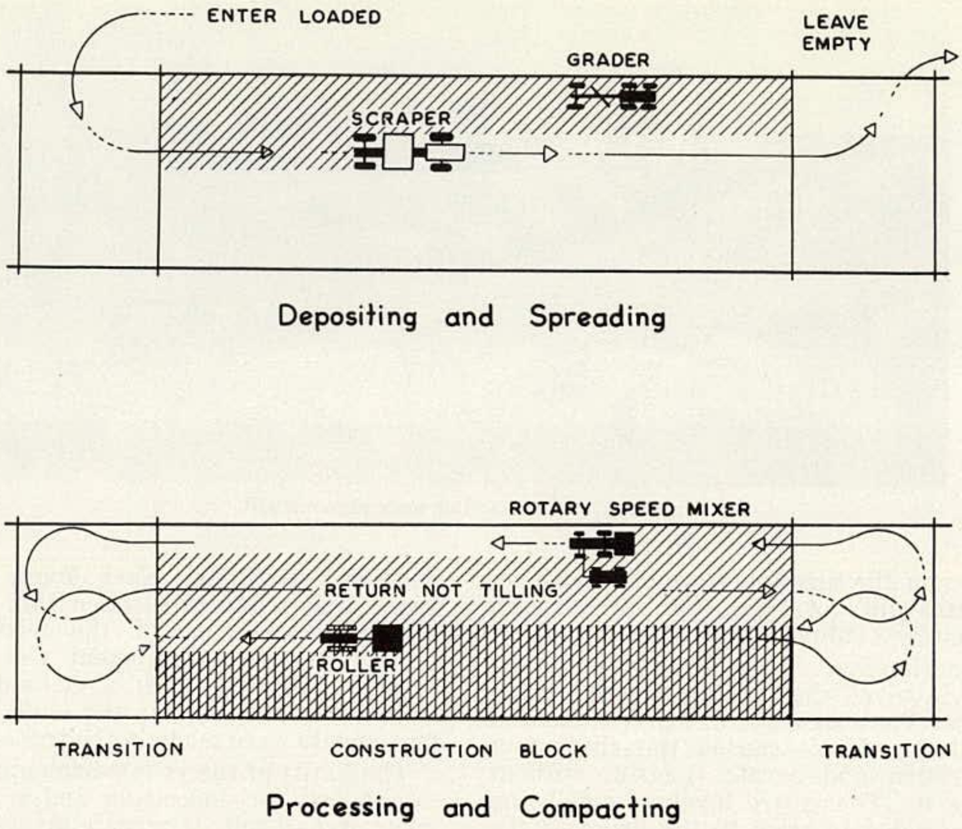


Figure 38. Equipment operation on embankment.

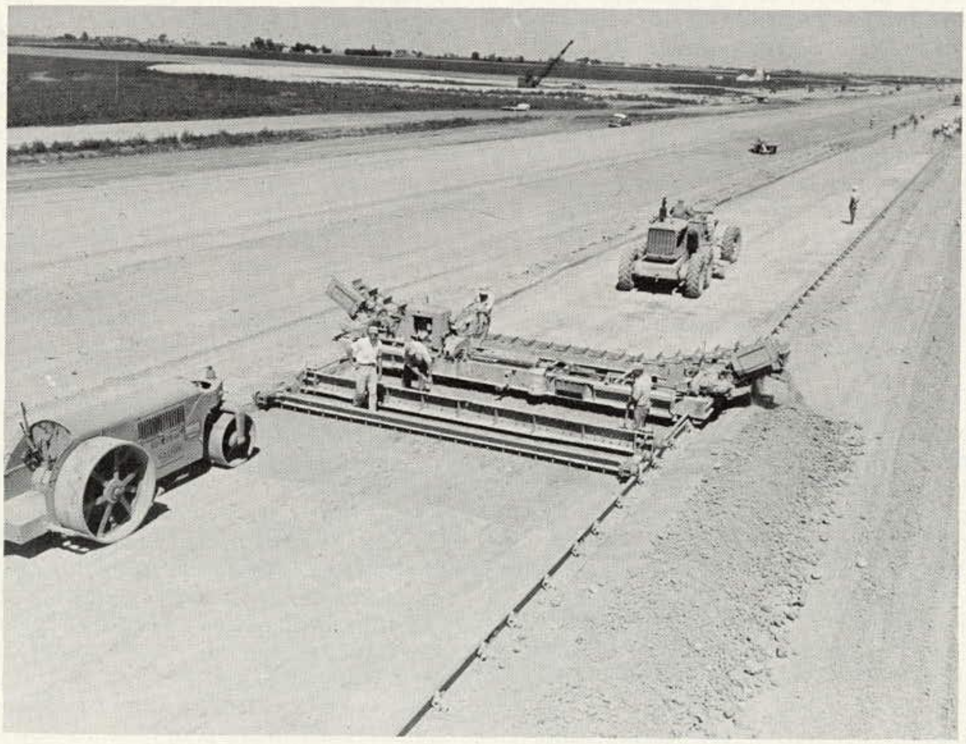


Figure 39. Subgrading top of embankment.

For efficient operation, it was necessary to have four to six blocks under construction in each loop at one time. This permitted the operations of placing, shaping, processing, and compacting to continue in some blocks while waiting for test results on others.

Embankment was constructed simultaneously in each of the four major loops. Construction in Loops 1 and 2 was started as soon as possible after the plans for these loops were completed.

Embankment construction within each loop involved the use of four to sixteen 4-wheel and 6-wheel self-propelled scrapers, two to nine rotary speed mixers, two to three blade graders, and numerous other items of miscellaneous equipment.

TABLE 9

A–6 EMBANKMENT CONSTRUCTION BY LOOPS

Const. Block No.[1]	LOOP 1		LOOP 2		LOOP 3		LOOP 4		LOOP 5		LOOP 6	
	Date Const.	Pit No.[2]	Date Const.	Pit No.[2]	Date Const.	Pit No.[2]	Date Const.	Pit No.[2]	Date Const.	Pit No.[2]	Date Const.	Pit No.[2]
F–1	5– 9–57 6– 6–57	2, 3	11– 1–56 5–24–57	1, 2	10–22–56 11– 1–56	1	9–18–56 9–28–56	2	9–14–56 9–27–56	2	9– 8–56 9–17–56	1
F–2	5– 6–57 6– 7–57	2, 3	11– 2–56 5–24–57	1, 2	10–16–56 10–25–56	1	9–29–56 10–12–56	2	9–24–56 10– 8–56	2	9–10–56 9–19–56	1
F–3	5– 8–57 6–10–57	2, 3	11– 3–56 5–16–57	2	10–11–56 10–22–56	1	10– 4–56 10–17–56	2, 3	9–27–56 10–10–56	2	9–18–56 9–24–56	1
F–4			11– 9–56 5–16–57	2	10–10–56 10–20–56	1	10– 5–56 10–19–56	2, 3	9–28–56 10–11–56	2	9–20–56 10– 4–56	1
F–5			11–10–56 5–27–57	2	9–29–56 10–13–56	1	10– 6–56 10–20–56	2, 3	10– 6–56 10–19–56	2	9–22–56 10– 4–56	1
F–6			11–19–56 5–27–57	2	9–26–56 10– 9–56	1	10–19–56 10–30–56	3	10–12–56 10–22–56	2	9–26–56 10– 6–56	1
F–7			11–19–56 5–16–57	2	9–25–56 10– 6–56	1	10–20–56 10–31–56	3	10–12–56 10–23–56	2	10– 2–56 10–17–56	1
F–8					9–24–56 10– 4–56	1	10–25–56 11–12–56	3	10–17–56 10–25–56	2	10– 5–56 10–18–56	1
F–9					9–15–56 9–25–56	1	10–30–56 11–12–56	3	10–24–56 11– 5–56	2	10– 9–56 10–22–56	1
F–10					9–12–56 9–20–56	1	11– 1–56 11–14–56	3	10–29–56 11– 9–56	2, 3	10–16–56 10–25–56	1
R–1	11–17–56 6– 6–57	2, 3	10–27–56 5–24–57	1, 2	10–20–56 10–30–56	1	9–17–56 9–29–56	2	9–10–56 9–24–56	2	8–30–56 9– 8–56	1
R–2	11–19–56 6– 6–57	2, 3	11– 2–56 5–27–57	2	10–13–56 10–24–56	1	9–27–56 10– 8–56	2	9–12–56 9–26–56	2	9– 1–56 9–13–56	1
R–3	5– 7–57 6–10–57	2, 3	11– 3–56 5– 8–57	2	10–11–56 10–23–56	1	10– 2–56 10–17–56	2, 3	9–29–56 10–16–56	2	9–14–56 9–25–56	1
R–4			11– 5–56 5– 8–57	2	10– 9–56 10–22–56	1	10– 4–56 10–18–56	2, 3	10– 2–56 10–17–56	2	9–19–56 10– 2–56	1
R–5			11– 5–56 5–16–57	2	10– 8–56 10–22–56	1	10–10–56 10–20–56	2, 3	10– 4–56 10–17–56	2	9–24–56 10– 4–56	1
R–6			11–19–56 5–16–57	2	9–29–56 10–13–56	1	10–16–56 10–25–56	3	10–10–56 10–22–56	2	9–27–56 10–10–56	1
R–7			11–13–56 5–16–57	2	9–21–56 10– 5–56	1	10–16–56 10–25–56	3	10–16–56 10–25–56	2	10– 3–56 10–17–56	1
R–8					9–20–56 10– 1–56	1	10–22–56 11– 2–56	3	10–19–56 10–31–56	2	10– 4–56 10–18–56	1
R–9					9–19–56 10– 1–56	1	11– 1–56 11–19–56	3	10–24–56 11– 8–56	2, 3	10– 6–56 10–19–56	1
R–10							11– 2–56 11–16–56	3	10–29–56 11– 9–56	2, 3	10–13–56 10–25–56	1

[1] Construction blocks numbered consecutively from west to east; F = flexible tangent, R = rigid tangent.
[2] Source of yellow-brown A–6 soil.

At the peak of operation nearly 25,000 cu yd of A–6 soil were placed in a 10-hr day, using over 300 pieces of equipment.

A summary of construction by blocks in each test loop is given in Table 9. The dates cover the time from the start of construction of the A–6 embankment through placing and shaping the top lift.

Because it was necessary to control the thicknesses of the structural layers of pavement to within close tolerances, the specifications required that the center 24 ft of embankment be finished with a mechanical subgrading machine operating on steel forms. The remainder was brought to the correct lines, grades and cross-sections by motor graders.

It was intended that the embankment moisture content and density be maintained during subgrading. Thus, the specifications prohibited blading material from high areas to low areas. Whenever it was necessary to add material to low areas, the entire top layer of embankment of the structural section or sections containing the low area was scarified, reprocessed and recompacted after the material was added.

In all sections where pavement design did not include a subbase, the embankment was subgraded approximately ¾ in. higher than required and brought to the final elevation just prior to the placement of the base course or surfacing. This provision assured that the sand-gravel material placed over the embankment as a protective covering would later be completely removed.

The specifications required the portion of the embankment finished with a mechanical subgrading machine to be tested with a template riding on the forms, and to show no surface variations in excess of ⅛ in. This permitted close control of thicknesses of subsequent pavement component layers.

Since the mechanical subgrader was designed primarily to shape granular material, experiments were necessary to establish procedures for its operation on the A–6 soil. The experimental work, conducted on one construction block of Loop 2, indicated that the surface must be allowed to "dry-back" to facilitate cutting to a smooth and tight finish. Consequently, processing and compacting the top lifts of embankment preceded subgrading by at least five days. Excessive "flaking" and "chunking" of the soil during subgrading was traced to track-type equipment and was corrected by using rubber-tired tractors to pull the rollers for compaction of the final lifts. Better results were obtained at slower speeds, and the forward speed of the subgrader was reduced with block and tackle arrangements. Six to 8-ton 3-wheel steel rollers were employed to seal the small amount of fine material that sifted beneath the planner of the subgrader.

It was evident during the trial subgrading that the subgrader must make thin cuts (¼ to ½ in.). Thus more than one pass was required to complete the operation. Blade graders and small self-propelled pneumatic-tired scrapers operated ahead of the subgrader to remove most of the excess material.

Subgrading operations proceeded from west to east on all loops. Table 10 gives the dates of processing and compacting top lifts and subgrading the embankment on all loops. Two subgrading outfits were used.

The grading specifications required the sand-gravel mulch (subbase material) to be placed on the embankment immediately after subgrading. However, the change in specifications (discussed in Section 1.2.3) caused a delay in obtaining the material. To prevent a further delay in the grading, the contractor was permitted to continue subgrading, but was required to maintain the desired moisture conditions of the embankment by sprinkling.

Water trucks, equipped with pressure spray nozzles, applied the water from the outside shoulders. The water was sprayed over the surface of the embankment with the heaviest concentration on the center portion. Nearly 3 million gallons of water were applied to the subgraded embankment before they were covered with the sand-gravel mulch.

2.3 CONSTRUCTION CONTROL

Construction control required extensive testing with a minimum of delay in construction operations. As a result, several special testing methods were developed which were either modifications or complete changes in standard AASHO procedures. Assembly-line techniques in the laboratory and radio communication with field crews also helped to speed the testing operation.

A continuous oven was developed for drying samples for moisture determinations. It consisted of an endless chain which passed the weighed wet samples under a battery of infrared heat lamps. Each sample took 23 min to dry, and samples could be introduced into the oven at a rate of 4 per min. Over 70,000 samples were dried in this oven during the various construction phases, and as many as 1,200 were dried in one day.

A radio-dispatched messenger delivered samples for field density determinations to the laboratory, and kept the field crews supplied with sample tubes and miscellaneous equipment.

There were 99 construction blocks in the test tangents and 28 in the turnarounds of the six test loops. In each block, the embankment generally was composed of nine lifts of A–6 soil. Each block-lift was tested and approved before construction of the next lift was started.

TABLE 10

CONSTRUCTION PROGRESS FOR SUBGRADING A–6 EMBANKMENT

Const. Block No.[1]	Date Top Lift Processed and Compacted						Date Center 24 Ft Subgraded					
	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6
F–1	7–24–57	6–21–57	7–26–57	7–29–57	7– 2–57	7–12–57	7–30–57	6–24–57	8– 5–57	8– 5–57	7–10–57	7–26–57
F–2	7–24–57	6–20–57	7–27–57	7–29–57	7–12–57	7–20–57	8– 2–57	6–27–57	8– 6–57	8– 6–57	7–16–57	7–29–57
F–3	7–25–57	6–20–57	7–27–57	7–30–57	7– 2–57	7–20–57	8– 3–57	7– 1–57	8– 6–57	8– 7–57	7–17–57	7–29–57
F–4		6–20–57	7–29–57	7–31–57	7–10–57	7–24–57		7– 3–57	8– 7–57	8– 8–57	7–22–57	7–30–57
F–5		6–27–57	7–29–57	7–31–57	7–17–57	7–24–57		7– 5–57	8– 7–57	8– 8–57	7–24–57	7–30–57
F–6		7– 6–57	7–30–57	8– 1–57	7–19–57	7–24–57		7– 8–57	8– 7–57	8– 9–57	7–25–57	7–31–57
F–7		7– 1–57	7–30–57	8– 2–57	7–20–57	7–25–57		7– 8–57	8– 8–57	8–10–57	7–26–57	7–31–57
F–8			7–31–57	8– 5–57	7–22–57	7–25–57			8– 8–57	8–12–57	7–27–57	8– 1–57
F–9			7–31–57	8– 5–57	7–22–57	7–25–57			8– 9–57	8–12–57	7–29–57	8– 2–57
F–10			8– 1–57	8– 6–57	7–24–57	7–26–57			8– 9–57	8–13–57	7–29–57	8– 2–57
R–1	7–24–57	7– 1–57	8– 1–57	7–27–57	7– 1–57	7– 8–57	7–30–57	7– 9–57	8–12–57	8– 6–57	7– 9–57	7–17–57
R–2	7–25–57	7– 1–57	8– 5–57	7–29–57	7– 1–57	7– 9–57	8– 2–57	7–10–57	8–13–57	8– 6–57	7–10–57	7–17–57
R–3	7–26–57	7– 2–57	8– 5–57	7–30–57	7– 2–57	7– 9–57	8– 3–57	7–10–57	8–13–57	8– 7–57	7–11–57	7–18–57
R–4		7– 2–57	8– 6–57	7–31–57	7– 9–57	7–10–57		7–11–57	8–15–57	8– 7–57	7–15–57	7–18–57
R–5		7– 3–57	8– 6–57	8–13–57	7– 9–57	7–10–57		7–12–57	8–15–57	8–19–57	7–16–57	7–19–57
R–6		7– 5–57	8– 6–57	8– 2–57	7–12–57	7–11–57		7–15–57	8–16–57	8– 9–57	7–17–57	7–22–57
R–7		7– 5–57	8– 7–57	8– 2–57	7–15–57	7–18–57		7–15–57	8–16–57	8– 9–57	7–24–57	7–24–57
R–8			8– 7–57	8– 3–57	7–15–57	7–18–57			8–19–57	8–10–57	7–24–57	7–25–57
R–9			8– 7–57	8– 5–57	7–18–57	7–19–57			8–19–57	8–12–57	7–25–57	7–25–57
R–10				8– 6–57	7–20–57	7–19–57				8–13–57	7–26–57	7–25–57

[1] Construction blocks in each loop numbered from west to east for each flexible tangent (F) and rigid tangent (R).

In most cases, all tests were made and the recommendation for acceptance or rejection was returned to the field within 90 min after the completion of rolling operations on a block-lift. As many as 25 block-lifts were completed in one day. More than 8,000 field density tests and 4,000 maximum density tests were made during the construction of the A–6 embankment on test tangents.

Two field density tests and one maximum

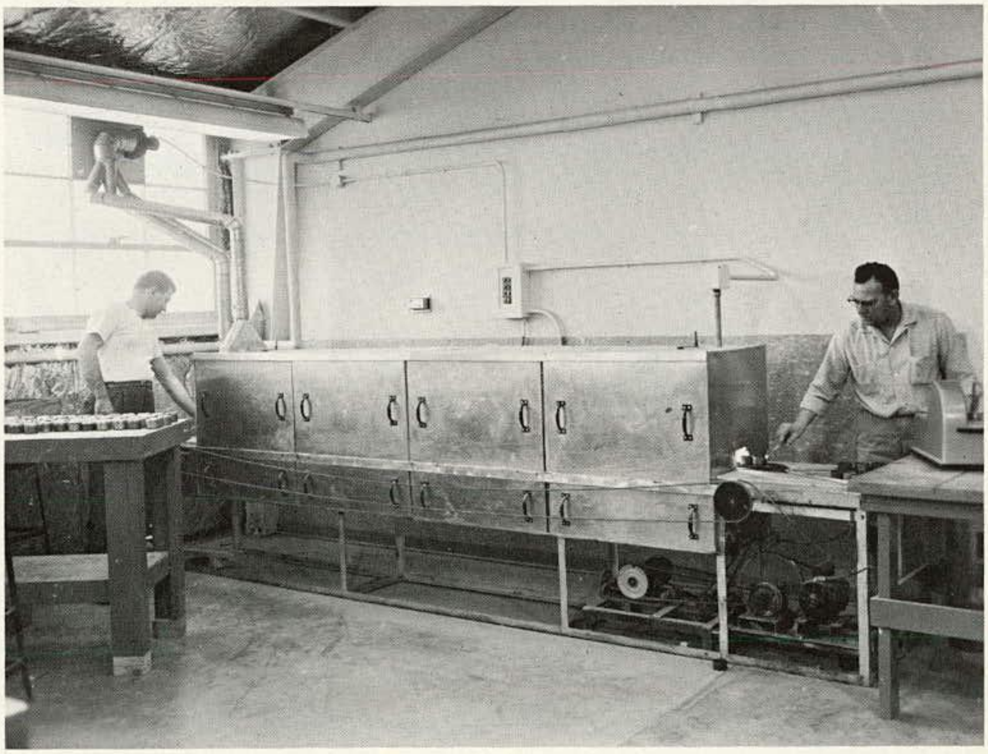


Figure 40. Continuous drying oven.

Figure 41. Obtaining soil sample for field density.

density test were made for each lift in each 100-, 120-, and 160-ft and in one-half of the 240-ft structural sections, making a total of six to twelve field density tests and three to six maximum density tests in each block-lift. Each field density test was the average of the densities of two samples. To obtain the percent compaction, the individual field densities were divided by the block-lift average maximum density.

The decision to accept or reject a block-lift was based on a statistical analysis of the com-

Figure 42. Extruding soil from tube and obtaining sample for moisture determination.

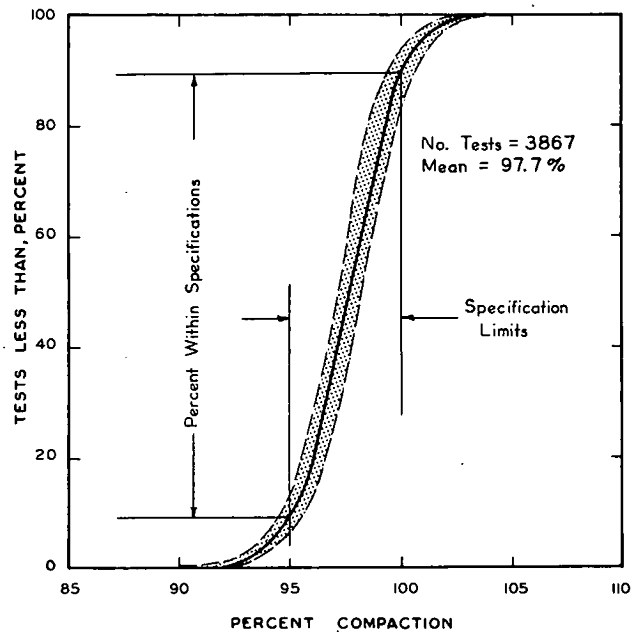


Figure 43. Embankment construction, flexible tangents.

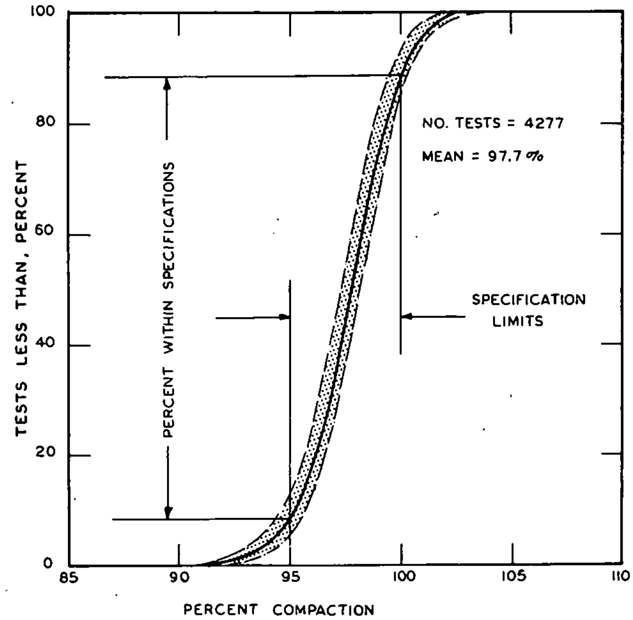


Figure 44. Embankment construction, rigid tangents.

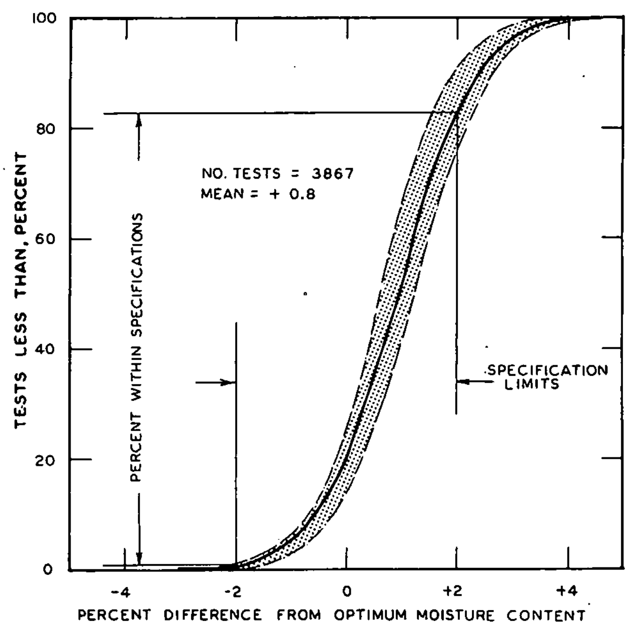


Figure 45. Constructed moisture content of embankment, flexible tangents.

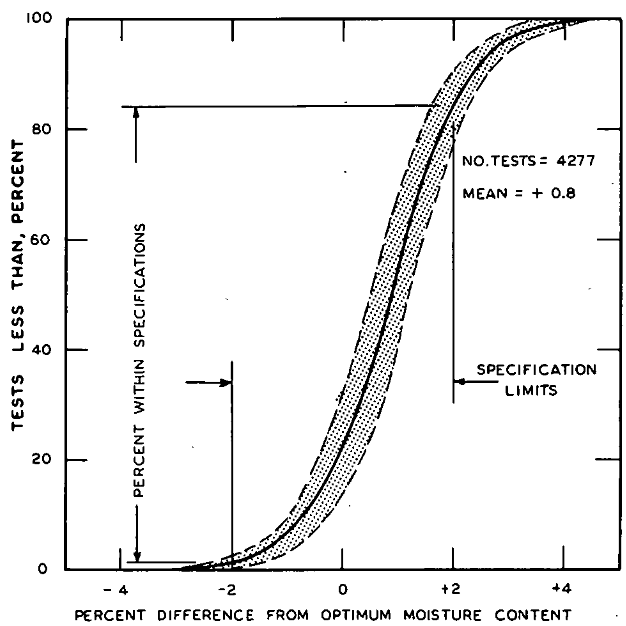


Figure 46. Constructed moisture content of embankment, rigid tangents.

paction data for that lift. The analysis involved computing the mean percent compaction and the standard deviation. From these computations the percentage of tests out of the specified limits was estimated. An acceptable percentage-out was selected after the job had been running for a short time. Acceptance was based on an allowable estimated 45 percent-out for individual block-lifts. Top lifts were held to 40 percent.

It should be noted that these percentages do not refer to individual tests, but represent estimates of the compacted condition of the entire block-lift determined by a statistical analysis of the results of only a few tests on that lift. It is necessary that the maximum permissible estimated value be higher than would be permitted if it were possible to test the entire area. Actually, with this method of acceptance control, more than 80 percent of individual field

TABLE 11

SUMMARY OF FIELD CONSTRUCTION DATA
(Obtained from Data System 2310)

Loop	Block	Flexible Tangent: Optimum Conditions[1] Moist. Cont. (%)	Flexible Tangent: Optimum Conditions[1] Dry Dens. (pcf)	Flexible Tangent: Field Data[2] No. Tests	Flexible Tangent: Field Data[2] Moist. Cont. (%)	Flexible Tangent: Field Data[2] Dry Dens. (pcf)	Flexible Tangent: Field Data[2] M.C. (% from opt.)	Flexible Tangent: Field Data[2] Comp. (%)	Rigid Tangent: Optimum Conditions[1] Moist. Cont. (%)	Rigid Tangent: Optimum Conditions[1] Dry Dens. (pcf)	Rigid Tangent: Field Data[2] No. Tests	Rigid Tangent: Field Data[2] Moist. Cont. (%)	Rigid Tangent: Field Data[2] Dry Dens. (pcf)	Rigid Tangent: Field Data[2] M.C. (% from opt.)	Rigid Tangent: Field Data[2] Comp. (%)
1	1	15.8	114.9	88	17.0	111.1	+1.2	96.6	15.7	115.3	70	17.0	111.8	+1.3	97.0
	2	15.4	115.9	99	16.3	112.5	+0.9	97.1	15.4	115.7	89	16.5	112.3	+1.1	97.1
	3	15.8	114.6	132	16.9	111.5	+1.1	97.3	16.0	114.2	128	17.0	111.2	+1.0	97.4
2	1	16.4	113.5	89	17.2	110.3	+0.8	97.5	16.1	114.5	90	16.9	111.1	+0.8	97.0
	2	16.0	113.6	72	17.2	110.4	+1.2	97.2	16.5	113.5	72	17.5	109.9	+1.0	96.8
	3	16.2	113.7	107	17.6	110.2	+1.4	96.9	15.9	115.1	90	17.2	111.1	+1.3	96.6
	4	16.1	114.4	89	17.0	111.4	+0.9	97.3	16.6	113.0	79	18.0	110.0	+1.4	97.4
	5	16.3	113.9	107	17.0	111.3	+0.7	97.7	16.1	113.9	90	16.9	111.3	+0.8	97.7
	6	16.1	114.1	63	17.3	110.7	+1.2	97.0	16.3	113.2	53	17.6	110.6	+1.3	97.7
	7	16.0	113.9	63	17.5	110.5	+1.5	97.0	16.6	113.3	47	18.0	110.3	+1.4	97.3
3	1	15.3	116.0	53	16.4	113.6	+1.1	97.9	15.4	115.4	90	16.1	113.3	+0.7	98.2
	2	15.3	116.1	53	16.0	113.7	+0.7	97.9	15.2	116.0	71	16.5	113.1	+1.3	97.4
	3	15.5	116.1	54	16.1	113.5	+0.6	97.8	15.5	116.2	90	16.1	114.0	+0.6	98.1
	4	15.4	115.2	89	16.1	113.2	+0.7	98.3	15.2	116.0	100	16.0	114.3	+0.8	98.5
	5	14.9	116.4	89	15.5	113.8	+0.6	97.8	15.3	116.2	120	16.3	112.9	+1.0	97.2
	6	14.1	118.5	89	14.6	115.8	+0.5	97.7	14.7	116.8	106	15.4	114.6	+0.7	98.2
	7	13.9	118.1	89	14.3	116.3	+0.4	98.5	14.2	117.7	89	14.4	116.0	+0.2	98.6
	8	14.1	118.5	108	14.2	116.4	+0.1	98.2	14.5	117.6	107	14.7	115.5	+0.2	98.3
	9	14.5	116.6	64	15.0	114.6	+0.5	98.3	14.3	117.7	108	14.5	115.2	+0.2	97.9
	10	14.0	115.8	52	15.1	114.0	+0.3	98.4	...	....	...	...	....	....	...
4	1	16.5	112.4	55	17.0	111.0	+0.5	98.7	16.7	112.7	107	16.8	111.3	+0.1	98.8
	2	15.9	113.7	89	16.6	111.4	+0.7	98.0	15.9	113.2	72	17.3	111.4	+1.4	98.4
	3	15.7	115.3	90	16.5	112.4	+0.8	97.5	15.7	114.3	90	16.8	112.4	+1.1	98.3
	4	15.8	114.9	54	16.6	112.4	+0.8	97.8	16.1	114.0	90	17.0	111.3	+0.9	97.6
	5	15.1	116.5	90	16.0	113.9	+0.9	97.8	15.8	115.3	108	16.6	112.4	+0.8	97.5
	6	15.6	115.7	100	16.1	112.8	+0.5	97.5	15.5	115.8	80	16.2	113.1	+0.7	97.7
	7	15.2	116.3	108	16.2	113.2	+1.0	97.3	15.4	116.4	80	16.2	113.4	+0.8	96.6
	8	15.5	116.2	72	16.5	112.6	+1.0	96.9	15.4	115.6	72	16.7	112.1	+1.3	97.0
	9	15.3	116.2	54	16.2	113.9	+0.9	98.0	15.3	115.4	90	16.2	113.3	+0.9	98.2
	10	15.4	115.5	54	16.5	112.7	+1.1	97.6	15.2	116.6	90	16.4	113.5	+1.2	97.4
5	1	16.4	112.5	90	17.4	110.4	+1.0	98.1	16.9	111.9	70	17.2	109.1	+0.3	97.5
	2	16.0	113.3	90	17.0	110.7	+1.0	97.7	16.4	112.2	90	16.8	109.4	+0.4	97.5
	3	16.2	116.3	54	17.1	111.4	+0.9	98.4	16.3	112.8	100	17.5	109.8	+1.2	97.3
	4	16.3	113.1	54	17.3	110.5	+1.0	97.7	16.6	113.1	72	17.5	110.6	+0.9	97.8
	5	16.8	112.9	108	18.0	110.3	+1.2	97.7	16.2	113.6	90	17.3	111.6	+1.1	98.2
	6	16.6	113.8	72	17.7	110.4	+1.1	97.0	16.7	113.6	108	18.2	109.7	+1.5	96.6
	7	16.7	113.0	54	17.4	111.2	+0.7	98.4	16.4	113.6	90	17.6	110.7	+1.2	97.5
	8	16.3	114.5	90	17.9	110.8	+1.6	96.9	16.2	114.0	90	17.2	111.4	+1.0	97.7
	9	15.3	116.2	89	16.7	112.4	+1.4	96.7	15.6	115.3	90	16.8	112.6	+1.2	97.7
	10	15.7	115.4	54	16.9	112.4	+1.2	97.4	15.4	116.3	72	16.0	113.5	+0.6	97.6
6	1	14.3	117.3	69	14.9	115.5	+0.6	98.5	14.8	117.1	72	15.0	113.7	+0.2	97.1
	2	14.3	117.8	75	14.8	115.4	+0.5	98.0	14.9	116.8	88	14.5	114.1	−0.4	97.7
	3	14.5	118.0	56	15.0	115.4	+0.5	97.8	14.4	117.7	84	14.6	115.4	+0.2	97.7
	4	13.9	118.2	96	14.8	115.5	+0.9	97.7	14.5	116.3	80	15.0	114.7	+0.5	98.6
	5	14.2	117.4	48	14.7	115.9	+0.5	98.7	14.0	117.8	80	14.5	115.8	+0.5	98.3
	6	14.3	117.8	54	15.1	115.0	+0.8	97.6	14.1	117.6	88	14.7	115.3	+0.6	98.0
	7	14.9	116.5	65	15.7	114.0	+0.8	97.9	14.9	116.8	119	15.4	114.1	+0.5	97.7
	8	15.0	116.5	54	15.8	113.9	+0.8	97.8	15.1	116.3	72	15.6	114.6	+0.5	98.5
	9	15.4	116.0	90	16.3	113.4	+0.9	97.8	15.3	115.5	64	16.1	113.3	+0.8	97.8
	10	15.4	115.7	90	15.9	113.8	+0.5	98.4	15.3	116.1	90	16.6	113.3	+1.3	97.6
1[3]		15.7	115.1	319	16.7	111.7	+1.1	97.0	15.7	115.1	287	16.8	111.8	+1.1	97.2
2[3]		16.2	113.9	590	17.3	110.7	+1.1	97.3	16.3	113.8	521	17.4	110.7	+1.1	97.2
3[3]		14.8	116.7	740	15.3	114.5	+0.6	98.1	14.9	116.6	881	15.6	114.3	+0.6	98.0
4[3]		15.7	115.3	766	16.4	112.6	+0.8	97.7	15.7	114.9	879	16.6	112.4	+0.9	97.8
5[3]		16.3	114.1	755	17.3	111.1	+1.1	97.5	16.3	113.6	872	17.2	110.8	+0.9	97.5
6[3]		14.6	117.1	697	15.3	114.8	+0.7	98.0	14.7	116.8	837	15.2	114.3	+0.5	97.9
All[3]				3867			+0.8	97.7			4277			+0.8	97.7

[1] Density determined by a one-point method, see Appendix B.
[2] Density determined from tube sample (see text).
[3] Loop average.

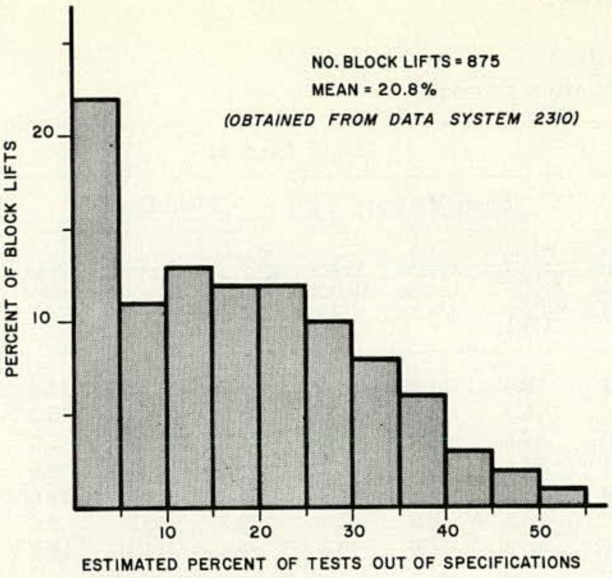


Figure 47. Embankment compaction, rigid and flexible tangents.

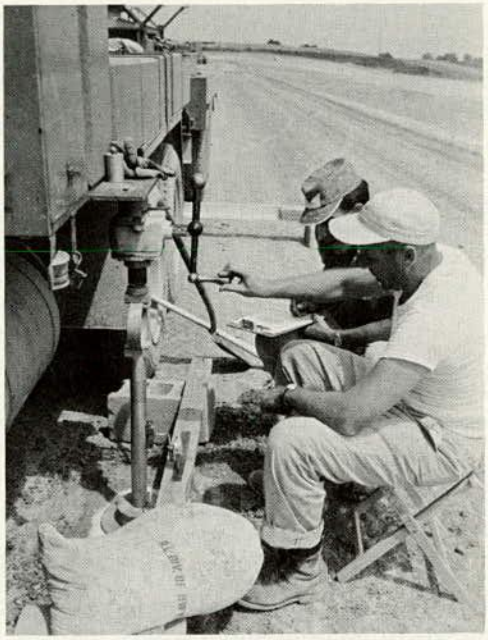


Figure 48. In-place CBR test on completed embankment.

density tests were within specification limits (see Table 12).

Appendix A describes the statistical analysis procedure for estimating percentage-out for the individual block-lifts. This procedure is based on recognized methods of quality control.

Maximum densities were determined by a procedure that used one molded specimen, the Proctor penetration needle, and a family of moisture-density-Proctor needle reading curves prepared from standard tests on the A-6 soil. Wet densities and corrections for moisture contents of the specimens corresponding to optimum conditions were obtained from the curves. This procedure is discussed in Appendix B.

Field densities were obtained from driven tube samples. The tubes were made from 3-in. diameter thin-walled tubing ($\frac{3}{32}$ in. thick), cut into $3\frac{7}{8}$-in. lengths, and beveled on one end. They were connected to a drop hammer by a pin for ease of removal. All tubes were numbered, weighed and the volumes determined (approximately $\frac{1}{70}$ cu ft).

Four 5-man field crews obtained the samples for field density determinations from random locations on the embankment. The tubes containing the samples were wrapped in aluminum foil and brought to the laboratory where a 7-man crew trimmed the ends of the samples, weighed them and obtained moisture samples. The moisture samples went through the drying oven, and the data were forwarded to the calculating unit for computation.

Summaries of construction control data obtained from tests taken at the completion of each block-lift of embankment are given in Tables 11 and 12 and in Figures 43 through 47. Each block-lift was tested and accepted individually. However, the data are presented as averages for blocks, tangents and loops, and as curves showing over-all distributions of test results.

All field compaction data are summarized in Table 11. Both optimum conditions and field conditions are included. For each construction block, the values shown for optimum conditions represent the average of one-half the number of tests given for the corresponding field data.

Table 12 and Fgures 43 through 46 illustrate the uniformity attained in embankment construction. Table 12 gives the percentages of individual tests that were within, above, and below the specification limits for both field moisture content and field density. Figures 43 and 44 depict the frequency distribution curves for the percent compaction. The frequency distribution curves for field moisture contents, expressed as percentage points above or below optimum, are shown in Figures 45 and 46. The curve shown in each of the figures is the mean curve for all the loops, and the shaded area represents the limits within which the distribution curves for each of the loops are contained.

Upon the recommendation of the Soils Panel an attempt was made to control the moisture content not only within the specification limits but on the west side of optimum moisture in order to increase the likelihood of obtaining uniform support strength in the embankment

TABLE 12

EMBANKMENT CONSTRUCTION CONTROL

Loop	Flexible Tangent						Rigid Tangent					
	Field Moisture			Field Density			Field Moisture			Field Density		
	Tests Within Specs.[1] (%)	Tests Above (%)	Tests Below (%)	Tests Within Specs.[2] (%)	Tests Above (%)	Tests Below (%)	Tests Within Specs.[1] (%)	Tests Above (%)	Tests Below (%)	Tests Within Specs.[2] (%)	Tests Above (%)	Tests Below (%)
1	81.2	18.8	—	81.8	6.3	11.9	78.0	21.6	0.4	81.1	5.5	13.4
2	77.1	21.7	1.2	80.1	8.2	11.7	77.9	21.5	0.6	78.6	8.4	13.0
3	91.5	7.4	1.1	76.9	15.1	8.0	88.6	10.0	1.4	79.8	15.0	5.2
4	87.5	12.3	0.2	82.3	10.0	7.7	86.1	12.9	1.0	83.6	10.8	5.6
5	80.8	18.9	0.3	81.5	8.6	9.9	80.1	18.3	1.6	79.1	9.9	11.0
6	87.4	12.1	0.6	80.3	13.5	6.2	89.5	8.2	2.3	79.2	12.2	8.6
All	82.9	16.8	0.3	80.4	10.7	8.9	83.4	16.0	0.6	80.2	11.1	8.7

[1] Specification limits, optimum ± 2 percentage points.
[2] Specification limits, 95 to 100 percent of maximum dry density.

material. The characteristics of the A-6 soils were such that the effect of changes in moisture content on soil strength was less when the moisture content was above optimum. Controlling the moisture content on the wet side of optimum also was advantageous in obtaining uniform density in that it assisted in avoiding overcompaction. As shown in Figures 45 and 46, the mean moisture content was 0.8 of a percentage point above optimum, and only about 20 percent of the tests indicated moisture contents less than optimum.

Figure 47 shows the distribution of the computed estimated percentage of tests out of the

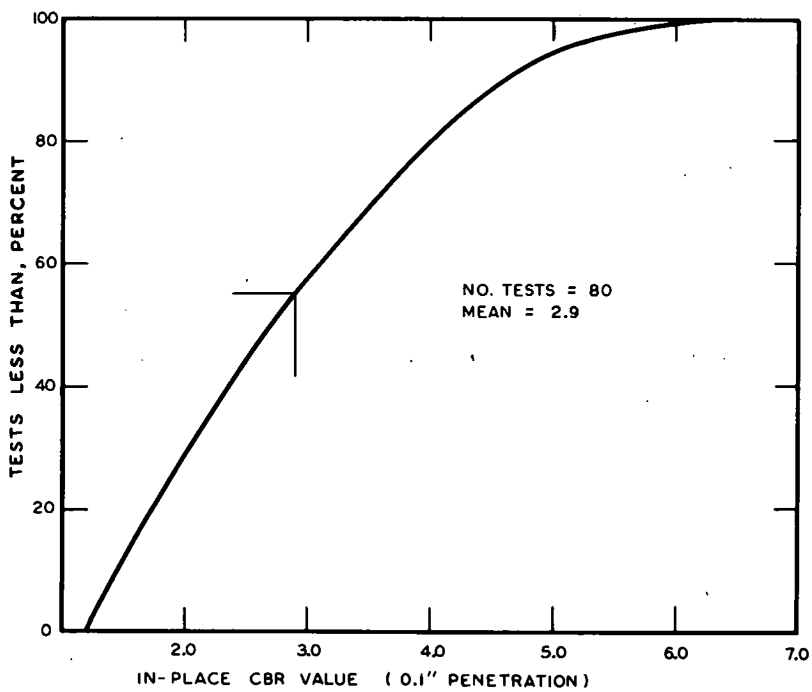


Figure 49. In-place CBR determination of embankment on flexible tangents at time of paving.

specification limits for the individual block-lifts. By comparing this curve with those in Figures 43 and 44 and with the information in Table 12, it can be seen that the mean estimated 20.8 percent of tests out of specifications compares with the actual 19.6 percent of individual tests out of specifications for the flexible tangents and 19.8 for the rigid. The 3 percent of block-lifts shown by the curve as being above the permissible estimated 45 percent of tests out of specification represents part of the early construction used to establish the maximum.

2.4 SUPPLEMENTARY TESTS

Certain supplementary tests were made to provide additional information on the A-6 soil and on the as-constructed characteristics of the completed embankments.

Density and in-place CBR tests were made on the top of the embankment at the time of paving. On rigid tangents, two structural sections were tested for each thickness of subbase in each of the five traffic loops, and three were tested for the 6-in. thickness in Loop 1. Three structural sections were tested for each

TABLE 13

SUMMARY OF CBR TESTS ON COMPLETED EMBANKMENT AT TIME OF PAVING [1]
(CBR at 0.1-in. penetration [2])
(Obtained from Data Systems 2110 and 2210)

Nominal Subbase Thickness (in.)	Item [3]	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6	Mean
				(*a*) FLEXIBLE TANGENT				
4	CBR		3.0	1.9	2.4	1.9		2.3
	Deg. S.		88	87	82	85		86
	N		3	3	3	3		12
8	CBR	3.0		3.6	2.2	1.4	2.4	2.5
	Deg. S.	88		86	85	87	85	86
	N	2		3	3	3	3	14
12	CBR				5.3	2.5	3.7	3.8
	Deg. S.				82	83	88	84
	N				3	3	3	9
18	CBR	3.9					3.6	3.7
	Deg. S.	82					86	84
	N	2					3	5
Mean	CBR	3.5	3.0	2.8	3.3	1.9	3.2	2.9
	Deg. S.	85	88	87	83	85	86	85
	N	4	3	6	9	9	9	40
				(*b*) RIGID TANGENT				
3	CBR		1.2	2.2	1.1	0.9	3.0	1.7
	Deg. S.		88	86	85	86	89	87
	N		2	2	2	2	2	10
6	CBR	1.3	1.4	3.3	1.1	1.3	2.6	1.8
	Deg. S.	87	88	84	88	88	91	88
	N	3	2	2	2	2	2	13
9	CBR			1.7	1.8	0.7	1.9	1.5
	Deg. S.			86	89	88	88	88
	N			2	2	2	2	8
Mean	CBR	1.3	1.3	2.4	1.3	1.0	2.5	1.7
	Deg. S.	87	88	85	87	87	89	87
	N	3	4	6	6	6	6	31

[1] Taken immediately prior to subgrading the subbase: June 27 to Sept. 24, 1958, on flexible tangents; May 16 to July 22, 1958, on rigid tangents.
[2] "Suggested Method of Test for California Bearing Rating Soils", p. 386–397, ASTM Publication, "Procedures for Testing Soils" (July 1950).
[3] N is number of structural sections tested; Deg. S. is degree of saturation of soil.

TABLE 14

SUMMARY OF FIELD DENSITIES ON COMPLETED EMBANKMENT AT TIME OF PAVING [1]
(Obtained from Data Systems 2110 and 2210)

Nominal Thickness Subbase (in.)	Item [2]	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6	Mean
				(a) FLEXIBLE TANGENT				
4	M.C.		+1.3	+1.4	+0.3	+0.8		+1.0
	Den.		98.2	98.2	97.8	97.6		98.0
	N	3	3	3	3	3		12
8	M.C.	+1.4		0.0	+0.3	+1.9	+0.4	+1.0
	Den.	98.8		99.2	99.2	96.5	98.9	98.5
	N	2		3	3	3	3	14
12	M.C.				−0.3	+2.4	−0.2	+0.6
	Den.				99.8	94.9	101.3	98.7
	N				3	3	3	9
16	M.C.	+0.8					+0.4	+0.6
	Den.	98.8					99.5	99.2
	N	2					3	5
Mean	M.C.	+1.1	+1.3	+0.7	+0.4	+1.7	+0.2	+0.8
	Den	98.8	98.2	98.7	98.9	96.3	99.9	98.5
	N	4	3	6	9	9	9	40
				(b) RIGID TANGENT				
3	M.C.		+2.0	+1.4	+1.9	+1.8	+0.9	+1.6
	Den.		97.0	97.6	96.0	94.7	98.8	96.8
	N		2	2	2	2	2	10
6	M.C.	+1.6	+0.9	+2.0	+1.4	+2.1	+0.9	+1.5
	Den.	98.2	99.1	95.4	98.5	96.4	100.2	98.0
	N	3	2	2	2	2	2	13
9	M.C.			+0.9	+0.9	+2.1	+0.5	+1.1
	Den.			98.1	100.3	96.3	99.4	98.5
	N			2	2	2	2	8
Mean	M.C.	+1.6	+1.5	+1.4	+1.4	+2.0	+0.8	+1.3
	Den.	98.2	98.1	97.0	98.3	95.8	99.5	97.8
	N	3	4	6	6	6	6	31

[1] Taken immediately prior to subgrading the subbase.
[2] M.C. is moisture content, expressed as plus or minus percentage points from optimum; density is expressed as percent of maximum dry density; N is number of structural sections tested.

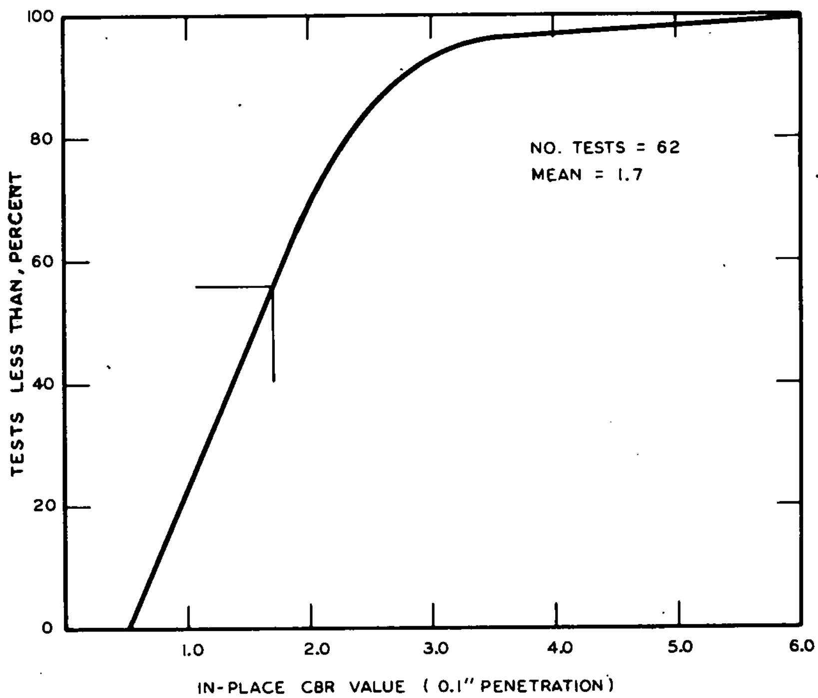


Figure 50. In-place CBR determination of embankment on rigid tangents at time of paving.

TABLE 15

MOISTURE CONTENT[1] OF EMBANKMENT SOIL AFTER PAVING

(Obtained from Data Systems 2111 and 2211)

	Rigid Tangent			Flexible Tangent		
Loop	No. Tests	Mean	Std. Dev.	No. Tests	Mean	Std. Dev.
1	10	+0.1	0.65	8	+0.7	0.51
2	24	+1.5	1.53	6	0.0	0.67
3	37	+0.3	1.35	12	−1.3	1.30
4	42	+0.1	1.08	28	−1.4	0.91
5	37	+0.8	0.88	9	−0.3	0.79
6	38	+0.3	1.19	6	−1.4	1.40
All	188	+0.5	1.24	69	−0.9	1.22

[1] Expressed as plus or minus percentage points from optimum.

thickness of subbase (except the 0-in. thickness) on the flexible tangent of each of the five traffic loops, and two were tested for the 8-in. and 16-in. thicknesses of subbase on the flexible tangent of Loop 1. The tests were taken in the center of the embankment near the end of each structural section and confined to a 2- by 3-ft area.

Two CBR tests were made at each location, using a 10-in. diameter surcharge plate and a total surcharge weight of 30 lbs. The reported value for each structural section was the average of the two tests. Two field density determinations were made for each of the two CBR tests taken at a given location. The reported density of the embankment for each structural section tested was the average of the four tests. The corresponding degree of saturation of the embankment soil also was determined. The results of the CBR tests and degree of saturation are given in Table 13. The distributions of the individual CBR values for all loops are shown in Figures 49 and 50. Table 14 is a summary of the results of the field density tests. The moisture content is expressed as percentage points above or below optimum, and field density as the percent of maximum dry density. The optimum moistures and maximum dry densities were obtained during construction. They represent the averages of the individual tests on the top lifts of embankment for the construction blocks containing the respective structural sections tested.

The moisture content of the top 2 in. of embankment soil also was determined when the completed pavements were cored. On flexible pavement tangents, tests were made only on structural sections that had a base course thickness of 3 in. or less. In general, one test was made for each structural section on rigid pavement tangents. The results are summarized in Table 15. The moisture contents are given as percentage points above or below the optimum moisture contents of top lifts determined during embankment construction.

During construction, a cooperative testing program was established. Samples of the Road Test materials were sent to more than 60 agencies. Appendix C discusses the program and gives summaries of tests on the embankment soil for material characteristics and design properties.

The Bureau of Public Roads conducted certain tests on the yellow-brown A-6 soil to provide information that was not being obtained at the project. Appendix D gives the results of tests for X-ray diffraction, classification, compaction, direct shear, triaxial compression, CBR and Hveem resistance value.

Chapter 3

Subbase—Flexible and Rigid Pavements

The same material was used as the subbase for both flexible and rigid pavements. The major portion of the material was placed on the embankment as a protective covering. This "sand-gravel mulch" was later processed and compacted as subbase.

This chapter describes the material, including the specification requirements and production, and discusses the procedures and controls for the construction of the sand-gravel mulch and of the subbase on both flexible and rigid tangents. It also includes summaries of material and construction control data.

3.1 MATERIALS AND MATERIALS CONTROL

The specifications for the subbase material included gradation limits, a plasticity requirement, and a California Bearing Ratio (CBR) requirement.

The gradation limits were considered only as maximum and minimum values. The contractor was required to submit a gradation formula, stating single percentages for the amounts of material passing each sieve. The specifications included plus and minus tolerances for variations from the approved formula.

Table 16 gives the specification requirements and approved gradation formula as revised after the decision in June 1957 to stabilize the original material with a friable, fine-grained soil before any material had been placed on the tangents (see Section 1.2.3).

This stabilization made necessary a revision in the percentage of material passing the No. 200 sieve and in the plasticity and CBR requirements. The original specifications required the percentage passing the No. 200 sieve to be 0 to 5, the fraction passing the No. 40 sieve to be non-plastic, and the CBR value to be not greater than 40. The original approved gradation formula required 72 percent passing the No. 4 sieve, 52 percent through the No. 10, 24.5 percent through the No. 40, and 3 percent passing the No. 200 sieve.

The subbase material was produced in accordance with the original specifications and stabilized with a friable fine-grained soil. The two materials were proportioned by weight and thoroughly mixed in a paving mixer. Samples of stabilized subbase material were obtained off the conveyor belt at the discharge end of the mixer. Results of gradation tests on these samples are summarized in Table 17 and shown graphically in Figure 51.

It will be noted that the gradation of the fraction of the material finer than the No. 4 sieve was on the low side of the formula. This was purposely done to compensate for the fines created by the additional handling necessary to deposit and spread the material on the grade. Comparing these data with those shown in Section 3.3 illustrates this point.

Tests by the central laboratory of the Illinois Division of Highways gave CBR values for 0.1-in. penetration ranging from 28 to 51, and indicated that the fraction passing the No. 40 sieve was non-plastic.

The specifications required that all subbase material be obtained from a single source.

TABLE 16

SPECIFICATIONS FOR SUBBASE MATERIAL

(a) GRADATION REQUIREMENTS[1]

Sieve	Spec. Limits (% passing)	Gradation Formula (% passing)	Allowable Tolerances (%)
1½ in.	100	100	—
1 in.	95–100	100	Spec. limits
¾ in.	90–100	95	Spec. limits
½ in.	80–100	90	±5
No. 4	55–100	73	±5
No. 10	40–80	54	±5
No. 40	10–30	27	±3
No. 200	5–9	7	±2

(b) OTHER REQUIREMENTS

Characteristic	Requirement	Test Procedure
PI	0–6	AASHO Des.: T91–54
CBR	Not greater than 60	—[2]

[1] Test method: AASHO Designation: T27–46 (square openings).

[2] "Suggested Method of Test for Bearing Ratio and Expansion of Soils," pp. 109–116, "Procedures for Testing Soils," ASTM (Sept. 1944).

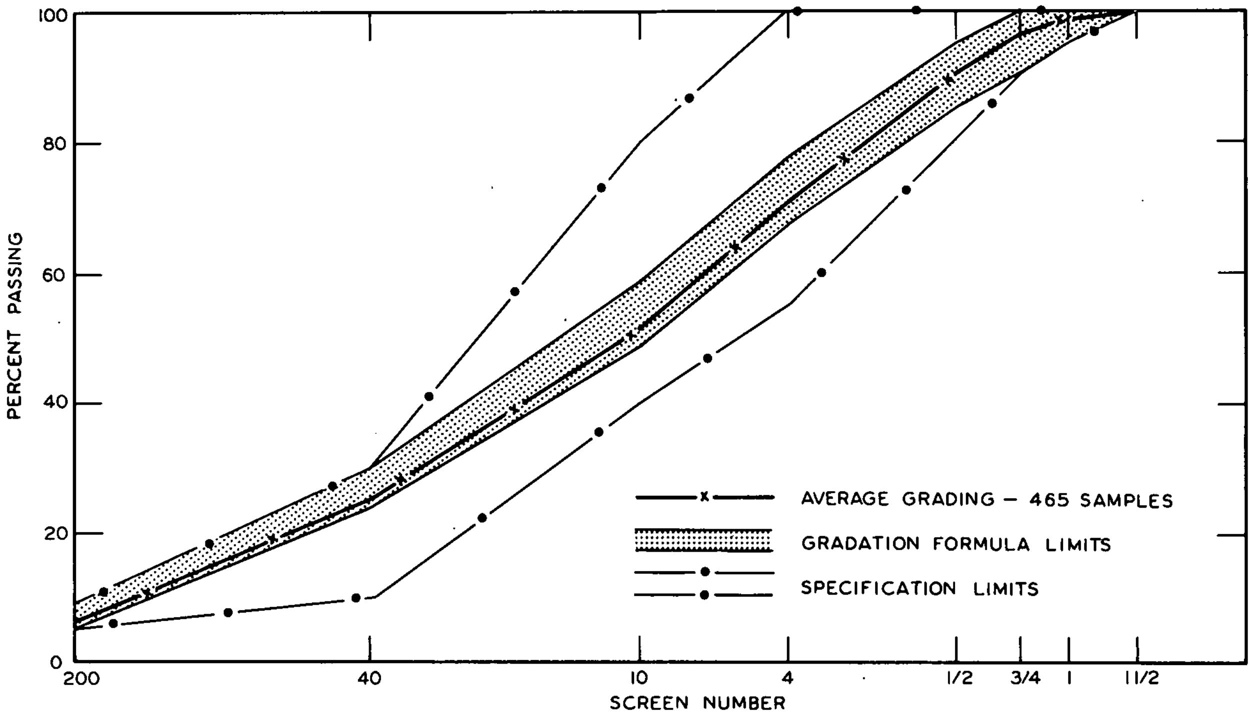


Figure 51. Gradation of subbase material.

Therefore, the entire quantity was furnished under the grading contract. The material not needed for the sand-gravel mulch construction was stockpiled at the production plant.

Material from a deposit near the west end of the project was produced to the original specifications by a washing and screening plant. Silica sand from the same deposit was blended with the sand-gravel mixture during production to increase the minus No. 40 sieve fraction.

The washing and screening plant was set up in August 1956. Production was under way in the first half of September, but difficulties were encountered in removing the clay. The problem was solved by increasing the washing capacity of the plant. However, material meeting the specifications was not obtained until late October.

The sand-gravel material was stabilized with soils from the top 18 in. of overburden along the south end of the deposit. This soil was a friable, fine-grained brown silt loam. Its plasticity index was generally less than 12, and 96 percent passed the No. 200 sieve.

The full 18 in. of soil were removed from an exposed face, and were pulverized, blended and dried by a rotary speed mixer. The processed material was stockpiled for use in the mixing plant.

The mixing plant consisted of a batching bin and a dual-drum 34-E paving mixer. A chute was added to the bin to transfer the

TABLE 17

Summary of Gradation Tests[1] on Stabilized Subbase Material (Summary of 465 Tests)
(Obtained from Data System 2331)

Sieve	Gradation Formula and Tolerances	Mean Percent of Material Passing	Standard Deviation	Percent of Tests: Within Tolerances	Percent of Tests: Above Tolerances	Percent of Tests: Below Tolerances
1½ in.	100	100	—	100	—	—
1 in.	95–100	100	0.90	99.8	0	0.2
¾ in.	90–100	96	1.96	99.4	0	0.6
½ in.	90 ± 5	90	2.39	97.7	0.4	1.9
No. 4	73 ± 5	71	3.13	87.8	1.7	10.5
No. 10	54 ± 5	52	2.91	85.5	0	14.5
No. 40	27 ± 3	25	2.60	83.9	0	16.1
No. 200	7 ± 2	6.5	0.76	97.4	0.2	2.4

[1] AASHO Designation: T27–46.

Figure 52. Washing and screening plant for subbase material.

batch to the skip of the mixer. The boom and bucket of the mixer were replaced by a conveyor belt to transfer the stabilized material to a stockpile. The soil and the sand-gravel mixture were proportioned by weight in 5000-lb batches to produce a material that would meet the revised specifications.

3.2 CONSTRUCTION PROCEDURES

3.2.1 Construction as Sand-Gravel Mulch

A layer of subbase material was placed over the subgraded embankment as the sand-gravel mulch to assist in bringing the moisture condition of the subgrade to a level near that found under pavements which have been in place for several years. The material was placed with a track-type self-propelled spreader which operated on the loose material being spread. The specifications prohibited all but the spreading and compacting equipment from operating within the center 24-ft portion of the roadbed. This required the development and modification of equipment to place the material from the shoulders.

The sand-gravel mulch was placed under the grading contract. Plans called for a uniform 6-in. thickness on all structural sections. Under the paving contract, the sand-gravel material would be brought to the subbase design thickness by adding or removing material as required. However, in the interest of economy on the paving contract, the mulch was placed at the subbase design thickness on all sections where that thickness was 3, 4, 6, 8 or 9 in. Three inches of material was placed on all sections where the pavement design did not include a subbase. On flexible pavement sections where subbase design thickness was 12 or 16 in., an 8-in. compacted thickness was placed. Only 3 in. of material was placed on all outside shoulders since the paving specifications required that this material be removed if equipment could not operate on it.

Figure 53. Mixing plant for stabilizing subbase material.

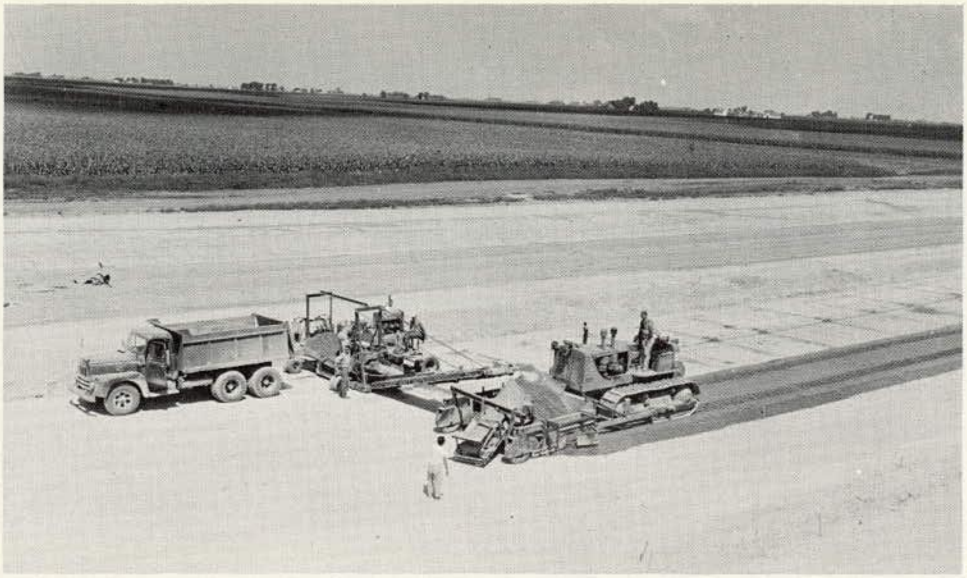


Figure 54. Placing sand-gravel mulch on embankment from outside shoulder.

All turnarounds were covered with the original sand-gravel material which was stabilized in-place with the fine-grained soil.

Precise controls on moisture content and density of the mulch were not required. The choice of compaction equipment and the method of compaction was left to the option of the contractor.

The material was placed on the center portion of the embankment (a 13-ft spread down the inside and the outside traffic lanes) with a conventional spreader mounted on the front of a track-type tractor. The spreader was modified with a plate across the front to confine the material and a short belt to convey the material into the spreader box. A pavement widening machine, modified to include an outrigger conveyor belt, transferred the material from trucks on the outside shoulders to the discharged off the outrigger belt directly into the spreader box. On the outside lanes, it was necessary to wipe the material off the outrigger belt into the hopper of the short belt on the front of the spreader. Two identical outfits were used.

The material was placed on the shoulders by conventional methods, with trucks dumping directly into the spreader box. Heavy timber planks were placed across the center portions of the embankment at certain transition areas for truck crossings onto the inside shoulder.

Spreading on the traffic lanes always preceded that on the shoulders. Daily operations were terminated in the transitions between construction blocks or between structural sections.

The modified pavement widening machine could convey material only in one direction. Thus, it was necessary to start at the west end and progress eastward on rigid tangents and start at the east end and progress westward on the flexible tangents.

Considerable difficulty was encountered in shaping and compacting the sand-gravel material. The loose material absorbed water from rains, and penetration of moisture into the embankment soil progressed rapidly. One moderate rainfall was sufficient to cause the embankment to be rutted by standard blade graders and pneumatic-tired rollers.

To achieve final shaping, blade graders were equipped with over-sized balloon tires at 12-psi pressure. Sufficient crown was provided during shaping to prevent the formation of low areas which would hold water.

Vibratory equipment was used to compact the mulch. Normally, one pass was sufficient. Water to assist in the compaction was applied to the material with hand-operated pressure spray bars and spray nozzles.

Table 18 is a record of the placing and compacting of the sand-gravel mulch on the tangents of the six loops.

3.2.2 Construction as Subbase for Flexible Pavements

Subbase construction involved processing and compacting the material previously placed on the embankments as the sand-gravel mulch and placing and compacting new material. The construction specifications were aimed at producing a uniform subbase at the required thickness throughout all tests sections.

It was specified that the subbase be compacted at the optimum working moisture content (plus or minus one percentage point) and to between 100 and 105 percent of standard AASHO density. The optimum working mois-

TABLE 18

RECORD OF CONSTRUCTION OF SAND-GRAVEL MULCH

Const. Block No.[1]	Date of Placing[2]						Date of Compacting					
	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6
F–1	8–26–57	8–22–57	9–23–57	10– 2–57	9–11–57	9–10–57	11– 4–57	10–30–57	10–31–57	10–19–57	11– 1–57	10–18–57
F–2	8–23–57	8–21–57	9–23–57	10– 2–57	9–11–57	9– 9–57	11– 4–57	10–30–57	10–31–57	10–19–57	11– 1–57	10–17–57
F–3	8–23–57	8–21–57	9–19–57	9–30–57	9–11–57	9– 9–57	11– 4–57	10–30–57	10–31–57	10–21–57	11– 1–57	10–17–57
F–4		8–21–57	9–18–57	9–30–57	9–10–57	9– 6–57		10–30–57	10–31–57	10–21–57	11– 1–57	10–17–57
F–5		8–20–57	9–18–57	9–27–57	9– 9–57	9– 6–57		10–30–57	10–31–57	10–21–57	11– 2–57	10–17–57
F–6		8–20–57	9–17–57	9–27–57	9– 7–57	9– 5–57		10–30–57	10–31–57	10–22–57	11– 2–57	10–15–57
F–7		8–19–57	9–16–57	9–26–57	9– 6–57	9– 5–57		10–30–57	10–31–57	10–30–57	11– 2–57	10–15–57
F–8			9–16–57	9–26–57	9– 5–57	9– 4–57			10–31–57	10–30–57	11– 2–57	10–14–57
F–9			9–14–57	9–25–57	9– 4–57	9– 4–57			10–31–57	10–30–57	11– 2–57	10–12–57
F–10			9–11–57	9–25–57	9– 3–57	9– 3–57			10–22–57	10–30–57	11– 2–57	10–12–57
R–1	8–27–57	8–15–57	9–24–57	9–14–57	8– 8–57	8–22–57	11– 5–57	10–29–57	10–21–57	11– 1–57	11– 5–57	10–28–57
R–2	8–27–57	8–16–57	9–24–57	9–16–57	8–12–57	8–23–57	11– 5–57	10–30–57	10–21–57	11– 1–57	11– 5–57	10–28–57
R–3	9– 3–57	8–16–57	9–24–57	9–16–57	8–12–57	8–23–57	11– 5–57	10–30–57	10–21–57	11– 1–57	11– 5–57	10–18–57
R–4		8–16–57	9–25–57	9–17–57	8–12–57	8–26–57		10–30–57	10–22–57	11– 1–57	11– 5–57	10–18–57
R–5		8–17–57	9–26–57	9–18–57	8–13–57	8–27–57		10–30–57	10–22–57	11– 1–57	11– 5–57	10–18–57
R–6		8–19–57	9–26–57	9–10–57	8–15–57	8–27–57		10–30–57	10–22–57	10–31–57	11– 4–57	10–28–57
R–7		8–19–57	9–28–57	9–19–57	8–15–57	8–30–57		10–30–57	10–19–57	10–31–57	11– 4–57	10–28–57
R–8			10– 2–57	9–23–57	8–19–57	8–30–57			10–19–57	10–31–57	11– 4–57	10–28–57
R–9			10– 3–57	9–23–57	8–20–57	8–31–57			10–19–57	10–31–57	11– 4–57	10–28–57
R–10				9–24–57	8–20–57	8–31–57				10–31–57	11– 4–57	10–28–57

[1] F = flexible, R = rigid.
[2] Center 26 ft only.

ture content, set by the engineer, was defined as that moisture content at which the material could be most readily and satisfactorily spread and compacted. (This moisture content, in the case of granular materials, is generally less than the optimum moisture content determined by the standard AASHO procedure.)

As specified, compaction was obtained with vibratory equipment. The compactors each had six vibrating shoes 26 in. wide, and compacted a width slightly in excess of 13 ft. The shoes, individually driven by electric motors, had 8-lb eccentric weights and operated at 4,200 vibrations per minute when not under load. The compactors had pneumatic-tired dual wheels on both axles.

The specifications prohibited all but the spreading and compacting equipment from operating within the center 24-ft portion of the roadbed, and permitted equipment to cross the roadbed only in designated transition areas and upon approved matting.

The center 24 ft was required to be subgraded to within 1/8 in. of the established grade with a subgrading machine operating on steel forms. This portion of the subbase was constructed above the established grade at all locations, since filling of low areas during subgrading operations was not permitted.

Pilot construction of subbase was started in the fall of 1957, with construction on the test tangents scheduled for the following spring. The major concern, particularly in thin subbase sections, was compacting the material to the required density without rutting the underlying earth subgrade.

Before starting subbase construction on the pilot sections, the top 6 in. of the earth embankment was scarified, adjusted for moisture content and recompacted to a density corresponding to that in the test loops. However, the earth subgrade on the pilot sections was more stable than that on the test tangents and it was evident that the determinations from the pilot work would need modification for construction on test tangents.

The pilot work pertaining to the flexible pavements was done on the north tangent between loops A and D. It included the construction of subbase to compacted thicknesses of 4 in., 8 in., 4 in. on 8 in. for a 12-in. thickness, and 8 in. on 8 in. for a 16-in. thickness.

The optimum working moisture content was established at 7 percent. At the 6 percent lower limit the material was almost too dry to obtain the required density. At the 8 percent upper limit it was slightly wet for compacting thicker lifts.

The pilot work indicated that two passes of the vibrator were sufficient for a 4-in. thickness, three passes for 8- and 12-in. thicknesses, and four passes on the top 8-in. lift of the 16-in. thickness. However, it was necessary to increase the number of passes on the test tangents. Generally, three to five passes were required for a 4-in. compacted thickness while five to eight passes were needed for greater thicknesses.

The pilot studies indicated that maximum compaction could not be obtained unless a water sheen developed on the surface of the material as the vibrator passed. Without this sheen the material tended to compress under the vibrating shoes and rebound as the shoes passed, leaving the subbase in an unstable condition. In the compaction procedure developed, the vibrator made one pass over the loose material, water was applied lightly to the surface, and the vibrating was continued. Vibration alone did not cause the water sheen to form, especially on thick layers, but the application of water apparently served as a primer to start the process. On test tangents, it was often necessary to prime with a light application of water more than once because more passes of the compactor were required.

As required by the specifications, pilot tests were made with steel-wheel and pneumatic-tired rollers for final compaction of the top surface of the subbase. However, there was a tendency for both rollers to displace the material and reduce the density. Furthermore, the surface of the subbase was left in a smooth, dense condition by the vibratory compactors, and additional compaction of top surfaces was not needed.

In May 1958, subbase construction began on the test tangents. Construction of 4- and 8-in. thicknesses of subbase required only the processing and compacting of the in-place material. The 12- and 16-in. thicknesses required processing and compacting the 8 in. of in-place material and placing and compacting additional material.

The construction was carried out in three separate stages. The first included the processing and compacting of the existing sand-gravel material on those sections having a total subbase thickness of 12 or 16 in. Additional material to bring these sections to the required thickness was then added and given the same compaction applied to the material when it served as the sand-gravel mulch. The second and third stages involved the processing and compacting of the in-place material on the remaining sections and the processing and compacting of the layer of additional material on the 12- and 16-in. thick sections. In the second stage, the processing and compacting were performed on those sections whose design included a base course, and the final stage involved only those sections having the asphaltic surfacing placed directly upon the subbase. Construction within a loop for each stage was started at the west end of the tangent and proceeded eastward.

The procedure described above was a deviation from the specifications in regard to construction with new material. The specifications required that the new material be compacted to the required density immediately upon completion of the spreading operation. The procedure used, however, provided for more uniform subbase construction in that all structural sections were constructed in the same manner without regard to whether or not additional material was needed. The procedure also permitted a continuous operation in spreading additional material.

A self-propelled track-type spreader was used to place the additional material. It was designed to operate on the loose material being spread, and was equipped with an oscillating screed adjustable for crown and depth of spreading. Material was deposited into the spreader from trucks operating on the outside

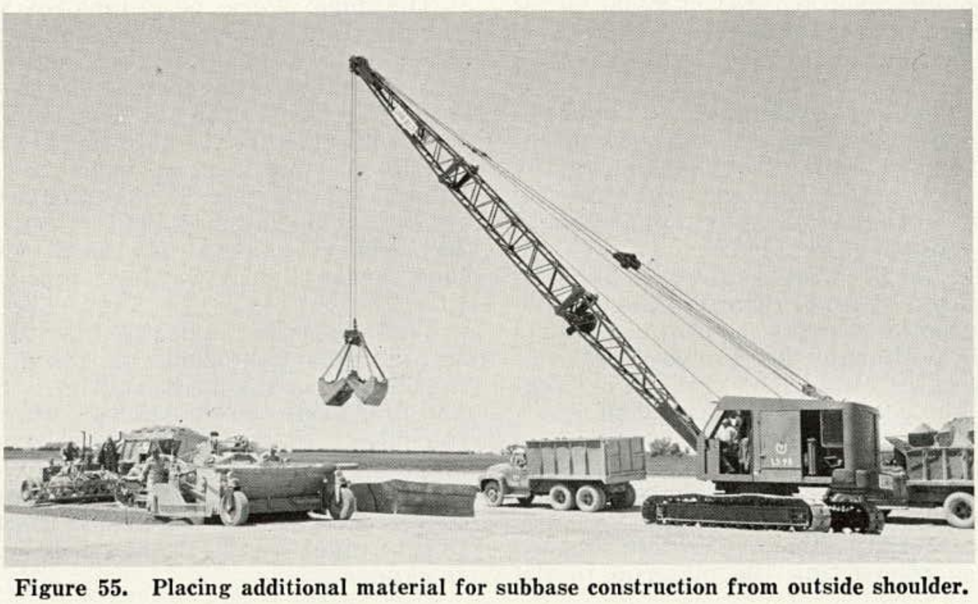


Figure 55. Placing additional material for subbase construction from outside shoulder.

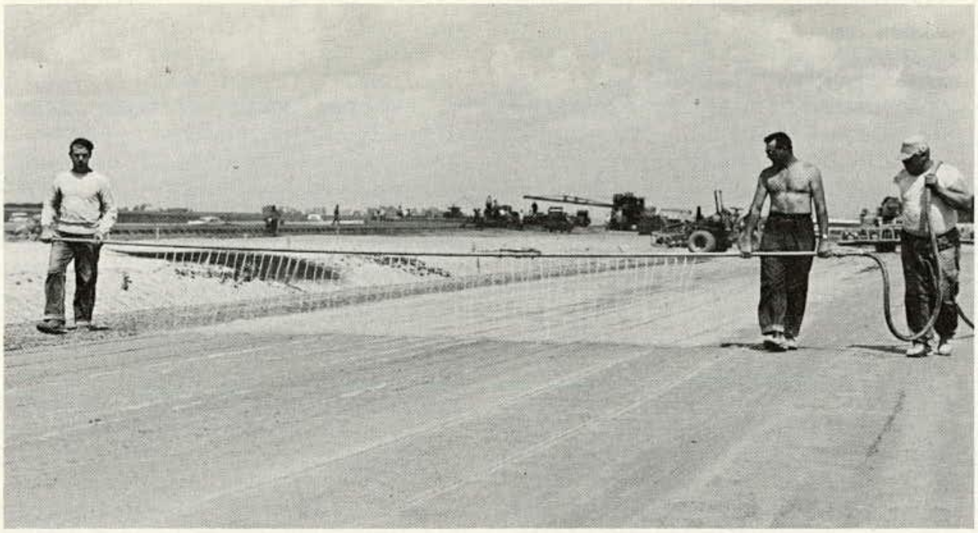


Figure 56. Adding water for compaction of subbase.

shoulders. The trucks dumped into a large metal box and a crane transferred the material to the spreader by clam bucket.

Prior to processing the in-place material, the surface was shaped to the proper cross-section and grade. Where necessary, material was bladed from the inside shoulder to the center to insure that after compaction the center 24 ft would contain sufficient material to permit subgrading in accordance with the specifications. Motor graders, equipped with low-pressure balloon tires, were used to shape the material.

Water was added to the material by hand-operated pressure spray bars and by outrigger pressure spray bars attached to trucks operating on the outside shoulders. Before adding the water, the material was scarified with the balloon-tired motor graders and with a farm-type spike tooth harrow pulled by a

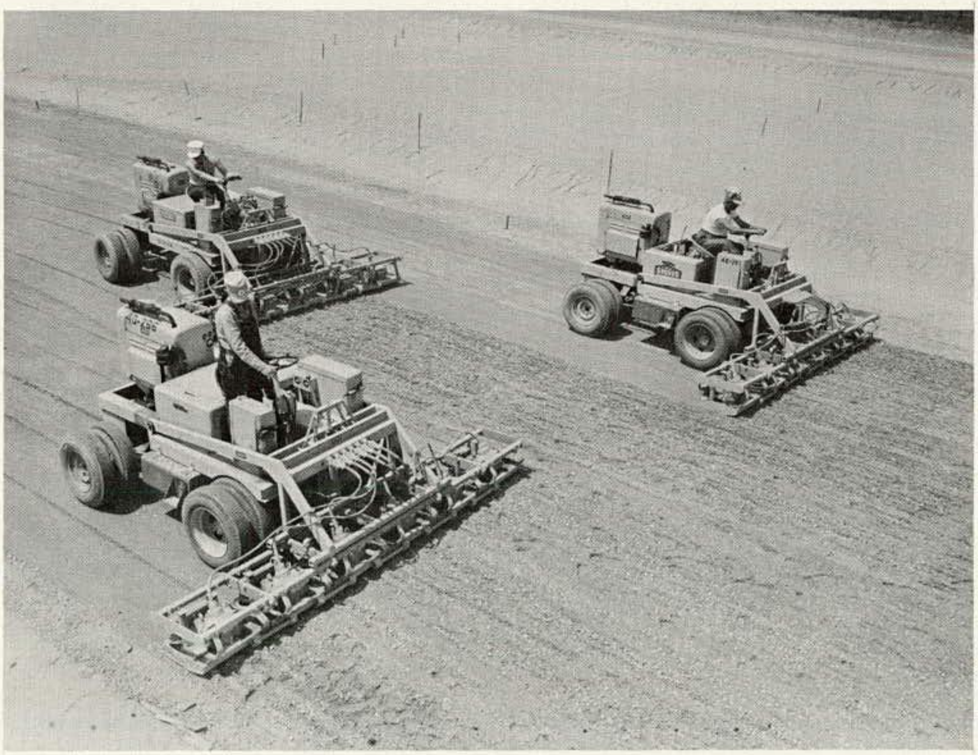


Figure 57. Compacting subbase.

small track-type tractor. Mixing of the water and subbase was accomplished with the harrow. Watering and mixing continued until the required moisture content was obtained.

It was necessary to add water to the material sufficiently ahead of compaction to permit absorption throughout the entire depth of the material. A minimum of one-half day was required. It was usually necessary to apply additional water to a section just before compaction to replace the moisture lost by evaporation.

The vibratory compactors were operated in teams of three to five. Compaction always progressed from west to east. The equipment could compact only when operated in a forward direction, and turning in the transitions could not be done without marring and rutting the subbase or damaging the earth subgrade. Therefore, at the end of the sections being compacted, the vibrator shoes were lifted and the machines were backed the length of the sections to start the next pass.

After a section was compacted and accepted, water was applied as needed to compensate for evaporation until the section was subgraded and covered with the next layer of material.

Subgrading of the subbase was performed immediately prior to constructing the next layer of pavement structure (base course or surfacing). A ½-ton tandem roller was used to compact the small amount of loose material on the surface. Tests conducted on the turnarounds and in a large transition on one of the tangents indicated that heavier rollers tended to crack the surface and displace the subbase material, thus destroying the compaction and risking damage to the underlying earth subgrade.

As stated in Chapter 2, the earth subgrade for sections which did not include a subbase was constructed approximately ¾ in. higher than the required elevation and covered with a 3-in. compacted thickness of subbase material to serve as the sand-gravel mulch. This material was removed just prior to resubgrading the embankment and constructing the next course (base or surfacing) over the earth subgrade. All material on the inside shoulder that could be removed without being contaminated

TABLE 19

RECORD OF FLEXIBLE SUBBASE CONSTRUCTION

Const. Block No.	Const. Sequence[1]	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6
F-1	1	5-26-58	...	...	...	5-14-58	4-21-58
	2	8-26-58	...	5-16-58	6-26-58	7-24-58	6-18-58
	3	8-26-58	...	...	7-30-58	...	...
F-2	1	...	...	...	5-19-58	5-15-58	4-21-58
	2	8-26-58	8-22-58	5-19-58	6-27-58	7-24-58	6-20-58
	3	9- 6-58	...	...	8- 4-58	...	...
F-3	1	5-24-58	...	...	5-19-58	...	4-21-58
	2	8-26-58	...	...	7- 9-58	7-25-58	6-20-58
	3	9- 6-58	9- 8-58	...	8- 4-58	...	...
F-4	1		...	...	...	...	4-21-58
	2		8-22-58	5-19-58	7-10-58	7-26-58	6-20-58
	3		9- 8-58	7- 1-58	8- 6-58	...	...
F-5	1		...	...	5-20-58	5-16-58	4-25-58
	2		...	5-19-58	7- 8-58	7-25-58	6-21-58
	3		9- 8-58	7- 2-58	8- 6-58	...	...
F-6	1		...	...	5-21-58	5-16-58	...
	2		8-23-58	5-19-58	7- 9-58	7-24-58	6-21-58
	3		9- 8-58	7- 2-58	...	...	...
F-7	1		...	...	5-23-58	...	...
	2		8-23-58	5-19-58	7-22-58	7-26-58	6-23-58
	3		...	7- 7-58	8- 7-58	...	...
F-8	1			...	5-23-58	5-20-58	4-28-58
	2			6- 6-58	7-23-58	7-28-58	6-27-58
	3			7- 3-58	8- 7-58	...	...
F-9	1			...	...	5-15-58	4-25-58
	2			6- 6-58	7-23-58	7-30-58	6-28-58
	3			7- 8-58	...	...	...
F-10	1			...	...	...	4-25-58
	2			...	7-23-58	8- 5-58	6-30-58
	3			...	...	...	...

[1] Construction sequence 1 is compaction of subbase for sections within block requiring additional material; Sequence 2 is compaction of subbase for sections within block whose design includes a base course; Sequence 3 is compaction of subbase for sections within block whose design does not include a base course.

TABLE 20

RECORD OF RIGID SUBBASE CONSTRUCTION

Construction Block No.	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6
R–1	6–23/24–58	6–20–58	5–7/8–58	5–29–58	5–29/6–5–58	5–14–58
R–2	6–24/58	6–20–58	5–10–58	6–2/7–58	6–4/5–58	5–15/16–58
R–3	6–24.52–58	6–20–58	5– 9–58	6–2/16–58	6–6–58	5–15–58
R–4		6–21–58	5–9/13–58	6–16/17–58	6–6/16–58	5–15/16–58
R–5		6–23–58	5–14/17–58	6–16/17–58	6–6/16–58	5–15/26–58
R–6		6–24–58	5–14/29–58	6–16/18–58	6–16/17–58	5–26/27–58
R–7		6–23–58	5–20/21–58	6–18–58	6–7/17–58	5–26/27–58
R–8			5–21/28–58	6–18–58	6–17–58	5–28–58
R–9			5–23/28–58	6–19–58	6–17–58	5–28/29–58
R–10				6–19/20–58	6–17–58	5–29–58

with the A-6 soil was bladed toward the center of the roadbed. Steel forms were set to the required line and grades, and the subgrading machine removed the subbase material and loaded it into trucks for delivery to a stockpile.

Table 19 shows the sequence of subbase construction for all flexible tangents. The dates apply to construction within the center portion of the roadbed.

3.2.3 Construction as Subbase for Rigid Pavements

With few exceptions, the procedures for constructing the subbase on rigid pavement tangents were identical to those for the flexible pavements. However, the quantity of sand-gravel material previously placed within the center portion of the roadbed was sufficient at all locations for the required thickness of subbase. Additional material was needed only on the shoulder portions. Prior to paving it was necessary only to process and compact the existing material on the center portion of the roadbed and on inside shoulders.

As with the flexible tangents, a greater number of passes of the vibratory compactors was required to obtain the specified density than was indicated by the pilot studies. Three to five passes were required for 3-in. thick sections and five to eight were required for the 6- and 9-in. thicknesses. The pilot work had indicated that two passes were sufficient for a 3-in. thickness and three for the 6-in. thickness.

The center 24-ft portion of the subbase was subgraded immediately before placing the portland cement concrete surfacing. Since the forms were set to the final grade of the pavement surface, it was necessary to change the depth of cutting for each change in pavement thickness. The excess material subgraded off the center portion was used to complete the subbase on the inside shoulder.

Table 20 is a record of subbase construction for all rigid pavement tangents. Dates apply to the placement of the material within the center portion of the roadbed.

3.3 CONSTRUCTION CONTROL

As with the embankment, an extensive program for construction control was initiated to insure uniformity of subbase throughout all test sections. This program included inspection of all details of the work and required considerable testing to determine the gradation of the material, maximum laboratory density, field density after compaction and after subgrading, and thickness of the completed subbase.

Routine gradation checks were made on the material as it was being loaded from the stockpile for delivery to the job to insure that segregation or degradation did not occur during loading. Samples also were obtained from the roadway after the material had been deposited and spread.

During placement of the material in 1957 as the sand-gravel mulch, six samples were obtained at random locations from each construction block on both flexible and rigid tangents. Routine sieve analyses on the No. 4, No. 40, and No. 200 sieves were made on all samples. One of each set of six samples was further used for complete sieve analysis and for CBR and plasticity tests. The results of the complete sieve analyses are summarized in Table 21 and are shown graphically in Figure 58. The results of the routine sieve analyses are plotted in Figure 59.

Plasticity tests, in every case, indicated that the subbase material was non-plastic. Laboratory CBR tests gave a mean value at 0.1-in. penetration of 34.7 with no value exceeding the specification maximum of 60. The results of these tests are shown in Figure 60.

In 1958, tests were made to check the gradation of the material placed in 1957 as sand-gravel mulch. Sieve analyses also were run on samples of the additional material placed in 1958 on those flexible pavement sections having 12-

TABLE 21

SUMMARY OF GRADATION TESTS[1] OF SUBBASE MATERIAL IN-PLACE (SUMMARY OF 99 TESTS)
(Obtained from Data System 2332)

Sieve	Gradation Formula and Tolerances	Mean Percent of Material Passing	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
1½ in.	100	100	..	100	..	..
1 in.	95–100	100	..	100	..	..
¾ in.	90–100	96.9	1.97	100	..	..
½ in.	90 ± 5	89.7	2.49	96.9	..	3.1
No. 4	73 ± 5	71.2	2.78	91.9	..	8.1
No. 40	27 ± 3	27.0	1.60	99.0	..	1.0
No. 200	7 ± 2	7.5	1.09	98.0	2.0	..

[1] AASHO Designation: T27–46.

and 16-in. thicknesses of subbase. The results of both sets of tests indicated no appreciable difference in gradation from that shown in Table 21.

The moisture content for compaction of the subbase was controlled by observation and by "feel." As previously indicated, the range of moisture content at which the specified density could be obtained was well within the maximum range permitted by the specifications.

Maximum laboratory density was determined by the method of AASHO Designation: T99-49, except that particles retained on the ½-in. sieve were removed from the samples and replaced with equal amounts by weight of material passing the ½-in. sieve and retained on the No. 4 sieve. At the beginning, two standard Proctor curves were developed from one composite sample obtained from each series of structural sections that were processed together. Later, only one curve was developed for each sample, and the number of samples was reduced as the range of maximum density was determined.

Field densities were determined in accordance with the method of AASHO Designation: T147-54, except that a rubber balloon apparatus

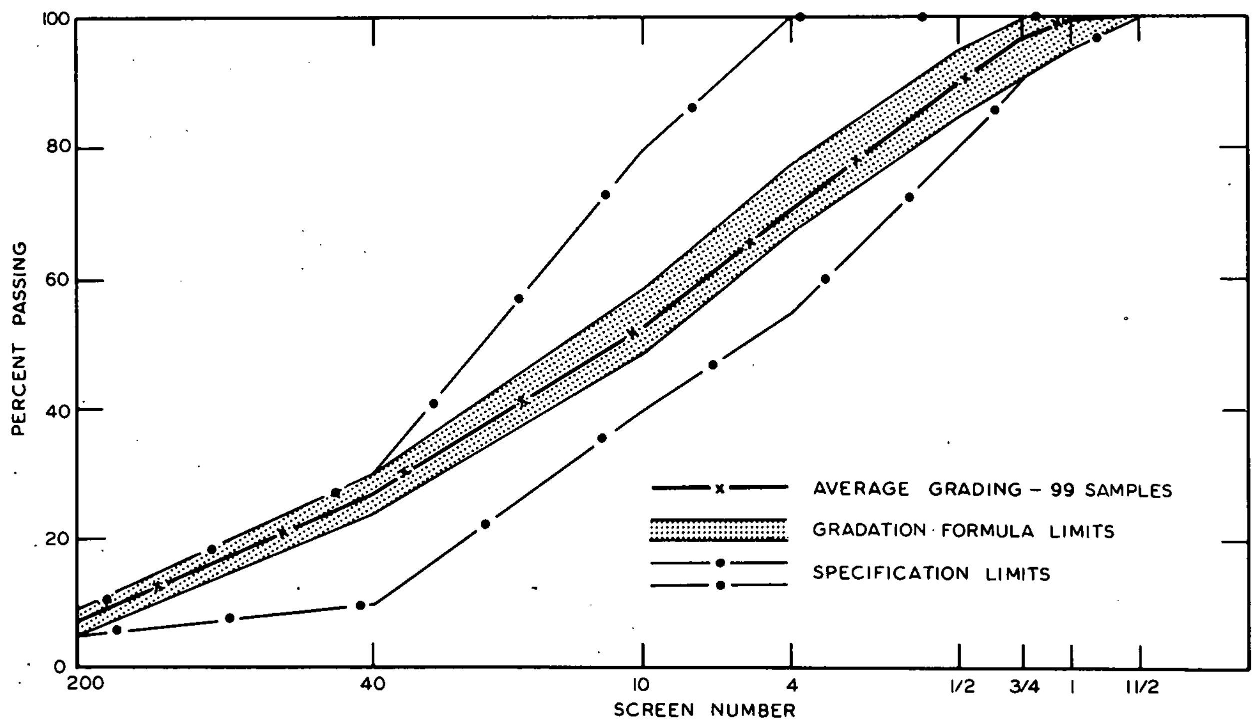


Figure 58. Gradation of subbase material in-place.

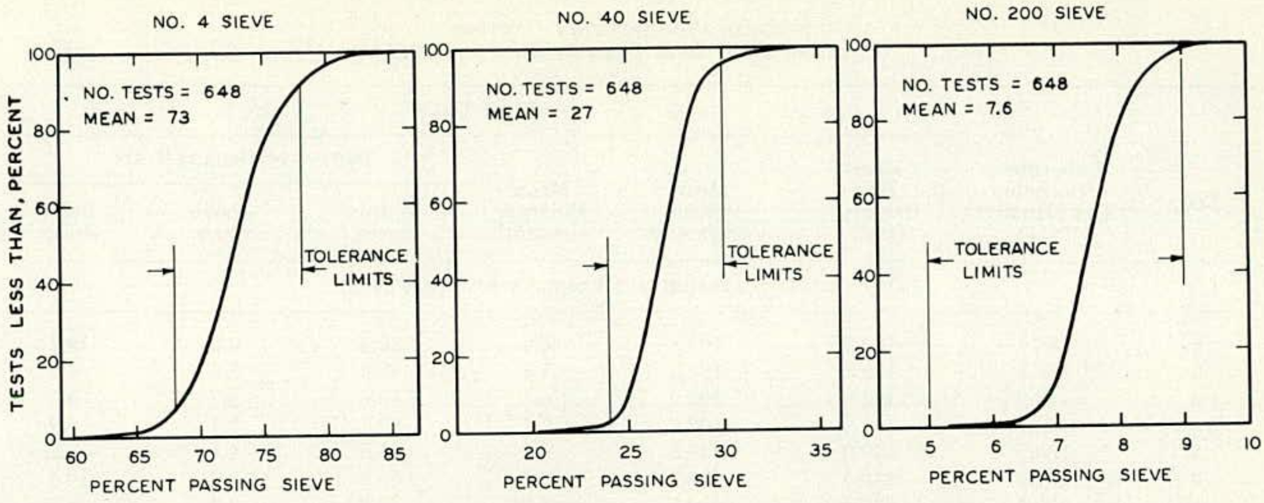


Figure 59. Gradation of subbase material in-place.

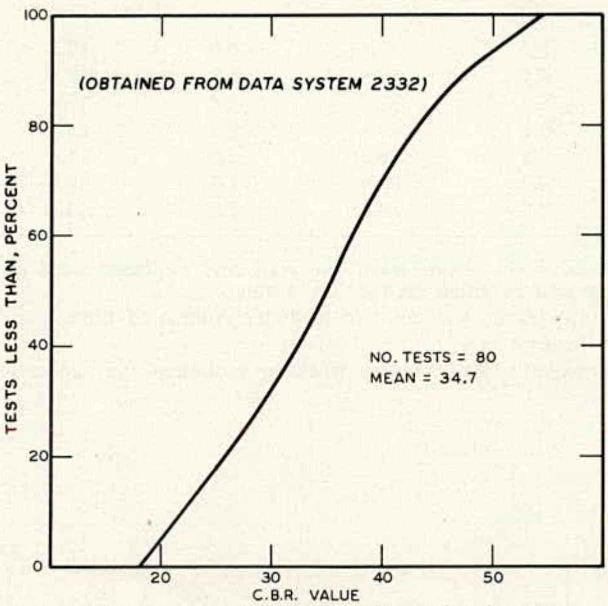


Figure 60. Laboratory CBR determinations on subbase material.

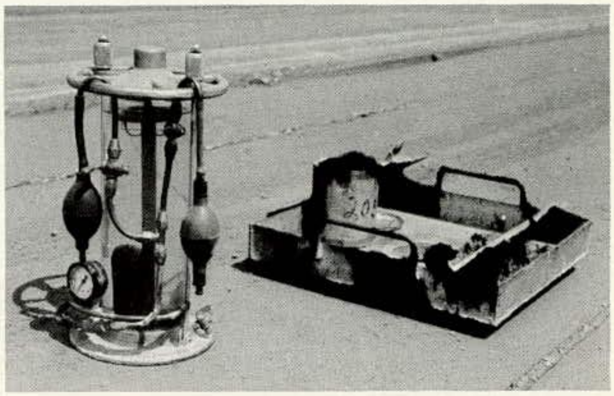


Figure 61. Volumeter used to measure volume of hole for field density test.

(Volumeter) was used to determine the volume of the hole. The volumeter was substituted for the sand cone to expedite the testing work. At least two field density tests were made at random locations on each layer of subbase material in each structural section, and at least four tests were made for acceptance or rejection. The density was determined for the full depth of the layer. In addition, the density was determined for both the top and bottom half in 8-in. layers on flexible tangents and 6- and 9-in. layers on rigid tangents. The procedure involved removing the top half of material and measuring the volume of the hole, then removing the remainder of the material and measuring the volume of the deepened hole. The density in the top and bottom were calculated separately and combined to calculate the density for the full depth of material.

The decision to accept or reject a layer of subbase in a section or series of sections was based on a statistical analysis of the compaction data for that layer and involved computing the mean percent compaction, the standard deviation, and the estimated percent of tests out of the specified limits. Acceptance was based on a maximum estimated 35 percent of tests out of specifications. The analysis procedure is described in Appendix A.

The results of the field density tests made during subbase construction are summarized by loops in Table 22, and are shown graphically in Figure 62 for flexible and in Figure 63 for rigid pavements. The curves illustrate the distribution of the results of compaction tests on all loops and the shaded areas represent the approximate limits within which the distribution curves for each of the loops are contained.

The estimated percent of compaction tests

TABLE 22

SUBBASE CONSTRUCTION CONTROL
(Obtained from Data Systems 2131 and 2221)

Loop	Laboratory Maximum Dry Density[1] (pcf)	Field Data: Mean Dry Density[2] (pcf)	Mean Percent Compaction[3]	Mean Moisture Content[4]	Percent of Density Tests: Within Specs.	Above Specs.	Below Specs.
			(a) FLEXIBLE PAVEMENTS (SUMMARY OF 979 TESTS)				
1	136.6	138.8	101.8	6.8	80.8	0.0	19.2
2	136.5	139.4	102.2	5.5	88.0	5.0	7.0
3	138.2	141.0	102.0	6.5	80.6	9.7	9.7
4	138.1	141.2	102.2	6.2	89.7	3.4	6.9
5	136.7	140.0	102.3	6.1	87.5	4.3	8.2
6	137.6	140.5	102.0	6.4	85.7	3.9	10.4
All	137.5	140.5	102.2	6.2	86.8	4.5	8.7
			(b) RIGID PAVEMENTS (SUMMARY OF 714 TESTS)				
1	136.0	137.7	101.3	5.7	82.1	0.0	17.9
2	135.7	137.7	101.5	6.4	80.8	1.3	17.9
3	138.6	140.7	101.5	6.0	81.8	2.8	15.4
4	137.7	139.9	101.6	6.4	83.4	3.2	13.4
5	135.6	138.1	101.8	6.5	85.5	3.2	11.4
6	136.7	138.9	101.6	6.2	85.4	1.3	13.3
All	136.9	139.1	101.6	6.3	83.8	2.2	14.0

[1] AASHO Designation: T99–49 except particles retained on ½-in. sieve were removed and replaced with an equal amount by weight of material passing the ½-in. sieve and retained on the No. 4 sieve.
[2] AASHO Designation: T147–54 except a rubber balloon apparatus was used to measure volume of hole.
[3] Specification limits, 100 to 105 percent of maximum dry density.
[4] Moisture contents computed with density tests, not comparable to optimum working moisture for moisture content prior to compaction.

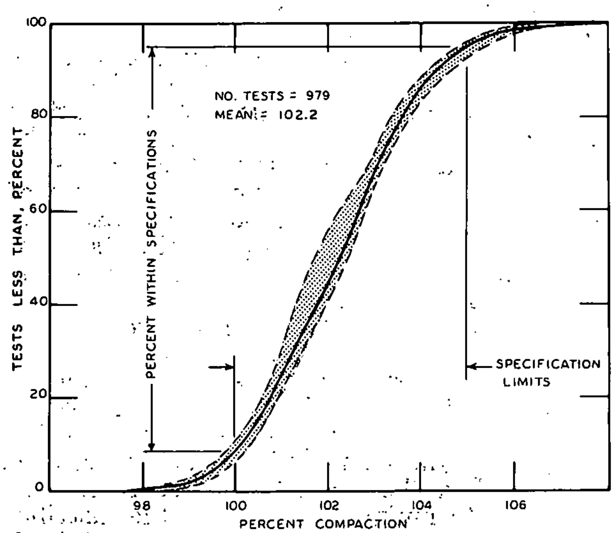


Figure 62. Subbase compaction on flexible tangents.

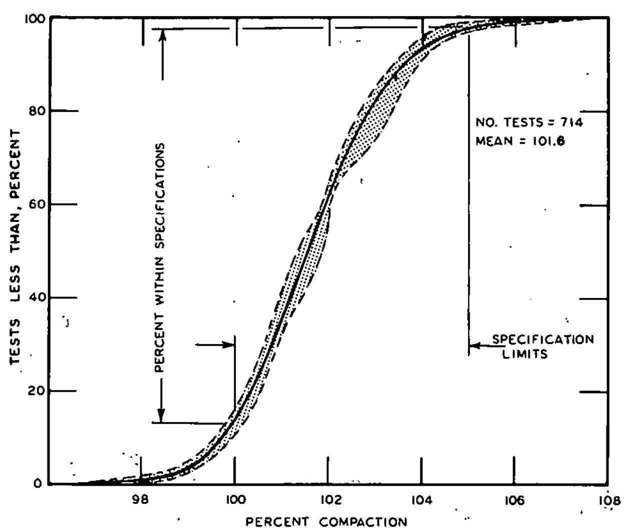


Figure 63. Subbase compaction on rigid tangents.

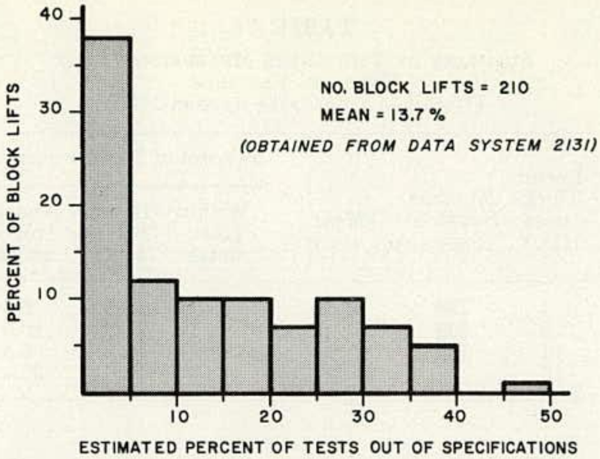


Figure 64. Subbase compaction, flexible tangent.

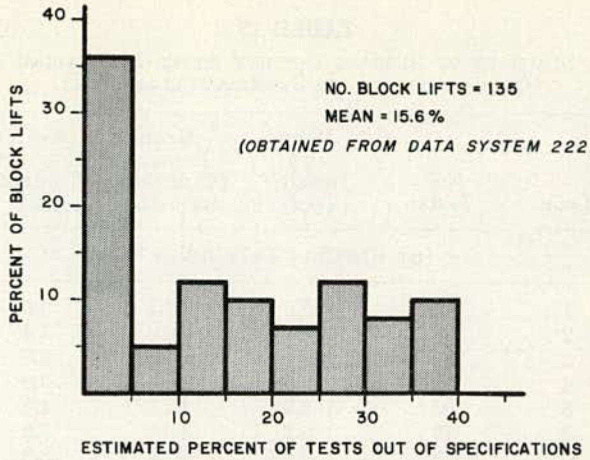


Figure 65. Subbase compaction, rigid tangent.

out of the specification limits for the construction blocks and units is shown graphically in Figure 64 (flexible) and Figure 65 (rigid pavements). The mean estimated values of 13.7 (flexible) and 15.6 (rigid) compare with the corresponding actual values of 13.2 and 16.2 given in Table 22.

Field density tests were also made on the subbase after subgrading. Generally, two tests were made at random locations on each structural section. The results are summarized by loops in Table 23, and are shown graphically in Figure 66 for flexible and in Figure 67 for rigid pavements.

Comparing Figure 66 with Figure 62 shows that on the flexible tangents the level of density after subgrading was approximately 5 percentage points lower than the level of density after compaction. Eighty-seven percent of all tests taken after compaction was within the specified range of 100 to 105 percent of maximum dry density. After subgrading, approximately the same percentage was within a range of 94 to 101 percent, and the mean had dropped from 102.2 to 97.6. The pilot studies had indicated that the density after subgrading could not be restored to the level obtained during compaction. Tests were made with pneumatic-tired and steel-wheel rollers of a weight that might be expected to restore the subbase density to the 100 to 105 percent level. Both tended to crack the surface and displace the material.

On rigid pavement tangents, the level of the subbase density after subgrading also was lower than that after compaction (97.7 percent as compared to 101.6 percent). However, comparing Figure 67 with Figure 63 shows that the distribution of test results within a 5 percent

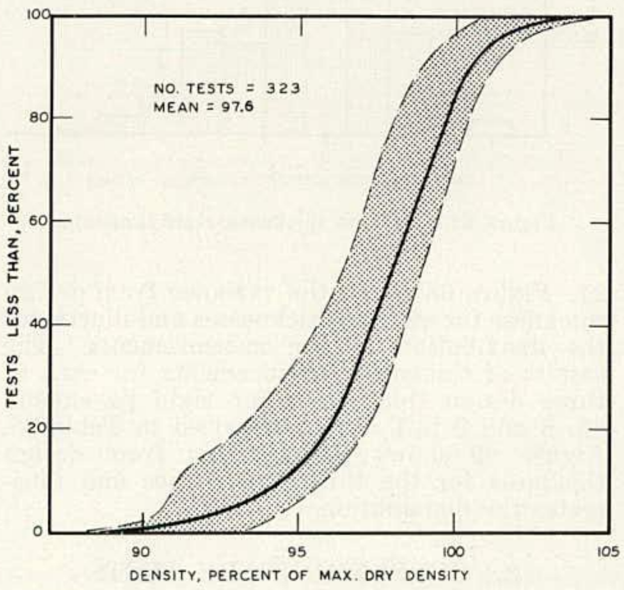


Figure 66. Subbase density after subgrading, flexible tangents.

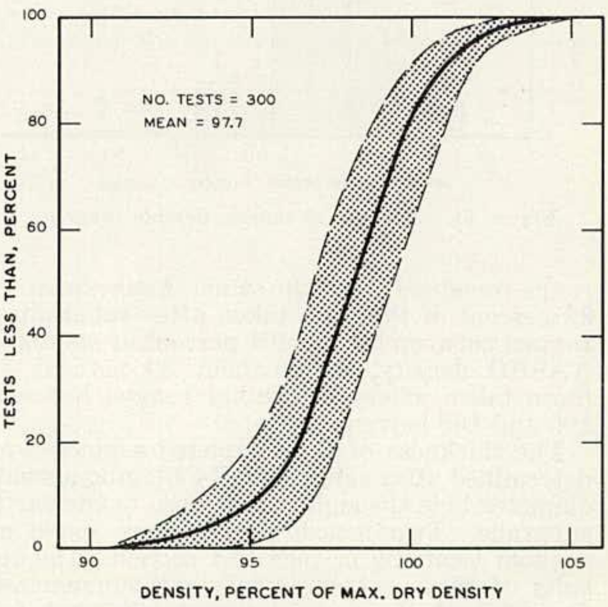


Figure 67. Subbase density after subgrading, rigid tangents.

TABLE 23

SUMMARY OF SUBBASE DENSITY AFTER SUBGRADING
(Obtained from Data Systems 2132 and 2222)

Loop	No. Tests	Mean Dry Density (pcf)	Mean Density (% of max. dry dens.)	Mean Moisture Content (%)
(a) FLEXIBLE PAVEMENTS				
1	15	133.0	97.4	3.6
2	26	132.1	96.3	3.4
3	30	132.2	95.8	3.5
4	82	136.1	98.6	3.9
5	93	132.9	97.5	4.0
6	77	136.2	98.7	3.9
All	323	134.5	97.6	3.8
(b) RIGID PAVEMENTS				
1	28	132.4	97.4	3.8
2	34	134.1	98.8	4.4
3	60	133.6	96.5	4.2
4	61	134.6	97.8	4.9
5	55	134.0	98.8	5.0
6	62	132.9	97.3	4.7
All	300	133.7	97.7	4.6

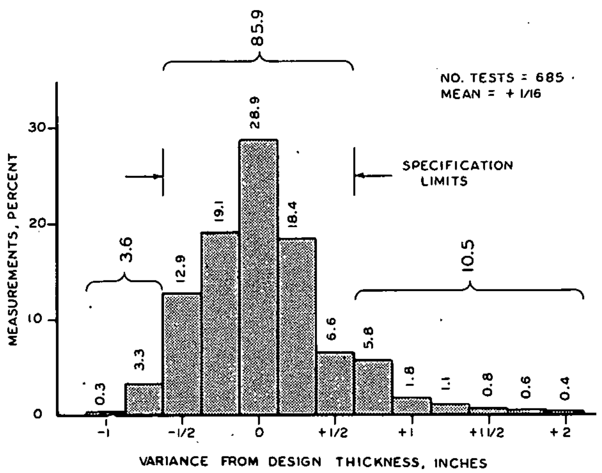


Figure 68. Subbase thickness, flexible tangents.

TABLE 24

SUMMARY OF THICKNESS MEASUREMENTS OF SUBBASE, FLEXIBLE
(Obtained from Data System 2130)

Design Thickness (in.)	Number Measurements	Mean (in.)	Percent of Measurements: Within Tolerances	Below Tolerances	Above Tolerances
4	288	4	90.0	3.8	6.2
8	209	8	83.7	5.8	10.5
12	120	12 1/8	90.9	0.8	8.3
16	68	16 7/16	66.1	1.5	32.4

TABLE 25

SUMMARY OF THICKNESS MEASUREMENTS OF SUBBASE, RIGID
(Obtained from Data System 2220)

Design Thickness (in.)	Number Measurements	Mean (in.)	Percent of Measurements: Within Tolerances	Below Tolerances	Above Tolerances
3	169	3 3/16	81.7	7.1	11.2
6	261	6 1/4	72.1	4.2	23.7
9	117	9 1/8	82.9	7.7	9.4

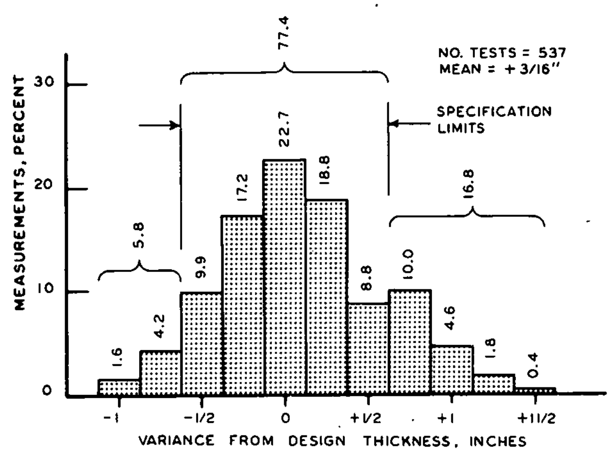


Figure 69. Subbase thickness, rigid tangents.

range remained about the same. Approximately 82 percent of the tests taken after subgrading ranged between 96 and 101 percent of standard AASHO density, while about 84 percent of those taken after compaction ranged between 100 and 105 percent.

The thickness of the completed subbase was determined after subgrading by digging a small diameter hole through the subbase to the earth subgrade. Two measurements were made at random locations in each test section. The results of these measurements are summarized for each of the four design thicknesses for flexible pavements (4, 8, 12 and 16 in.) in Table 24. Figure 68 shows the variance from design thickness for all four thicknesses and illustrates the distribution of the measurements. The results of thickness measurements for each of three design thicknesses for rigid pavements (3, 6 and 9 in.) are summarized in Table 25. Figure 69 shows the variance from design thickness for the three thicknesses and illustrates the distribution.

3.4 SUPPLEMENTARY TESTS

The moisture content of the subbase on both flexible and rigid pavement tangents was de-

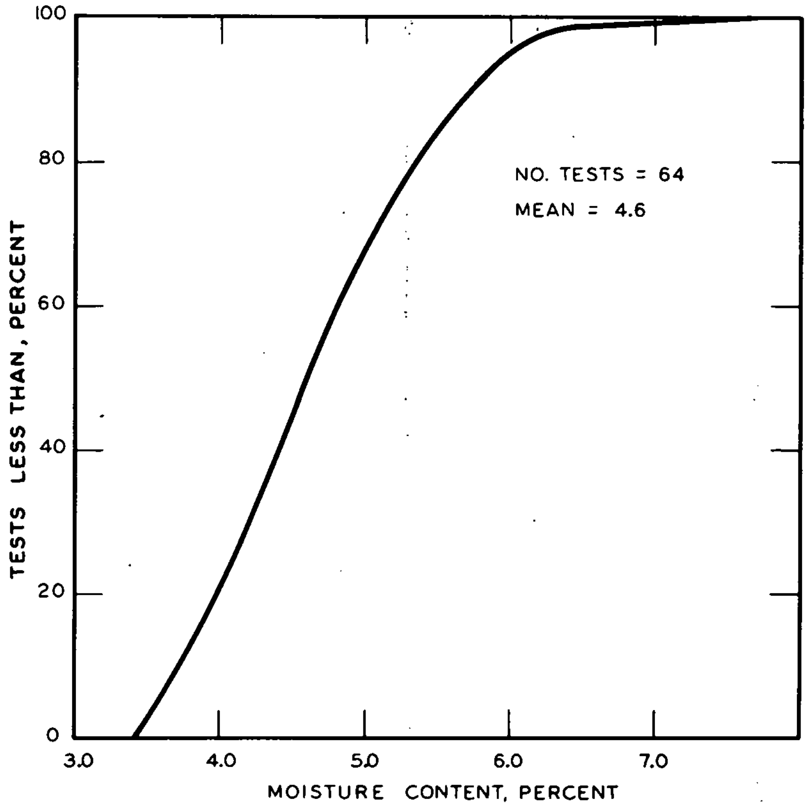


Figure 70. Subbase moisture content on flexible tangents after paving.

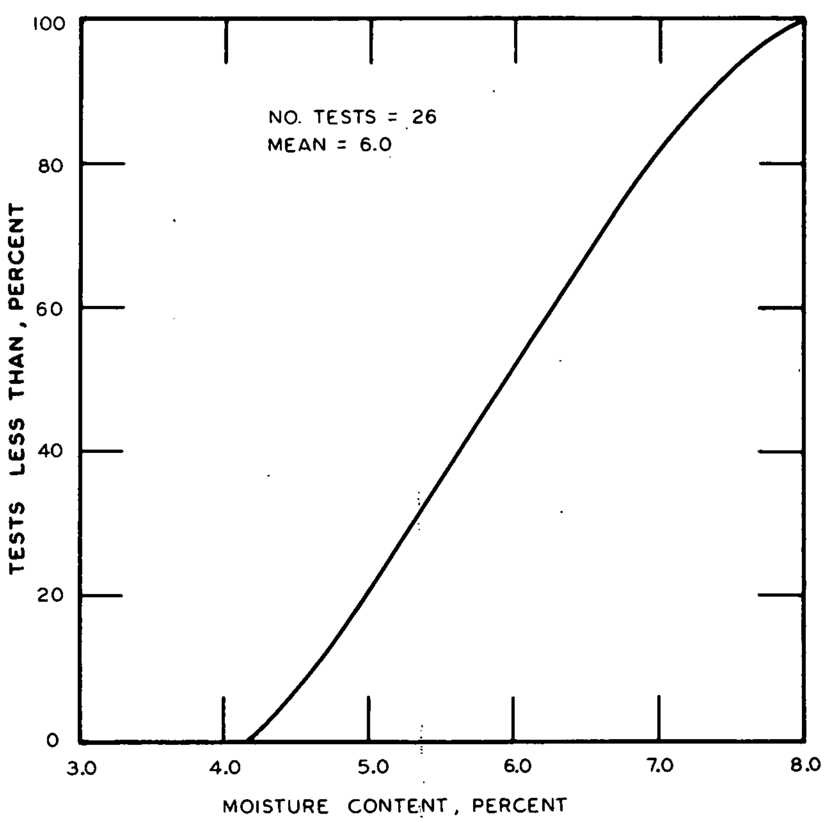


Figure 71. Subbase moisture content on rigid tangents after paving.

TABLE 26

MOISTURE CONTENT OF SUBBASE AFTER PAVING
(Obtained from Data Systems 2111 and 2211)

Loop	No. Tests	Mean Moisture Content (%)
(a) FLEXIBLE PAVEMENTS		
1	6	5.7
2	4	5.9
3	8	5.0
4	31	4.2
5	9	4.7
6	6	4.4
All	64	4.6
(b) RIGID PAVEMENTS		
2	4	6.3
3	4	4.5
4	10	6.4
5	2	6.0
6	6	6.0
All	26	6.0

termined when the completed pavements were cored. The results of the tests are summarized by loops in Table 26. The distribution of the individual values is shown in Figure 70 for the flexible and in Figure 71 for the rigid pavements. A comparison of Tables 23 and 26 indicates that the moisture content of the subbase on both flexible and rigid tangents increased approximately 1 percent during the period from the subgrading of the subbase to the completion of the paving. The mean moisture content on flexible pavement tangents increased from 3.8 to 4.6 percent, and on the rigid pavement tangents from 4.6 to 6.0 percent.

Tests were conducted in the laboratory to determine the permeability of the subbase material. Samples were obtained from a stockpile of aggregate salvaged during construction, and the fraction of material passing the No. 200 sieve was about 1.5 percent higher than that of the subbase material in place (approximately 10 percent as compared to 8.5). The tests were run in a constant head permeameter, both with and without the use of vacuum. The results of sixteen tests conducted without using vacuum gave a mean permeability value at 20 C of 4.58 ft per day (1.61×10^{-3} cm/sec). The corresponding mean moisture content and density were 5.6 percent and 130.8 pcf. The values ranged from 0.02 ft per day at 7.6 percent moisture and 135.5 pcf to 10.35 ft per day at 5.8 percent moisture and 128.8 pcf. Eight tests were run with vacuum saturated samples. The mean permeability value at 20 C was 3.95 ft per day (1.40×10^{-3} cm/sec), and the mean moisture content and density were 5.8 percent and 132.0 pcf, respectively. The results of the eight tests ranged from 0.01 ft per day at 7.0 percent moisture and 138.8 pcf to 11.60 ft per day at 5.5 percent moisture and 129.6 pcf.

Additional information on the subbase material obtained through the Cooperative Materials Testing Program is summarized in Appendix C.

The results of the routine sieve analyses for the No. 4, No. 40 and No. 200 sieves on samples of the in-place material were analyzed by plotting the means and ranges of each set of six samples on statistical quality control charts.* These analyses indicated that a good control was obtained.

* ASTM Manual on Quality Control of Materials, Special Technical Publication 15-C, American Society for Testing Materials (January 1951).

Chapter 4

Base Course—Flexible Pavement

This chapter describes the materials and discusses the procedures and controls for the construction of the crushed stone base in the main factorial experiment sections and the special bases used in the sections included in a separate experiment where base-type was a variable.

4.1 CRUSHED STONE BASE

The base course for the sections included in the main factorial experiment in each flexible pavement tangent was constructed of crushed dolomitic limestone. The center portion consisted of a special gradation of crushed stone termed "Crushed Stone Base Course, Special"; the shoulder portions were a standard gradation of crushed stone used by the Illinois Division of Highways termed "Crushed Stone Base Course, Type A" (see Fig. 11, Section 1.3.2).

4.1.1 Materials and Materials Control

In an effort to obtain uniformity of gradation, the specifications for the Crushed Stone Base Course, Special, required that the material be produced in two separate sizes, Size C coarse aggregate and Size D fine aggregate, and be combined in proper proportions on the job site.

The specified gradation limits were considered only as maximum and minimum values. The contractor was required to submit a gradation formula, stating single percentages for the amount of material passing each sieve. The specifications set plus and minus tolerances for variations from the approved formula.

Table 27 gives the specification requirements for the Crushed Stone Base Course, Special, and the approved gradation formula.

To meet the requirement that the two sizes of aggregate be obtained from a single source, it was necessary to add a small amount of dolomitic stone screenings (100 percent passing No. 40 sieve and approximately 20 percent passing No. 200 sieve) to the mixture. The two aggregates and the screenings were pro-

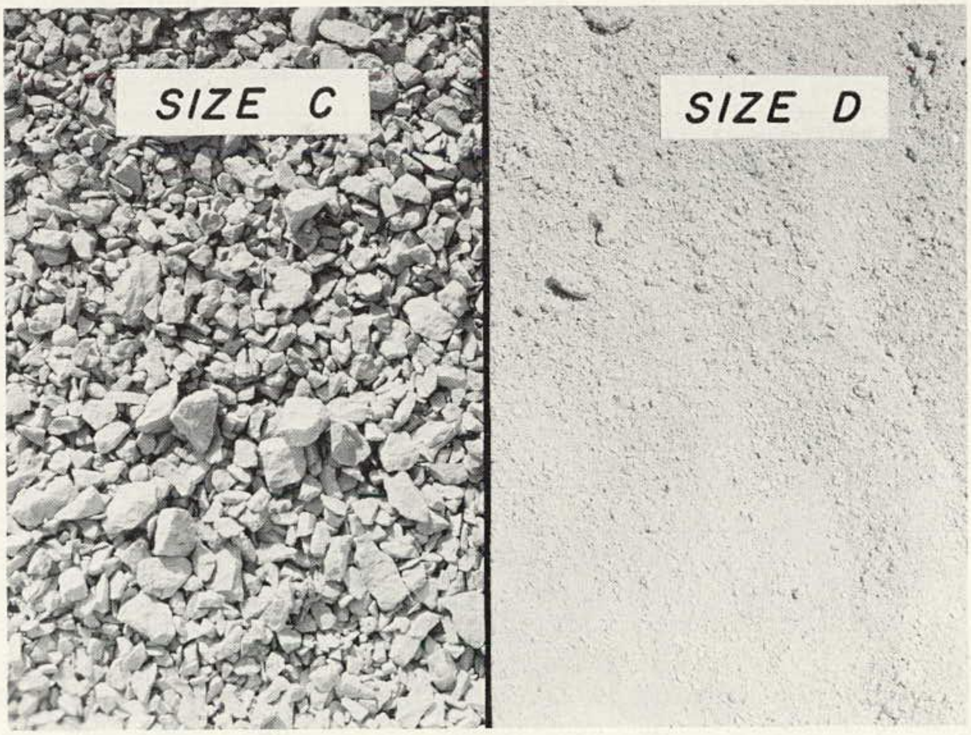


Figure 72. Aggregates for crushed stone base course, special.

TABLE 27

SPECIFICATION REQUIREMENTS, CRUSHED STONE BASE COURSE, SPECIAL

(a) GRADATION[1]

Sieve	Gradation Limits (% passing)			Graduation Formula (% passing)	Tolerances (%)
	Size C	Size D	Mixture		
1½ in.	100		100	100	
1 in.	65–80		80–100	88	±5
¾ in.	50–70		70–90	83	±5
½ in.	30–50	100	60–80	70	±5
No. 4	0–5	95–100	40–60	46	±5
No. 10		65–85	28–46	35	±5
No. 40		30–60	16–33	21	±5
No. 100		15–35	7–20	14	±3
No. 200		6–25	3–12	10	±2

(b) DELETERIOUS SUBSTANCES

Substance	Maximum Percent, by Weight		Determined by Test Designation
	Size C[2]	Size D[1]	
Soft and unsound fragments, including chart	5.0	5.0	ASTM C235–57T
Coal and lignite	1.0	1.0	
Clay lumps	0.25	0.25	AASHO T112–56
Material finer than the No. 200 sieve	1.0	...	AASHO T11–49
Other deleterious or injurious Material	1.0	1.0	
Thin or elongated pieces (5 times average thickness)	15.0	15.0	

(c) OTHER REQUIREMENTS

Characteristic	Requirement	Test Procedure
PI	Not more than 4	AASHO Designation: T91–54
Liquid limit	Not more than 25	AASHO Designation: T89–54
CBR	75 or greater	—[3]
Percentage of wear (Los Angeles machine)	Not to exceed 35	AASHO Designation: T96–56
Soundness (5 cycles sodium sulfate)	Not to exceed 15	AASHO Designation: T104–57

[1] AASHO Designation: T27-46.
[2] Combined amount of the first five items shall not exceed 5 percent.
[3] "Suggested Method of Test for Bearing Ratio and Expansion of Soils," pp. 109–116, "Procedures for Testing Soils," ASTM (Sept. 1944).

TABLE 28

SUMMARY OF PLANT GRADATIONS ON CRUSHED STONE BASE COURSE, SPECIAL
(SUMMARY OF 170 TESTS)
(Obtained from Data System 2125)

Sieve	Gradation Formula and Tolerances	Mean Percent of Material Passing	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
1½ in.	100	100	—	100	—	—
1 in.	90 ± 5	90	3.04	93.4	1.8	4.8
¾ in.	81 ± 5	81	3.65	88.7	1.8	9.5
½ in.	68 ± 5	68	3.53	88.0	6.0	6.0
No. 4	50 ± 5	48	2.24	94.7	0.0	5.3
No. 10	35 ± 5	35	1.55	99.4	0.0	0.6
No. 40	21 ± 5	20	0.90	100	0.0	0.0
No. 100	14 ± 3	13.5	0.68	100	0.0	0.0
No. 200	10 ± 2	10	0.60	100	0.0	0.0

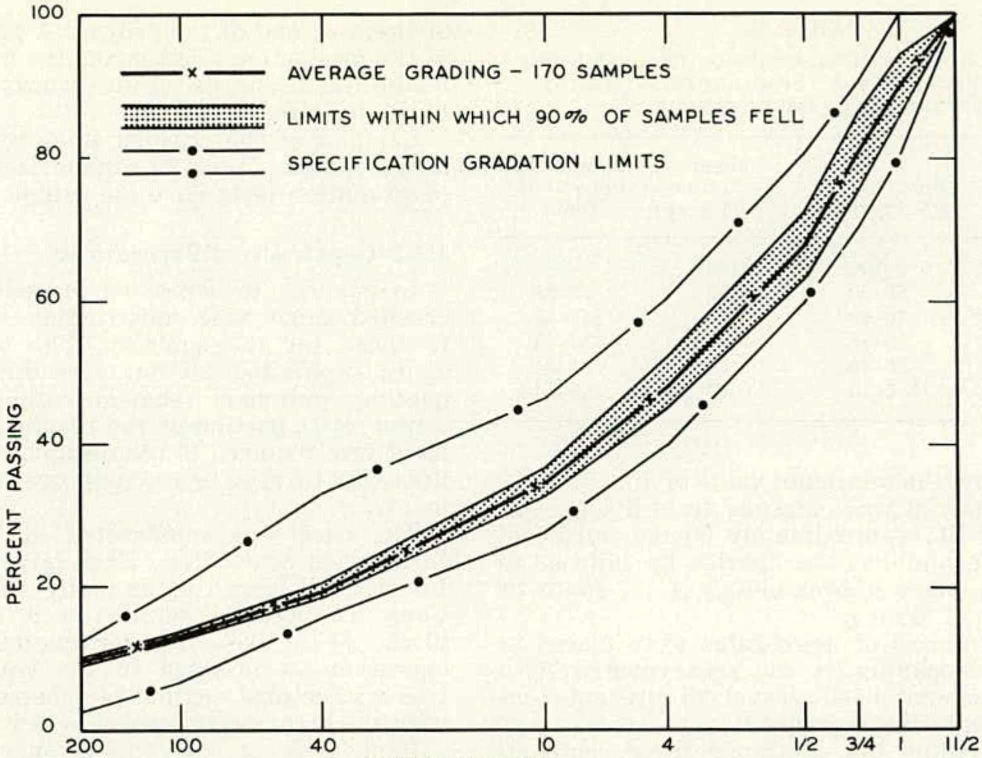


Figure 73. Gradation of crushed stone base, special—plant samples.

portioned by weight and mixed with water in a paving mixer on the job site. Samples were obtained at the discharge end of the mixer. Results of tests made on these samples are summarized in Table 28 and are shown graphically in Figure 73.

The material was obtained from a source used to supply aggregates for regular highway construction. Soundness tests on the Size C coarse aggregate (five cycles of the sodium sulphate test) indicated a weighted average loss of 7 percent, with a range between 3.8 and 10.7 percent. The percentage of wear (Los Angeles machine) was 28 at 500 revolutions and the individual tests ranged from 25.3 to 30.7. The bulk saturated specific gravity and percent absorption was 2.64 and 1.5, respectively, for the Size C coarse aggregate; and 2.71 and 1.1, respectively, for the Size D fine aggregate.

The crushed stone mixture was non-plastic, and its laboratory CBR value averaged 107.7 for eleven tests. The test results ranged between 52 and 160 with two of the eleven tests

Figure 74. Batch plant—crushed stone base, special.

TABLE 29

SUMMARY OF PLANT GRADATIONS ON CRUSHED STONE BASE COURSE, TYPE A (SUMMARY OF 252 TESTS)
(Obtained from Data System 2127)

Sieve	Spec. Limits (% Passing)	Mean Gradation (% Passing)	Range of 90 Percent of Tests
1 in.	100	100	—
½ in.	60–90	82	73–88
No. 4	40–60	50	42–58
No. 8	25–50	37	30–43
No. 16	20–40	29	24–34
No. 200	5–15	12	9.5–14.5

being below the minimum value of 75.

The material was obtained from a source at Lockport, Ill., approximately 50 mi. northeast of Ottawa, and was transported by railroad to the batch plant approximately 1 mi south of the center of Loop 4.

The two sizes of aggregates were placed in separate stockpiles in an area covered with bituminous stabilized gravel to prevent contamination of the aggregates.

The batching bin contained three compartments. The weigh hopper, located immediately below the bin, was equipped to weigh the individual sizes accumulatively. The coarse aggregate was placed in one compartment of the bin by a conveyor belt. The material was removed from a vertical face of the stockpile by a front-end loader and placed in the hopper over the conveyor belt. To prevent segregation at the end of the belt, a baffle and step plates were added to confine the material and direct its flow into the bin. The fine aggregate and screenings were placed directly into the bins by a crane and clam bucket, as segregation of these two materials was not a problem.

The three materials were proportioned by weight in 5,000-lb batches, containing 57 percent Size C coarse aggregate, 38 percent Size D fine aggregate and 5 percent stone screenings. The batches were delivered to the job site in trucks having four batch compartments and were mixed with water in a dual drum 34-E paving mixer.

Additional information on the material for the Crushed Stone Base Course, Special, was obtained through the Cooperative Materials Testing Program. Summaries of material characteristics and design properties are included in Appendix C.

The material specifications for the Crushed Stone Base Course, Type A, were the same as those used by the Illinois Division of Highways. The gradation was controlled between the specification limits without the use of a gradation formula and tolerances.

The source of the Type A base material was located at Utica, Ill., approximately 4 mi south of the west end of the project. A pugmill mixer at the production plant mixed the material with water and discharged it into trucks for delivery to the job site.

Samples of the crushed stone were obtained at the mixer. Table 29 summarizes the results of gradation tests on these samples.

4.1.2 Construction Procedures

In general, the specified procedures for the crushed stone base construction were similar to those for the subbase. The specifications again prohibited all but spreading and compacting equipment from operating within the center 24-ft portion of the roadbed. All equipment was required to use designated transition areas for turning or crossing over the embankment.

The base was constructed in layers 3 in. thick when compacted. Each layer was spread for the full length of as many structural sections as possible within each construction block. At the close of a working day, spreading operations terminated in the transitions between structural sections or construction blocks with the layer being spread over its full width.

Each layer of base was compacted immediately after placement. The specifications permitted the use of a self-propelled pneumatic-tired roller, a 3-wheel roller, or a combination of both. However, no change in rollers or combinations of rollers was permitted after commencing construction on the test tangents, and use of identical rollers was required in all test loops.

The control of moisture content and density was identical to that for the subbase. The moisture content at time of spreading was to be within plus or minus one percentage point of the optimum working moisture content, and compaction between 100 and 105 percent of standard AASHO density.

The specifications required that the center 24 ft of the base course be finished with a mechanical subgrading machine operating on steel forms. Filling of low areas was not permitted during subgrading operations. Where the surface was below the elevation of the final grade, material was added and the entire top layer was loosened within the section or sections containing the low areas, adjusted for moisture content if necessary, and recompacted to the required density before trimming and finishing the surface. The specifications permitted a tolerance of ± ⅛ in. from established grade.

From the pilot studies, an optimum working moisture content was established at 7 percent for the Crushed Stone Base Course, Special, and at 8 percent for the Crushed Stone Base Course, Type A. It was determined that a combination of the two types of rollers was most effective. Usually, four passes of a pneumatic-tired roller followed by two coverages of a 3-wheel roller were sufficient to obtain the specified density on a 3-in. layer. The pneumatic-tired roller was self-propelled and had eleven tires on two axles. It was operated at a gross weight of 8 tons and developed 160 lb per in. of tire tread width. The 3-wheel roller, 7- to 10-ton capacity, developed a compression weight of 275 lb per in. of drive-roll width.

Construction within a loop began at the west end of each tangent and progressed eastward. As soon as possible after subgrading the subbase or embankment within a construction block, the initial 3-in. layer of base was placed on the sections whose design included a base course. This was followed by placing the second layer on the structural sections having a 9-in. base thickness and then the top layer for the 6- and 9-in. thicknesses.

The construction of a layer of base course within a section was started with the special base material on the center portion and then continued with the Type A base material on the inside shoulder. Each layer of base for a particular section was completed on both the center portion and the inside shoulder before the next layer was placed. The crushed stone base on the outside shoulders was placed at a later date.

The material was placed with a self-propelled mechanical spreader identical to that used for subbase construction. Water was mixed with the special gradation of base material in a dual drum 34-E paving mixer equipped with a belt to convey the material to the spreader. A pugmill mixer was used to adjust the moisture content of the Type A base material at the production plant, and the paving mixer was used as a means of conveying this material to the spreader.

As with the subbase, each layer was maintained in a moist condition until the next layer was placed. The completed base course was kept damp until it was subgraded and primed.

Subgrading of the completed base was performed just prior to applying the prime coat. It was subgraded ¼ in. above final grade to allow for compaction of loose material on the surface. A short section of screen was attached to the conveyor of the subgrading machine to produce some fine material as the excess from the center was deposited on the inside shoulder. The fines were distributed along the subgrade planer behind the machine to fill the small holes caused by the coarse aggregate being pulled out of the base. The subgraded base was recompacted with a 6- to 8-ton 3-wheel roller having a weight of 209 lb per in. of drive-roll width. The loose material on the surface of the base was keyed in place by one pass of the roller. This was followed by alternate wetting and rolling until the subgraded surface was smooth and free of

TABLE 30

RECORD OF BASE COURSE CONSTRUCTION, FLEXIBLE

Const. Block No.	Const. Sequence[1]	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6
F-1	1	8-30-58	8-26-58	5-26-58	7-21-58	8-5-58	6-27-58
	2	9-11/12-58	9-3/4-58	6-4-58	7-29/8-4-58	8-14/18-58	7-2/11-58
F-2	1	9-2-58	8-26-58	5-26/27-58	7-21-58	8-5/6-58	6-30-58
	2	9-12/13-58	9-3/5-58	6-5-58	7-29/8-4-58	8-18/20-58	7-2/11-58
F-3	1	9-3/4-58	8-27-58	5-27, 6-27-58	7-21/22-58	7-1, 7-22, 8-6-58	6-30-58
	2	9-12/15-58	9-4/6-58	6-16, 7-2, 7-8-58	7-29/8-4-58	7-24, 8-8, 8-27-58	7-8/11-58
F-4	1		8-27/28-58	5-28-58	7-22-58	7-1, 7-22, 8-6-58	6-30/7-1-58
	2		9-4/9-58	6-5/16-58	8-2/4-58	7-25, 8-8, 8-27-58	7-8/11-58
F-5	1		8-28-58	5-28/29-58	7-23-58	8-6/7-58	7-1/8-58
	2		9-8/9-58	6-5/17-58	8-2/4-58	8-22/28-58	7-17-58
F-6	1		8-29-58	5-29/6-5-58	7-24-58	8-7/8-58	6-28, 7-9-58
	2		9-8/9-58	6-16/17-58	8-5/7-58	8-25/28-58	7-18, 7-23-58
F-7	1		8-29-58	6-6/7-58	7-25/29-58	8-8/11-58	6-28, 7-9-58
	2		9-8/9-58	6-18/19-58	8-5/8-58	8-25/29-58	7-18, 7-21, 7-24-58
F-8	1			6-17/23-58	7-29-58	8-11/12-58	7- 9-58
	2			6-24/26-58	8-6-58	8-26/29-58	7-28-58
F-9	1			6-17/18-58	7-18, 8-4-58	8-13-58	7-10/17-58
	2			6-24/26-58	7-22, 8-8, 8-14-58	8-26/9-2-58	7-18/28-58
F-10	1			6-21, 6-28-58	7-19, 8-4-58	8-13-58	7-17-58
	2			6-28, 7-1, 7-8-58	7-23, 8-7, 8-14-58	8-27/9-2-58	7-24/28-58

[1] Construction Sequence 1 is subgrading for base; Sequence 2 is base course construction.

Figure 75. Depositing and spreading crushed stone base; planks in transition for equipment crossing.

cracks and depressions, and the material was at the required density. The formation of a lather on the surface was used as a guide to control the amount of water and compactive effort applied.

A chronological record of construction of the base course within the center portion of the roadbed is given in Table 30. The dates given include base course construction for the special base-type wedge sections as well as the crushed stone base course.

4.1.3 Construction Control

Construction control of the crushed stone base course involved extensive testing to determine the gradation of the in-place material, the maximum laboratory density, field moisture content before compaction, field density after compaction and after subgrading, and the thickness of the completed base. All data presented in this section concern the Crushed Stone Base Course, Special, used in the center portion of the roadbed, and were obtained from tests on samples of the in-place material.

One sample for gradation tests was obtained from each 3-in. layer in each structural section. The results of these tests are summarized in Table 31 and shown graphically in Figure 76. These data indicate that no appreciable change in average gradation occurred during the handling and spreading of the material (see Table 28, Section 4.1.1). A slight increase (1 to 2 percent) occurred in the fraction of material finer than the No. 4 sieve size. The percentage of tests within the specification limits was

TABLE 31

GRADATION CRUSHED STONE BASE COURSE,[1] SPECIAL; SUMMARY FROM ROAD (SUMMARY OF 271 TESTS)
(Obtained from Data System 2126)

Sieve	Gradation Formula and Tolerances	Mean Percent of Material Passing	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
1½ in.	100	100	—	100	—	—
1 in.	90 ± 5	90	3.52	88.5	7	4.5
¾ in.	81 ± 5	80	4.59	75.5	8	16.5
½ in.	68 ± 5	68	4.69	75	12.5	12.5
No. 4	50 ± 5	50	3.56	83.5	9	7.5
No. 10	35 ± 5	36	2.21	92.5	7	0 5
No. 40	21 ± 5	21	1.46	99.5	0.5	0.0
No. 100	14 ± 3	14.5	1.02	98.5	1.5	0.0
No. 200	10 ± 2	11.5	0.86	81.5	18.5	0.0

[1] AASHO Designation: T27–46.

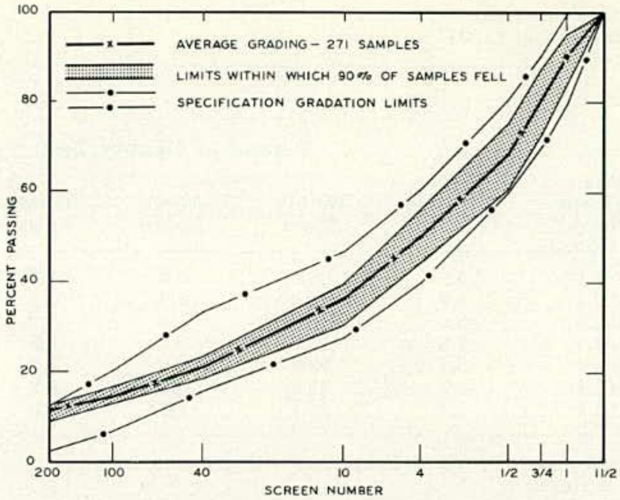


Figure 76. Gradation of crushed stone base, special—samples from road.

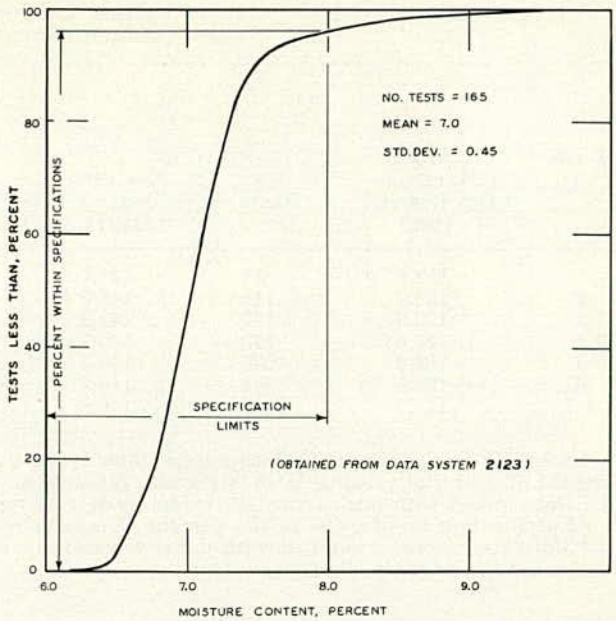


Figure 77. Moisture content of crushed stone base course, special.

somewhat higher for the samples obtained at the plant.

Frequent checks on moisture content were made during placement of special base material. Samples were obtained at the paving mixer at an approximate rate of one sample for every two hours of production. As shown in Figure 77, the mean moisture content of 7.0 percent was the same as the established optimum working moisture content. Approximately 96 percent of all tests were within the permitted plus or minus one percentage point.

The maximum laboratory density was determined by the method of AASHO Designation: T99-49, except that particles retained on the ½-in. sieve were removed from the samples and were replaced with an equal amount, by weight, of material passing the ½-in. sieve and retained on the No. 4 sieve. At the beginning of construction, one standard moisture-density curve was developed for each of two composite samples obtained from each layer of base in each block or series of structural sections constructed as a unit. Later, as the range of maximum laboratory density was determined, only two to four tests were made per day.

Four field density tests were made at random

Figure 78. Determining field density of base with nuclear density equipment.

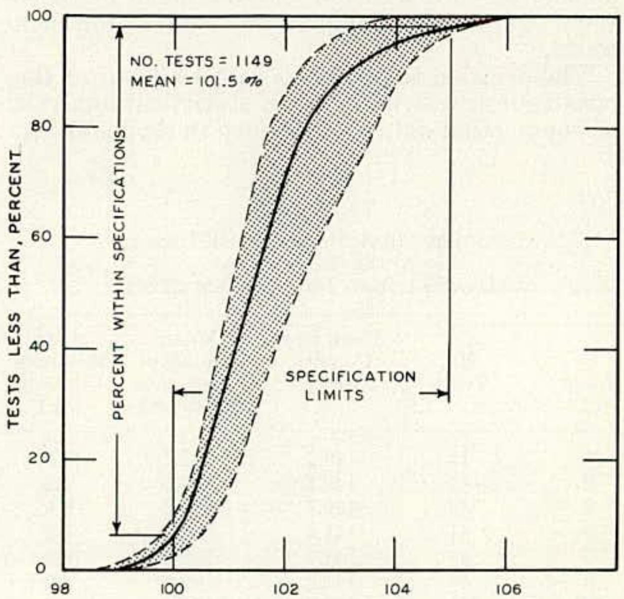


Figure 79. Crushed stone base compaction.

TABLE 32

BASE CONSTRUCTION CONTROL
(Obtained from Data System 2123)

Loop	Laboratory Maximum Dry Density[1] (pcf)	Field Data: No. Tests	Mean Dry Density[2] (pcf)	Mean Percent Comp.[3]	Mean Moisture Content[4]	Percent of Density Test: Within Specs.	Above Specs.	Below Specs.
1	138.8	59	141.1	101.6	5.9	98.3	0.0	1.7
2	138.7	144	142.0	102.4	5.9	93.0	4.9	2.1
3	138.9	190	141.9	102.2	5.9	85.8	5.3	8.9
4	138.6	172	140.1	101.1	5.6	93.6	0.6	5.8
5	138.6	280	140.5	101.4	6.1	93.9	0.0	6.1
6	138.8	306	140.3	101.1	5.7	91.8	0.0	8.2
All	138.7	1149	140.8	101.5	5.8	92.1	1.6	6.3

[1] AASHO Designation: T99–49 except plus ½-in. material was removed and replaced with an equal amount by weight of material passing ½-in. sieve and retained on No. 4 sieve.
[2] Determined with nuclear moisture-density testing equipment.
[3] Specification Limits, 100 to 105 percent of maximum dry density.
[4] Moisture content computed with density tests; not comparable to optimum working moisture for moisture control prior to compaction.

locations on the center portion of each layer of material in each structural section using nuclear density measuring equipment. The equipment is described in HRB Special Report 38. The variability and reliability of the equipment is described in a paper entitled "Evaluation of Nuclear Moisture—Density Testing Equipment", by W. N. Carey, Jr., J. F. Shook, and J. F. Reynolds, presented at the ASTM Annual Meeting, June 1960, Atlantic City, New Jersey. Prior to its use, the equipment was calibrated in the laboratory to establish a curve of gamma ray counts versus wet density, and the results were checked against those obtained by standard methods. The count used to determine the density at each location was the mean of three separate counts. The gage was rotated for each count.

The decision to accept or reject a layer of the base course was based on a statistical analysis of compaction data as described in Appendix A.

TABLE 33

CRUSHED STONE BASE COURSE DENSITY AFTER SUBGRADING
(Obained from Data System 2124)

Loop	No. Tests	Mean Dry Density (pcf)	Mean Density (% of max. dry dens.)	Mean Moisture Content (%)
1	11	140.1	102.6	5.3
2	41	140.6	101.3	4.5
3	36	139.3	100.3	3.8
4	41	141.4	102.0	3.2
5	45	141.7	102.2	4.3
6	44	141.6	102.1	4.9
All	218	140.9	101.7	4.2

Acceptance was based on an allowable estimated 35 percent out of specification limits.

Test data showing the compaction control attained during construction are given in Table 32 and Figure 79.

The estimated percentage of tests out of the specified limits computed from the compaction data for the block-lifts is shown graphically in Figure 80. The mean estimated percentage of tests outside the specified limits was 8.0 as compared to the actual 7.9 in Table 32.

The in-place density also was determined after subgrading the base course. Approxi-

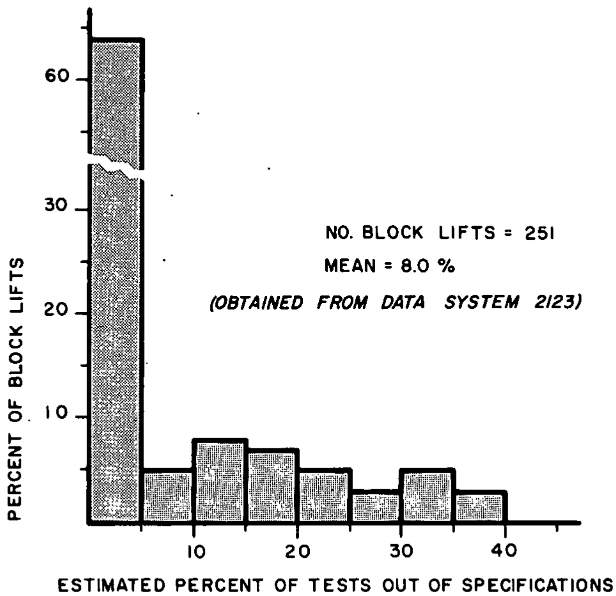


Figure 80. Crushed stone base compaction.

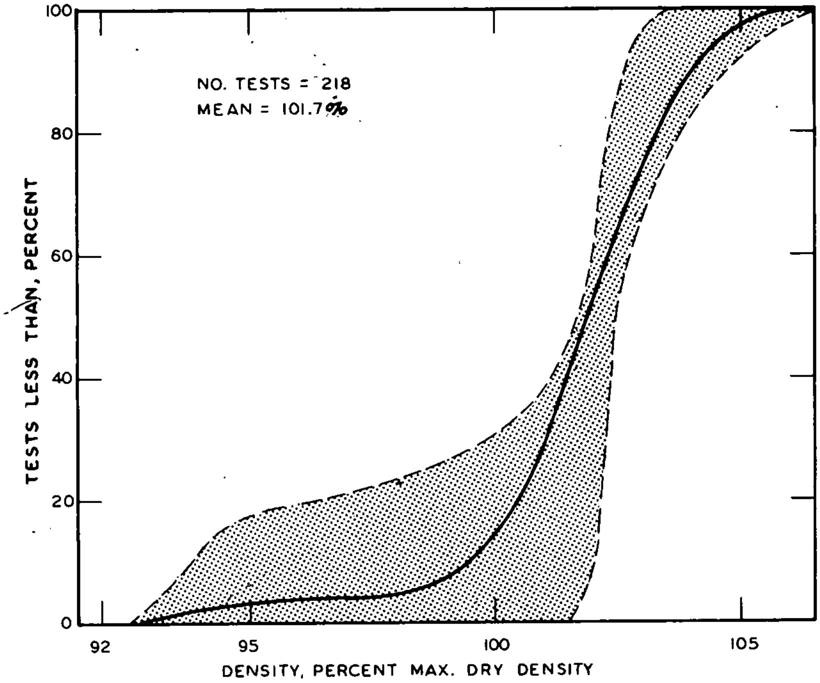


Figure 81. Crushed stone base density after subgrading.

mately 40 tests were made on each flexible tangent in the five traffic loops, and 11 were made on Loop 1. The results of these tests are summarized by loops in Table 33 and are shown graphically in Figure 81. The shaded area in the figure represents the area containing the curves for the individual loops.

The mean density after subgrading was 101.7 percent of maximum dry density as compared to 101.5 after compaction. The mean density after subgrading for Loop 3 was 100.3 percent while that for the other loops ranged from 101.3 to 102.6. Subgrading of the base course was started on Loop 3, and the resultant subgraded densities of the first few structural sections were relatively low. This accounts for the lower mean density on this loop and for the odd shape of the shaded area in the figure.

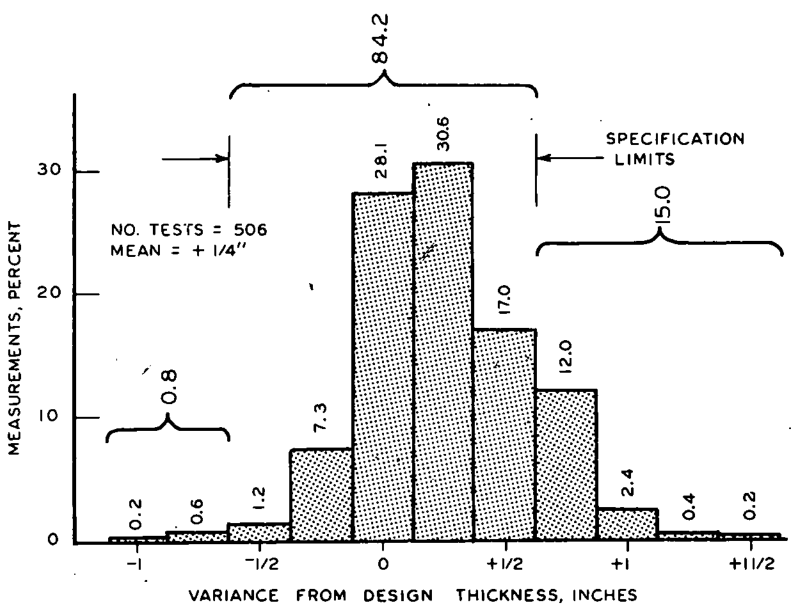


Figure 82. Crushed stone base thickness.

TABLE 34

SUMMARY OF THICKNESS MEASUREMENTS OF CRUSHED STONE BASE

Design Thickness (in.)	No. Measurements	Mean (in.)	Percent of Measurements: Within Tolerances	Below Tolerances	Above Tolerances
3	238	3 5/16	82.8	0.4	16.8
6	186	6¼	85.0	0.5	14.5
9	82	9¼	87.8	1.2	11.0

The thickness of the crushed stone base after subgrading was measured by digging small diameter holes to the surface of the subbase or earth embankment and measuring the depth of the holes. Two determinations were made at random locations in each test section. The results of these measurements are presented in Table 34 and Figure 82.

4.2 SPECIAL TYPES OF BASE

Each flexible pavement tangent of the four major loops contained six structural sections which had base type as a design variable. In these sections the base course was constructed as a "wedge" with the thickness decreasing in the direction of traffic. The experiment included four types of base: gravel, bituminous-treated, cement-treated, and the special gradation of crushed stone used as the base for the sections in the main factorial experiment.

4.2.1 Materials and Materials Control

The material specifications for the special base types were developed by subcommittees of the Advisory Panel on Materials and Construction.

The specifications, material characteristics and plant operations for the special gradation of crushed stone (Crushed Stone Base Course, Special) are discussed in detail in Section 4.1.1.

The gravel base (Gravel Base, Special) was a well-graded uncrushed gravel obtained by

TABLE 35

SPECIFICATION REQUIREMENTS FOR GRAVEL BASE, SPECIAL

(*a*) GRADATION [1]

Sieve	Gradation Limits (Percent Passing): Size E	Size F	Mixture	Gradation Formula Allowable Tolerances: (% passing)	(%)
1½ in.	100		100	100	
1 in.	90–100		90–100	95	±5
½ in.	30–60	100	65–95	72	±5
No. 4	0–10	90–100	40–65	48	±5
No. 16		45–85	20–45	32	±5
No. 40		20–50	10–30	20	±4
No. 100		10–35	5–17	12	±3
No. 200		5–20	3–10	8	±2

(*b*) DELETERIOUS SUBSTANCES

Substance	Maximum Percent by Weight: Size E	Size F	Determined by Test Designation
Soft and unsound fragments	5.0	5.0	ASTM C235–57T
Coal and lignite	1.0	1.0	—
Clay lumps	0.25	0.25	AASHO T112–56
Material finer than the No. 200 sieve	1.0	—	AASHO T11–49
Other deleterious or injurious material	1.0	1.0	—

(*c*) OTHER REQUIREMENTS

Characteristic	Requirement	Test Procedure
Plasticity index	2 to 6	AASHO Designation: T91–54
Liquid limit	Not more than 25	AASHO Designation: T89–54
Percentage of wear (Los Angeles machine)	Not to exceed 45	AASHO Designation: T96–56
Soundness (5 cycles of sodium sulfate)	Not to exceed 15	AASHO Designation: T104–57

[1] AASHO Designation: T27–46.

TABLE 36

SUMMARY OF PLANT GRADATION ON GRAVEL BASE, SPECIAL (SUMMARY OF 26 TESTS)
(Obtained from Data System 2121)

Sieve	Gradation Formula and Tolerances	Mean Percent of Material Passing	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
1½ in.	100	100	—	100	0.0	0.0
1 in.	95 ± 5	98.8	1.44	100	0.0	0.0
½ in.	72 ± 5	73.0	3.66	80.8	11.5	7.7
No. 4	48 ± 5	46.2	1.96	96.2	0.0	3.8
No. 16	32 ± 5	32.5	1.55	100	0.0	0.0
No. 40	20 ± 4	20.6	1.43	100	0.0	0.0
No. 100	12 ± 3	11.4	0.99	100	0.0	0.0
No. 200	8 ± 2	7.6	0.82	100	0.0	0.0

Figure 83. Batching aggregate for gravel base course, special.

combining a natural subangular to rounded washed material (Size E coarse aggregate) with a blend of natural sand and a plastic friable, fine-grained soil (Size F fine aggregate).

The specification controls for the gravel base material were similar to those for the special gradation of crushed stone. It was required that the two sizes of aggregate be combined and mixed on the job site to produce the specified gradation. Table 35 gives the material requirements for the Gravel Base, Special.

The Size F aggregate consisted of a combination of a coarse sand, a fine blend sand, and a plastic fine-grained soil. However, the soil and natural sands were transported to the job site and combined during the batching of the gravel base. The coarse aggregate, sands and soil were proportioned by weight and mixed with water in a paving mixer. Samples for gradation tests were obtained at the mixer. Table 36 is a summary of the test results.

The results of 24 tests on the gravel base mixture gave an average liquid limit of 18.9 and an average plasticity index of 3.1. The corresponding standard deviations were 2.1 and 1.9.

The Size E coarse aggregate and the coarse sand and soil for the Size F aggregate were obtained at Ottawa, Ill. The source of the blend sand was located at Essex, Ill., approximately 50 mi southeast of the job site. The materials were trucked to the batching plant on the job site. The coarse aggregate and the two sizes of sand were placed in a three-compartment batching bin by a crane and clam bucket. The soil was weighed separately on a small platform scale and deposited in the batch trucks by a small conveyor belt.

The four materials were proportioned and mixed in a dual-drum paving mixer in 5,000-lb batches containing 57 percent Size E aggregate, 27 percent coarse sand, 9 percent blend sand and 7 percent soil. The mixer was the same as that used for the crushed stone base material.

The bituminous-treated base was a mixture of the sand-gravel subbase and the 85-100 penetration grade paving asphalt used in the asphaltic concrete. The gradation and physical characteristics of the aggregate are covered in Section 3.1, and a discussion of the paving asphalt is included in Section 5.1.

The specifications required the asphalt content to be not less than 5 nor more than 6.5 percent of the weight of the mixture and permitted a 0.3 percentage point variation from the established content. Based on laboratory studies, the design asphalt content of the mix was 5.2 percent. The results of extraction and Marshall tests run on samples obtained from the trucks at the plant are summarized in Table 37.

The bituminous-treated base material was mixed in the automatic batch-type plant used to produce the asphaltic concrete binder course. The plant operations were the same as those for the binder course (Section 5.1) except that the screens in the gradation unit of the plant were removed. Both the aggregate and the asphalt were heated to approximately 290 F before being combined and mixed. The material was proportioned in 4,000-lb batches and 55 sec was required to introduce and mix the asphalt with the aggregate.

The cement-treated base was a mixture of the sand-gravel subbase and Type I portland cement obtained from the same source as the cement used in the surfacing of the rigid pavements.

The specifications required the cement and aggregate to be porportioned by weight, and set the limits on the cement at not less than 4 nor more than 5 percent of the dry weight of the material. A minimum compressive strength of 650 psi at the age of 7 days was specified.

Samples of the cement and aggregate were furnished to a few outside laboratories to develop a mix design. As a result, the actual cement content was set at 4 percent of the weight of the material.

The plant for the portland cement concrete was used to batch the cement treated base. The material was proportioned in 10,000-lb batches. The cement and aggregate were weighed separately and discharged into transit-mix trucks for mixing with water and delivery to the placement sites.

4.2.2 Construction Procedures

Except for the spreading operations, the construction of the crushed stone and gravel base wedge sections was the same as that described in Section 4.1.2 for the crushed stone base in the factorial sections. Spreading of the material was started at the end of the section where the total base thickness was the greatest. The thickness of each lower layer was either 3 or 4 in. when compacted, and the top layer was always 3 in. thick. The number of 4-in. layers was the minimum needed to obtain the required maximum total base thickness.

TABLE 37

SUMMARY OF EXTRACTION TESTS AND MARSHALL TESTS ON BITUMINOUS TREATED BASE COURSE
SUMMARY OF 14 EXTRACTIONS AND 26 MARSHALL TESTS
(Obtained from Data System 2146)

Item	Mixture Design	Extraction Tests[2]		Marshall Tests	
		Mean	Std. Dev.	Mean	Std. Dev.
Percent passing:					
1 in.	100	100	—		
½ in.	90 ± 5	91	1.9		
No. 4	73 ± 5	72	3.7		
No. 40	27 ± 5	27	2.3		
No. 200	7 ± 2	5.6	0.9		
Asphalt (%)[1]	5.2 ± 0.3	4.8	0.1		
Stability				1650	240
Flow				10	1.7
Bulk density				2.38	0.02
Voids (% total vol.)				6.2	0.8
Voids (% filled)				64.7	3.1

[1] Percent asphalt is by weight of total mix.

[2] Control tests have shown that the extraction test underestimated percent asphalt by 0.1 to 0.2 percentage points. The tests were made in a reflux type extractor in which the asphalt was removed by use of hot trichlorethylene solvent. The process was similar to that used by the highway departments of Washington and Pennsylvania.

Figure 84. Batching aggregate for cement-treated base course.

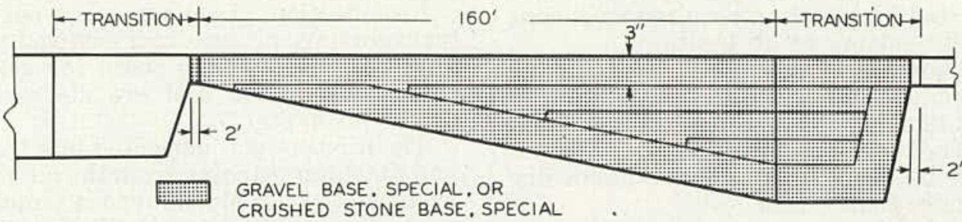


Figure 85. Base construction in wedge sections.

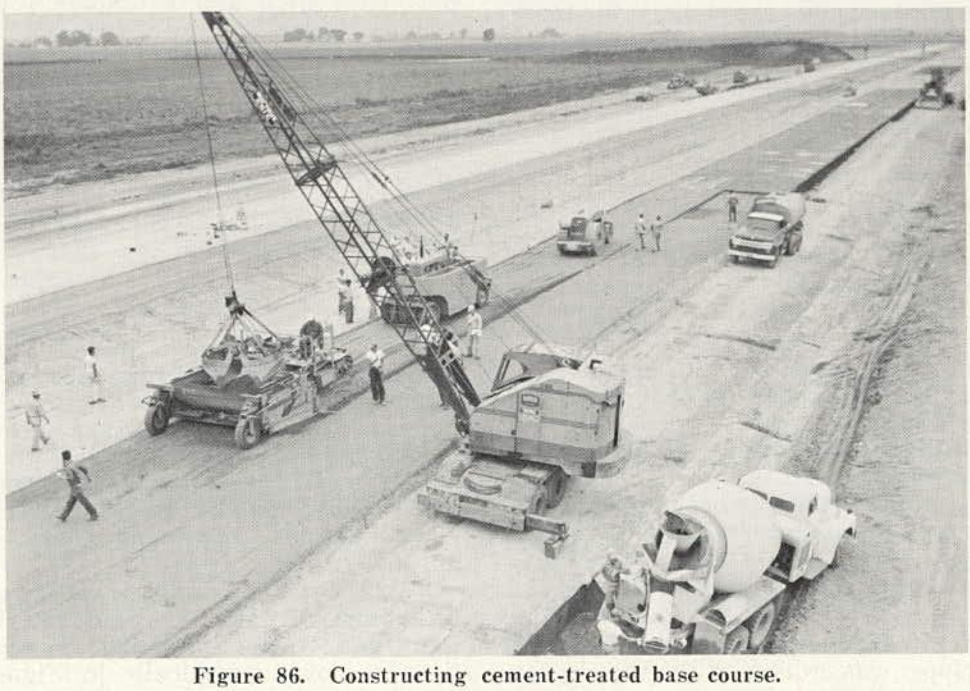


Figure 86. Constructing cement-treated base course.

Figure 85 shows the method of placement of the crushed stone and gravel base wedge sections. The initial layer was constructed parallel to the subgrade to a point where the top of the layer was within 3 in. of the elevation of the top of the finished base; the top of each succeeding layer was maintained parallel to the grade of the finished surface of the base; and the top layer extended throughout the entire length of the section.

The cement-treated base in each structural section was completed in one day, including trimming the surface and applying the bituminous curing coat. A curing period of at least 7 days was permitted before the asphaltic concrete surfacing was constructed. The shoulder portion of the base, composed of the Type A crushed stone, was not constructed until after curing of the cement-treated base was completed.

The cement-treated base was constructed in layers not less than 4 in. nor greater than 5 in. thick when compacted. The top surface of each layer was parallel to the grade of the surface of the completed base. The top layer extended throughout the entire length of the section and was constructed higher than required to permit subgrading by cutting at all locations.

In the compaction of the cement-treated base, the specifications permitted a variation in moisture content of ± 1 percentage point from optimum. The density of any layer was not permitted to be below the field maximum dry density by more than 5 pcf.

The specifications contained provisions aimed at preventing partial setting of the cement-treated base material before the construction was complete. The time between the addition of water to the mixture and spreading on the roadbed could not exceed 30 min, and not more than 2½ hr could elapse between the addition of water to the mixture and the completion of compaction.

A self-propelled mechanical spreader, identical to the one for spreading the crushed stone base course, was used for placement of the cement-aggregate mixture. The mixture was discharged from transit-mix trucks into a large metal box and transferred to the spreader by a crane and clam bucket operating on the outside shoulder.

Compaction was obtained with a self-propelled pneumatic-tired roller at 265 lb per in. of tire tread width and a 3-wheel roller at 275 lb per in. of drive-roll width. Usually, four passes of the pneumatic-tired roller followed by two coverages with the 3-wheel roller were sufficient to obtain the required density.

The surface of the cement treated base was trimmed to within plus or minus ⅛ in. of the established grade by a mechanical subgrading machine operated on steel forms. The subgraded surface was rolled with the ½-ton tandem roller used to roll the subgraded subbase.

Immediately after the forms were removed, a rapid curing liquid asphalt, RC-2, was applied to the surface and side slopes of the cement treated base at the rate of 0.2 gal per sq yd.

For the bituminous-treated base wedge sections, the specifications required the top layer to extend throughout the total length of the section and be uniformly 3 in. thick at all locations where the total thickness of base was 3 in. or greater. The thickness of lower layers was to be not less than 2 nor greater than 3 in. Only one layer of base was placed per day in any structural section. The layer construction for the bituminous-treated base was the same as that for the gravel and crushed stone base wedge sections (see Figure 85).

The surface of the center 24-ft portion of the completed base was checked for grade and cross-section while the top layer was still workable, and any variations in excess of ¼ in. were immediately corrected. The shoulder portions, with the Type A base material, were not constructed until the bituminous-treated base in a section was completed.

Specification requirements pertaining to transporting, placing and compacting the mixture were identical to those for asphaltic concrete construction and are discussed in more detail in Chapter 5.

The mixture was deposited into the spreading and finishing machine from the outside shoulder by means of a "Multipurpose Gradall." Guide wires were set to control the thickness of each layer.

Each layer of bituminous-treated base was compacted with a 3-wheel, a self-propelled pneumatic-tired and a tandem roller. Initial rolling of each layer was done with one pass of the 3-wheel roller at 134 pounds per inch-width of drive wheel. The pneumatic-tired roller, making numerous passes at 200 lb per in. of tire tread width, was used for intermediate rolling. Final rolling was done with the tandem roller at 120 lb per in. of roller width. Compaction to a minimum of 96 percent of maximum laboratory density was required.

4.2.3 Construction Control

The testing program for the base in the crushed-stone base wedge sections was the same as that for the crushed stone base in all other sections. The data are included in the summaries in Section 4.1.3.

The tests for construction control of the gravel base included sieve analyses, moisture determinations, maximum laboratory density, and field density.

Table 38 is a summary of 29 gradations run on samples of the gravel base material obtained immediately after spreading. The mean gradation is shown graphically in Figure 87. The

TABLE 38

SUMMARY OF GRADATION TESTS[1] OF GRAVEL BASE COURSE MATERIAL FROM ROAD (SUMMARY OF 29 TESTS)
(Obtained from Data System 2121)

Sieve	Gradation Formula and Tolerances	Mean Percent of Material Passing	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
1½ in.	100	100		100		
1 in.	95 ± 5	98.5	1.57	100		
½ in.	72 ± 5	74.3	4.32	79.4	10.3	10.3
No. 4	48 ± 5	48.9	3.41	86.2	6.9	6.9
No. 16	32 ± 5	35.0	2.66	77.3	22.7	0.0
No. 40	20 ± 4	22.8	2.32	82.8	17.2	0.0
No. 100	12 ± 3	12.8	1.14	93.1	6.9	0.0
No. 200	8 ± 2	9.1	0.84	89.7	10.3	0.0

[1] AASHO Designation: T27–46.

distribution of the results of compaction tests taken on each layer in the six gravel base wedge sections is shown in Figure 88.

Control tests for the construction of the cement-treated base included optimum moisture and maximum density determinations, field moisture content, field control density, field in-place density and 7-day compressive strengths.

The optimum moisture content and maximum dry density were determined by the method of ASTM D698-56T, Method C. Samples of the mixture obtained from the roadway just prior to compaction for determining the field control density were compacted in a Proctor mold at the existing moisture content of the material. This method permitted the percent compaction based on wet density to be calculated in the field. Samples for moisture determination and calculation of dry density were processed in the laboratory. The compacted specimens for determining field control density were also used to determine the 7-day compressive strength of the cement-treated base.

Table 39 is a summary of the test data for laboratory maximum density, field control density, field in-place density and 7-day compressive strength of the cement-treated base.

The field control density was determined for each layer of cement-treated base in each structural section. Acceptance of a layer was based on a comparison of the in-place densities with the field control density of the layer. The distribution of the in-place density tests, expressed as the difference between the field

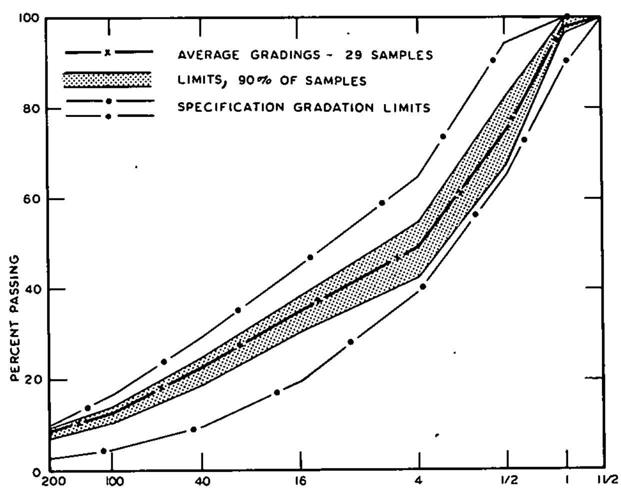


Figure 87. Gradation of in-place gravel base material.

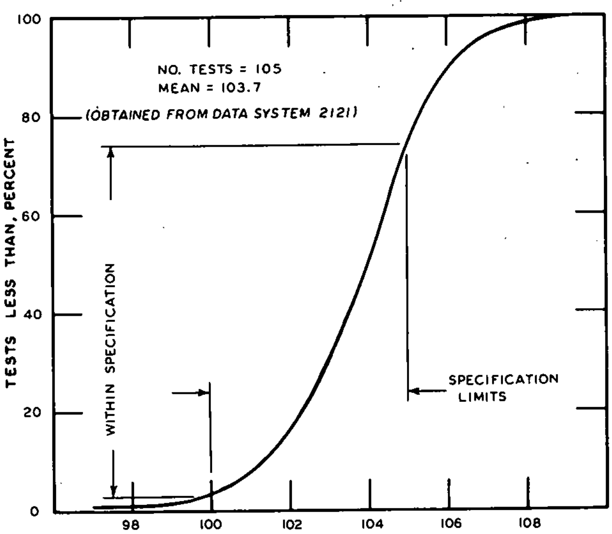


Figure 88. Gravel base, special—compaction.

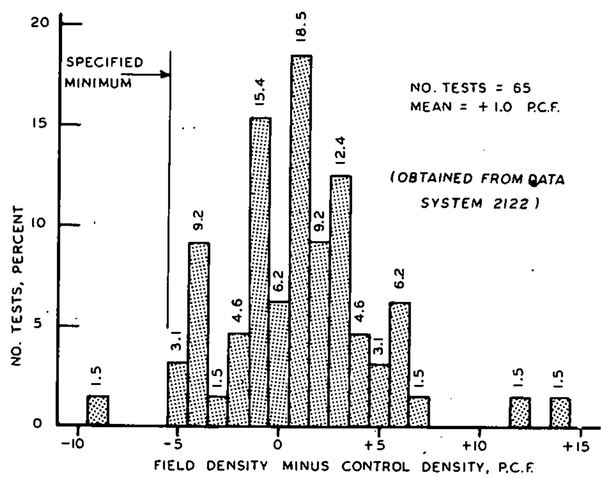


Figure 89. Field density, cement-treated base.

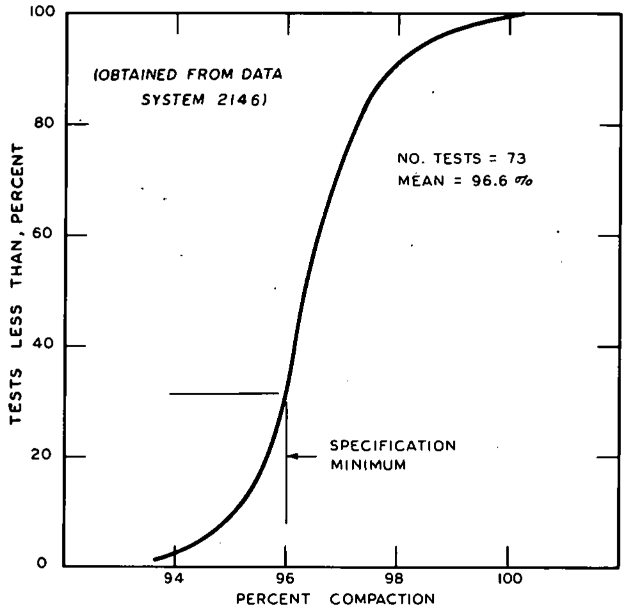


Figure 90. Bituminous-treated base course compaction.

TABLE 39

SUMMARY OF DENSITY AND COMPRESSIVE STRENGTH DATA FOR CEMENT-TREATED BASE
(Obtained from Data System 2122)

Item	No. Tests or Samples	Mean	Standard Deviation
Laboratory density[1]:			
Optimum moisture (%)	4	8.0	0.2
Maximum dry density (pcf)	4	137.7	1.4
Field control density[1]:			
Moisture content (%)	41	7.1	0.5
Dry density (pcf)	40	137.4	2.0
Field in-place density[2]:			
Moisture content (%)	65	6.8	0.6
Dry density (pcf)[3]	65	138.4	3.3
Compressive strength, 7-day (psi)[4]	35	840.0	122.0

[1] ASTM D698-56T, Method C.
[2] AASHO Designation: T147-54, except rubber balloon apparatus was used to measure volume of hole.
[3] Specifications required field in-place density to be not more than 5 pcf less than field control density.
[4] Specified minimum, 650 psi.

density and the control density, is shown by the bar graph in Figure 89.

The program for material and construction control of bituminous-treated base course included temperature checks, extractions, Marshall tests and field densities. Temperatures of the mix were taken in each truck at the plant and again prior to spreading. Temperature of the mixture just prior to initial rolling and prior to compaction with the pneumatic-tired roller also was recorded. Maximum laboratory density was determined daily by averaging the bulk density determinations of the Marshall test briquets. One sample taken at a random location was removed from each layer of base in each test section for field density determination. The samples were approximately 7 in. square and were quartered. The field density at each location was the average of individual tests on two of the four quarters.

The distribution of the results of the density tests on the compacted bituminous-treated base, expressed as percent of maximum laboratory density, is shown in Figure 90.

Chapter 5

Surfacing—Flexible Pavement

The surfacing on flexible pavement tangents was an asphaltic concrete consisting of a binder and surface course, except that twelve sections in Loop 2 had a bituminous surface treatment consisting of two cover coats and a seal coat.

This chapter discusses the materials and production of the asphaltic mixtures and describes the methods used to construct the surfacing on the flexible pavements. It includes summaries of data illustrating materials and construction control.

5.1 MATERIALS AND MATERIALS CONTROL

The asphaltic concrete binder and surface course mixtures consisted of two sizes of a crushed dolomitic limestone coarse aggregate, a natural siliceous coarse sand, a natural siliceous fine blend sand and a mineral filler (limestone dust) combined with an 85-100 penetration grade paving asphalt.

The specifications required that all coarse aggregates for both the binder and surface course mixtures be furnished from a single source and plant. The same requirement applied to the coarse sand, the fine blend sand, and the mineral filler.

The specification requirements for the aggregates are given in Table 40. The gradations of the fine aggregates are the gradation limits for the blends of the coarse and the fine sands.

The 85-100 penetration grade paving asphalt had to meet the requirements of AASHO Designation: M20-54. It was further specified that it be supplied from one source and one homogeneous lot of material. The producer was required to provide certification of the source and proportions of the crudes along with an analysis of the physical properties and chemical composition of the asphalt cement.

The contractor was furnished mix designs that were within the specified composition limits for the binder and surface course mixtures. The specifications included plus and minus tolerances for variations from these designs. The designs were developed with the gradations of the combined aggregates near the midpoints of the specified composition limits.

The mix designs were established by the Marshall method, using 50 blows per face, and conformed to the design criteria recommended by the Advisory Panel for Materials and Construction (Table 41).

During the pilot-studies and early construction on test tangents, minor revisions were made in the designs of the binder and surface course mixtures. The final mix formulas are given in Table 42, which also includes the specification composition limits and tolerances.

The 85-100 penetration grade asphalt cement was stored in two tanks at the refinery, and the inlets were sealed until the material was delivered to the job site. The producer certified the source and proportions of the crudes to be 50 percent West Texas and 50 percent Wyoming. The analysis included in the certification is given in Table 43.

Samples of the asphalt cement from the refinery tanks were tested by the Illinois Division of Highways for material acceptance. In addition, each truck load was sampled and tested for penetration at the job site prior to use. Check tests on the penetration of the asphalt cement also were made on samples obtained daily from the tanks and feeder lines of the asphalt plants.

Table 44 is a summary of the results of twelve tests on the samples from the two storage tanks at the refinery. Figure 91 shows graphically the results of the penetration tests on samples obtained from the trucks at the job site.

Tests on the aggregates for the asphaltic mixtures indicated a bulk saturated specific gravity of 2.81 for the coarse aggregates and 2.69 for the fine, and water absorptions of 2.1 and 1.9 percent, respectively. Soundness and abrasion tests on the coarse aggregates gave average losses of 11.6 and 23.7 percent, and corresponding ranges of 5.0 to 17.5 and 21.6 to 26.2 percent.

As required by the specifications, two separate hot-mix plants were used to produce the asphaltic mixtures. The binder course mixture was produced in an automatic batch-type and the surface course in a continuous-type plant. Temperatures were taken and extraction and Marshall tests were run at the plant. Two samples for Marshall tests were taken from one truck selected at random from each group

TABLE 40

AGGREGATE SPECIFICATION REQUIREMENT

(a) GRADATION [1]
(Percent Passing Sieve)

Sieve	Coarse Aggregate				Fine Aggregate		Mineral Filler
	Binder		Surface		Binder	Surface	
	Size G	Size H	Size J	Size K			
1 in.	100						
¾ in.	75–85	100	100				
½ in.	0–10	90–100	15–25	100			
⅜ in.		53–63	0–10	70–80			
No. 4		13–23		25–35	100	100	
No. 10		0–5		0–10	80–90	80–90	
No. 30					—	—	100
No. 40					45–50	40–50	—
No. 80					18–28	18–28	95–100
No. 200					0–6	0–5	65–100

(b) DELETERIOUS SUBSTANCES

Substance	Maximum Percent by Weight		Determined by Test Designation
	Coarse Aggregate [2]	Fine Aggregate [3]	
Soft and unsound particles, including chert ...	5.0	—	ASTM C235–57T
Coal and lignite ...	1.0	—	AASHO T113–57
Clay lumps ...	1.0	1.0	AASHO T112–56
Lightweight particles (shale, etc.) ...	2.5	—	AASHO T150–49
Elongated particles (5 times average thickness)	5.0	—	

(c) OTHER REQUIREMENTS

Characteristic	Requirement		Determined by Test Designation
	Coarse Aggregate	Fine Aggregate	
Percentage of wear (Los Angeles machine) ..	Not to exceed 35	—	AASHO T96–56
Soundness (5 cycles sodium sulfate) ...	Not to exceed 15	Not to exceed 10	AASHO T104–57

[1] AASHO Designation: T27–46
[2] Combined amount of first four items not to exceed 5 percent.
[3] Total amount of all substances not to exceed 3 percent.

of approximately eight trucks (100 tons of mix). Two samples for extraction tests were taken from one of each group of approximately 16 trucks (200 tons).

Marshall specimens were molded with a mechanical compactor calibrated to simulate the conventional hand compaction procedure. Two specimens were compacted from each sample, and the reported Marshall test result was the mean of the individual tests on the two specimens. The extraction test data were obtained by the hot method.

A summary of the results of Marshall and extraction tests is given in Tables 45 and 46. Figures 93 and 94 show the distribution of the results for asphalt content determined by the extraction tests.

Control tests run in the laboratory at the project site indicated that the asphalt content determined by extraction tests was 0.1 to 0.2 percentage points lower than the actual asphalt content. If a 0.2 percentage point correction were made to the distributions shown in Figures 93 and 94, the percent of tests within the specification limits would be increased from 63.8 to 96.8 for the binder course and from 82.3 to 94.8 for the surface course.

All materials for the binder and surface course mixtures, including asphalt cement, were delivered to the job site by trucks. The asphalt cement was obtained from a refinery at Whiting, Ind. The sources of the aggregates for the mixtures used on test tangents were within a 50-mi radius of the project site. The

TABLE 41
DESIGN CRITERIA

Characteristic	Binder Course	Surface Course
Stability	1,500–2,500	1,500–2,500
Flow	10–16	10–16
Total voids (%)[1]	4–6	3–5
Voids filled with asphalt (%)[1]	70–80	75–85

[1] Computations based on apparent specific gravity of aggregates.

TABLE 42
ASPHALTIC CONCRETE BINDER AND SURFACE COURSE MIXING FORMULAS[1]

Sieve	Binder Course		Surface Course	
	Composition Limits	Formula and Tolerances	Composition Limits	Formula and Tolerances
1 in.	100	100		
¾ in.	88–100	95 ± 5	100	100
½ in.	55–86	77 ± 5	86–100	90 ± 5
⅜ in.	45–72	56 ± 5	70–90	80 ± 5
No. 4	31–50	38 ± 5	45–70	64 ± 5
No. 10	19–35	26 ± 4	30–52	45 ± 4
No. 20	12–26	18 ± 4	22–40	31 ± 4
No. 40	7–20	11 ± 4	16–30	20 ± 4
No. 80	4–12	7 ± 3	9–19	11 ± 3
No. 200	0–6	3 ± 1	3–7	5 ± 1
Asphalt content	4–6.5	4.5 ± 0.3	4.5–7.5	5.4 ± 0.3

[1] Gradation based on percent of weight of total aggregate passing each size of sieve. Asphalt content based on total weight of mixture.

crushed limestone coarse aggregates were from Troy Grove, Ill., the regular coarse sand from Ottawa, Ill., the blend sand from Essex, Ill., and the mineral filler from Pontiac, Ill.

The aggregates were stored in separate stockpiles and protected against intermixture by wooden barriers. The surface of the storage area was covered with a bituminous stabilized gravel to prevent contamination of the aggregates.

The cold aggregate feeders of both asphalt plants consisted of twin bins of two compartments each, with one bin accommodating the two sizes of coarse aggregate and the other the two sizes of sand. An apron feeder was used for the sands and a reciprocating feeder for the coarse aggregates in the control of the cold aggregate feed of materials to the driers. Two revolving cylindrical driers, operating in tandem, were used for each plant.

On both plants the collected dust was conveyed to a 3-cu yd surge hopper and metered into the flow of aggregates at a uniform rate by a vane-type metering device. On the continuous plant the vane drive was interlocked with the other aggregate feeds and by-passed the gradation unit. On the batch-type plant the unit was hand-operated, and the dust was fed into the boot of the hot elevator and carried through the gradation unit into the fine bin.

The hot aggregates were separated into four sizes in the gradation unit. The amount of over-size and under-size material in any one hot bin could not exceed 5 and 10 percent, respectively (Table 47).

TABLE 43
CERTIFIED ANALYSIS OF THE ASPHALT CEMENT

Characteristic	Value
Softening point (°F)	117
Penetration at 77 F	88
Ductility at 77 F, (cm)	150+
Flash point, Cleveland (°F)	585
Solubility in CCl_4 (%)	99.8
Specific gravity, at 60 F	1.024
Loss on heating, 325 F, 5 hr (%)	0.012
Penetration of residue from loss on heating tests, at 77 F, 100 g, 5 sec, as compared to penetration before heating (%)	78
Spot test, standard naphtha solvent	Neg.
Viscosity, Saybolt furol, at 275 F (sec)	159
Chemical analysis:	
Carbon (%)	84.86
Hydrogen (%)	10.35
Oxygen (%)	2.89
Nitrogen (%)	0.57
Sulfur, Baumé (%)	3.89
C/H (atomic ratio)	0.683

TABLE 44
SUMMARY OF TEST RESULTS ON ASPHALT CEMENT
(Obtained from Data System 2145)

Characteristic	Specification Requirements AASHO Designation: M20–54	Mean Value
Penetration at 77 F, 100 g, 5 sec	85–100	91
Flash point, open cup, (°F)	450+	556
Ductility at 77 F, 5 cm per min (cm)	100+	150+
Loss on heating, 325 F, 5 hr (%)	1.0−	0.03
Penetration of residue from loss on heating test, at 77 F, 100 g, 5 sec, as compared to penetration before heating (%)	75+	84
Solubility in carbon tetrachloride (%)	99+	99.91[1]
Ash (%)	1.0−	0.05
Spot test, standard naphtha solvent	Neg.	Neg.

[1] Solubility in carbon disulfide.

TABLE 45

SUMMARY OF MARSHALL TEST RESULTS (SUMMARY OF 100 TESTS ON BINDER AND 72 ON SURFACE)
(Obtained from Data System 2140)

Characteristic	Mix Design Requirement	Mean Value	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
(a) BINDER COURSE MIXTURE						
Stability	1,500–2,500	1,770	190	92.1	0.0	7.9
Flow	10–16	11.2	0.64	98.0	0.0	2.0
Voids (% tot. vol.)	4–6	4.8	0.52	91.0	1.0	8.0
Voids (% filled)	70–80	68.2	2.70	18.2	0.0	81.8
(b) SURFACE COURSE MIXTURE						
Stability	1,500–2,500	2,000	125	100.0	0.0	0.0
Flow	10–16	11.1	1.22	83.3	0.0	16.7
Voids (% tot. vol.)	3–5	3.6	0.43	90.3	0.0	9.7
Voids (% filled)	75–85	77.9	2.21	91.7	0.0	8.3

TABLE 46

SUMMARY OF EXTRACTION TEST RESULTS (SUMMARY OF 127 TESTS ON BINDER AND 96 TESTS ON SURFACE)
(Obtained from Data System 2141)

Sieve	Mix Design	Mean Value	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
(a) BINDER COURSE MIXTURE						
1 in.	100	100	—	100	0.0	0.0
¾ in.	95 ± 5	96	2.21	99.2	0.0	0.8
½ in.	77 ± 5	76	3.30	96.1	3.9	0.0
⅜ in.	56 ± 5	57	2.71	85.8	13.4	0.8
No. 4	38 ± 5	36	2.18	94.5	0.0	5.5
No. 10	26 ± 4	25	1.29	99.2	0.0	0.8
No. 20	18 ± 4	19	1.03	98.4	1.6	0.0
No. 40	11 ± 4	13	0.98	96.9	3.1	0.0
No. 80	7 ± 3	8	0.81	97.6	2.4	0.0
No. 200	3 ± 1	4.3	0.49	74.0	26.0	0.0
Asphalt content [1]	4.5 ± 0.3	4.2	0.13	63.8	0.0	36.2
(b) SURFACE COURSE MIXTURE						
¾ in.	100	100	—	99.0	0.0	1.0
½ in.	90 ± 5	92	2.43	88.6	10.4	1.0
⅜ in.	80 ± 5	81	3.17	87.5	10.4	2.1
No. 4	64 ± 5	63	4.06	82.3	4.2	13.5
No. 10	45 ± 4	46	2.99	81.2	12.5	6.3
No. 20	31 ± 4	34	1.68	67.7	30.2	2.1
No. 40	20 ± 4	22	2.06	88.6	10.4	1.0
No. 80	11 ± 3	13	1.07	88.5	11.5	0.0
No. 200	5 ± 1	5.9	1.16	63.5	32.3	4.2
Asphalt content [1]	5.4 ± 0.3	5.2	0.18	82.3	0.0	17.7

[1] Percent asphalt by total weight of mix. Control tests have shown that the extraction tests underestimated percent asphalt by 0.1 to 0.2 percentage points. Extractions made in a reflux-type extractor in which the asphalt is removed by use of hot trichlorethylene solvent. The process was similar to that used by the highway departments of Washington and Pennsylvania.

Preparation of the mixtures in the two plants was primarily the same to the stage of material separation and deposit into the hot bins.

In the batch-type plant a weigh-hopper was located directly beneath the bins which contained five individual compartments simultaneously measuring all sizes of aggregate, including the mineral filler. The mineral filler was supplied to the weigh-hopper by its own feeding and measuring system. The twin pug-mill was located beneath the weigh-hopper. The aggregates were dry-mixed for 15 sec before the asphalt cement was introduced, and the total mixing time was 55 sec. The material was discharged in 4,000-lb batches into insulated trucks.

In the continuous-type plant, two apron feeders were located beneath the hot bins. Each feeder drew from two bins and had two calibrated feeder gates synchronized to deliver the correct proportion of aggregate mixture to the pug-mill. The mineral filler was measured and added at this stage through a screw conveyor. The reclaimed dust collected from the dryers also was added at this point, and the material was fed into the twin-shaft pug-mill by an elevator system simultaneously with the asphalt cement. The material was mixed in the pug-mill for 85 sec and discharged into the trucks.

The temperature differential between the hot aggregates and the asphalt cement was not permitted to be greater than 25 F, and the temperature of the mixture was not to vary more than 20 F from a set temperature within the 250 to 325 F specification limits.

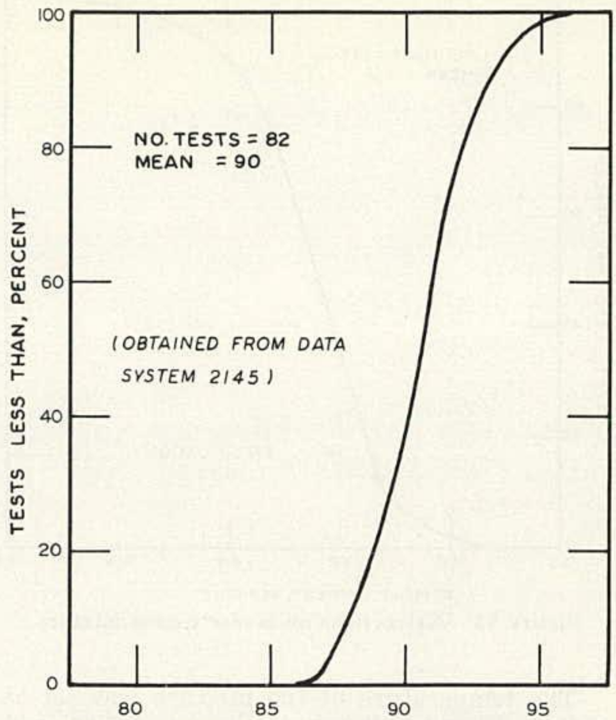


Figure 91. Penetration at 77 F of asphalt cement.

Figure 92. Automatic batch-type plant used to produce binder course mixture; dryers in tandem.

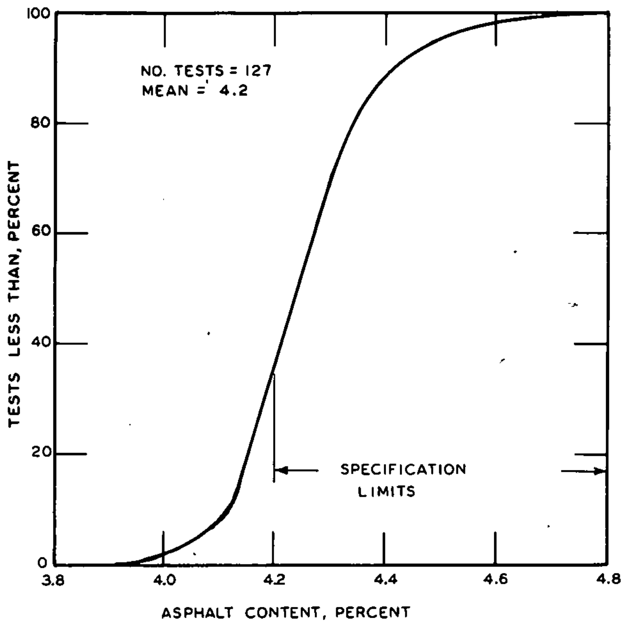


Figure 93. Extractions on binder course mixture.

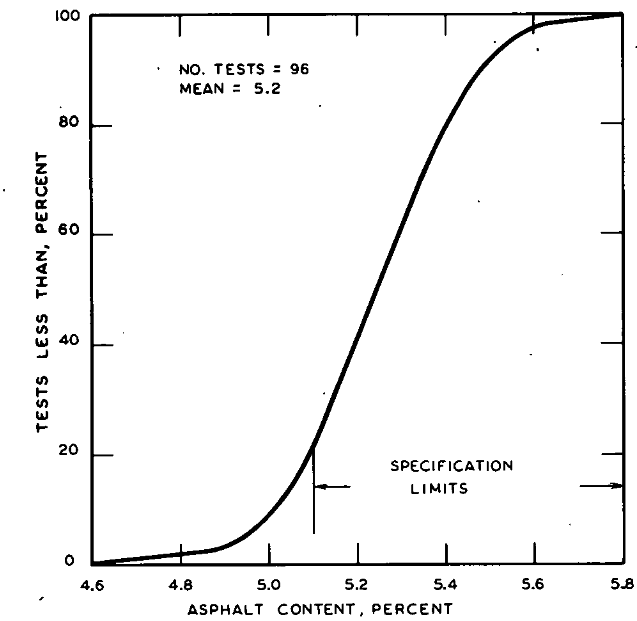


Figure 94. Extractions on surface course mixture.

The temperature of the mixture was set at 290 F. The asphalt cement also was maintained at a temperature around 290 F. The first dryer was operated at 250 F and the second at 325 F, which produced sand bin temperatures in the range of 310 to 320 F.

The two plants were calibrated to produce material at a rate of 100 tons per hour. Analysis of combined gradations of hot-bin samples was the principal control of the plant operation. Stockpile gradations and cold-feed analyses also were made to assist in plant control.

The gradation of material in each of the four hot bins was determined for each hour of production from a composite sample of at least three separate batches. These four gradations were combined mathematically to produce the theoretical gradation of the mixture. A summary of the hot-bin analyses is given in Table 48.

Stockpile gradations were determined from two composite samples representing each 3 ft of stockpile placement for each size aggregate. Additional stockpile gradations were determined from samples taken at random time intervals. Specific gravity determinations were made as routine checks of tests previously made on samples taken at the material sources. Gradation analyses were made for periodic adjustment of cold-feed gates to minimize hot-bin "overrun" of aggregates.

The asphalt plants were located at the west end of the project. The batch plant was set up in the fall of 1957, and installation of the continuous plant was completed in the spring of 1958. Except for 28 tons of binder mixture, all asphaltic concrete was produced in 1958 between May 13 and October 4.

A bituminous surface treatment, consisting of a double cover and a seal coat, was placed on twelve structural sections of Loop 2. The

TABLE 47

HOT-BIN AGGREGATE SIZES

Bin No.	Binder Course	Surface Course
1	Passing No. 10 sieve, 95 (+)	Passing No. 10 sieve, 95 (+)
2	Passing ⅜-in. sieve, 95 (+) Passing No. 10 sieve, 10 (−)	Passing No. 4 sieve, 95 (+) Passing No. 10 sieve, 10 (−)
3	Passing ⅝-in. sieve, 95 (+) Passing ⅜-in. sieve, 10 (−)	Passing ⅜-in. sieve, 95 (+) Passing No. 4 sieve, 10 (−)
4	Passing 1-in. sieve, 95 (+) Passing ⅝-in. sieve, 10 (−)	Passing ¾-in. sieve, 95 (+) Passing ⅜-in. sieve, 10 (−)

TABLE 48

SUMMARY OF HOT-BIN ANALYSES (SUMMARY OF 173 TESTS ON BINDER AND 130 TESTS ON SURFACE)
(Obtained from Data System 2143)

Sieve	Mix Design	Mean Value	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
(a) BINDER COURSE FOR TEST TANGENTS						
1 in.	100	100	—	100	—	—
¾ in.	95 ± 5	95.8	0.91	100	—	—
½ in.	77 ± 5	75.2	2.36	93.1	0.0	6.9
⅜ in.	56 ± 5	55.9	1.90	100	—	—
No. 4	38 ± 5	35.9	1.52	97.1	0.0	2.9
No. 10	26 ± 4	24.4	0.89	99.4	0.0	0.6
No. 20	18 ± 4	19.6	1.39	99.4	0.6	0.0
No. 40	11 ± 4	13.2	1.45	96.5	3.5	0.0
No. 80	7 ± 3	7.9	1.12	98.8	1.2	0.0
No. 200	3 ± 1	3.6	0.48	97.7	2.3	0.0
(b) SURFACE COURSE FOR TEST TANGENTS						
1 in.	100	100	—	100	—	—
½ in.	90 ± 5	91.5	1.07	100	—	—
⅜ in.	80 ± 5	80.7	0.86	100	—	—
No. 4	64 ± 5	63.6	1.91	97.7	0.0	2.3
No. 10	45 ± 4	45.7	2.11	98.5	0.0	1.5
No. 20	31 ± 4	33.3	2.29	89.2	10.0	0.8
No. 40	20 ± 4	19.8	2.19	94.6	4.6	0.8
No. 80	11 ± 3	10.7	0.92	99.2	0.8	0.0
No. 200	5 ± 1	4.8	0.30	100	—	—

crushed stone aggregates were obtained from Troy Grove, Ill., the same source as that of the coarse aggregates for the asphaltic concrete. The bituminous material, obtained from a refinery at Whiting, Ind., was a medium curing cutback liquid asphalt, Grade MC-5, meeting the requirements of AASHO Designation: M82-42. The specification requirements and test results for the cover coat and seal coat aggregates and for the asphalt are given in Tables 49 and 50.

TABLE 49

AGGREGATES FOR BITUMINOUS SURFACE TREATMENT
(SUMMARY OF 5 TESTS)

Sieve	Cover Coat Aggregate		Seal Coat Aggregate	
	Specification Limits	Mean Value	Specification Limits	Mean Value
⅝ in.	100	100		
½ in.	80–100	90		
⅜ in.	25–65	37	100	100
No. 4	0–5	0.6	20–45	32
No. 10	—	—	0–5	0.5
No. 200	0–1.5	0.2	0–1.5	0.4

The bituminous material for the prime coat was a medium curing cutback liquid asphalt, Grade MC-1, meeting the AASHO M82-42 specifications. It also was obtained from Whiting, Ind. The specification requirements and test results of this material are given in Table 51.

TABLE 50

BITUMINOUS MATERIAL, GRADE MC-5, FOR BITUMINOUS SURFACE TREATMENT[1] (SUMMARY OF 2 TESTS)

Characteristic	Spec. Limits	Mean Value
Flash point, Tag open cup (°F)	150+	257[2]
Viscosity, Saybolt furol, 180 F (sec)	300–600	387
Distillate (% by vol. of tot. dist. to 600 F)	20–75	35.5
Residue from distillation test to 680 F (% vol. by diff.)	82+	93.3
Tests on residue from distillation:		
Penetration, 77 F, 100g, 5 sec	120–300	240
Ductility at 77 F (cm)	100+	147
Solubility in carbon tetrachloride	99.5	99.9[3]
Spot test, standard naphtha solvent	Neg.	Neg.

[1] AASHO Designation: M82–42.
[2] Determined by Cleveland open cup.
[3] Solubility in carbon disulfide.

TABLE 51

BITUMINOUS MATERIAL, GRADE MC–1, FOR PRIME COAT[1]
(SUMMARY OF 9 TESTS)

Characteristics	Spec. Limits	Mean Value
Flash point, tag open cup (°F)	100+	160+
Viscosity, Saybolt furol, 122 F (sec)	75–150	130
Distillate, by vol. of tot. dist. to 680 F:		
to 437 F	20–	7.4
to 500 F	25–65	36.1
to 600 F	70–90	73.8
Residue from distillation to 680 F (% vol. by diff.)	60+	71.5
Tests on residue from distillation:		
Penetration 77 F, 100 g, 5 sec	120–300	268
Ductility at 60 F (cm)	100+	140
Solubility in carbon tetrachloride	99.5	99.9[2]
Spot test	Neg.	Neg.

[1] AASHO Designation: M82–42.
[2] Solubility in carbon disulfide.

Two other binder and surface course mixtures, in addition to the standard mixtures for test tangents, were used in sections of the east turnarounds of the five traffic loops. The stability of these mixtures was varied from that used on test tangents by changing the asphalt content and the coarse component of the fine aggregate. A rounded river sand was used for the coarse sand in the low stability mix and a processed stone sand in the high stability mix. To obtain a rounded river sand within the specification limits, it was necessary to blend a Mississippi River sand from Moline, Ill., (approximately 100 mi west of the project) with 13 percent of a fine sand obtained from Ottawa. The stone sand was obtained from Troy Grove, Ill. Data concerning these two mixes are presented in Table 52.

5.2 CONSTRUCTION PROCEDURES

The construction specifications for surfacing on flexible tangents prohibited all except spreading and compacting equipment from operating on the center 24-ft portion of the roadbed and permitted equipment to cross the roadbed only at transition areas between construction blocks. This made it necessary to develop a means of depositing the material into the paving machine from the outside shoulders, and to alter the pressure distributor to permit priming from the same area.

5.2.1 Asphaltic Concrete Construction

The layers of binder and surface course were placed with a bituminous paving machine. Guide wires were used to control the elevation of the screed. Compaction was accomplished with 3-wheel, pneumatic-tired, and tandem rollers.

TABLE 52

SPECIAL STABILITY MIXES ON TURNAROUNDS
(Obtained from Data System 2147)

Characteristic	Low Stability Mix[1]				High Stability Mix[2]			
	Binder Course		Surface Course		Binder Course		Surface Course	
	Mix Design	Mean	Mix Design	Mean	Mix Design	Mean	Mix Design	Mean
Stability	1500–2500	1430	1500–2500	1395	1500–2500	1935	1500–2500	2180
Flow	10–16	11.0	10–16	9.5	10–16	12.7	10–16	12.4
Voids (% tot. vol.)	4–6	5.9	3–5	4.3	4–6	4.8	3–5	2.5
Voids (% filled)	70–80	61.4	75–85	71.5	70–80	68.8	75–85	83.7
Gradation (% passing):								
1 in.	100	100	—	—	100	100	—	—
¾ in.	95 ± 5	95	100	100	95 ± 5	95	100	100
½ in.	77 ± 5	77	93 ± 5	92	77 ± 5	77	93 ±5	93
⅜ in.	56 ± 5	56	80 ± 5	79	56 ± 5	57	80 ± 5	80
No. 4	38 ± 5	34	57.5 ± 5	56	38 ± 5	35	57.5 ± 5	57
No. 10	26 ± 4	23	41 ± 4	41	26 ± 4	25	41 ± 4	40
No. 20	18 ± 4	19	31 ± 4	34	18 ± 4	19	31 ± 4	31
No. 40	11 ± 4	14	23 ± 4	25	11 ± 4	16	23 ± 4	26
No. 80	7 ± 3	8	14 ± 3	14	7 ± 3	11	14 ± 3	17
No. 200	3 ± 1	3.4	5 ± 1	3.4	3 ± 1	4.8	5 ± 1	5.1
Asphalt content (%)	4.2 ± 0.3	3.9	4.9 ± 0.3	4.7	4.7 ± 0.3	4.4	5.9 ± 0.3	5.4

[1] Rounded river sand. [2] Stone sand.

Figure 95. Depositing mixture in paver from outside shoulder.

The temperature of the mix at the time of placement was required to be within 20 deg of that set at the plant (290 F). The compacted density could not be less than 96 percent of the maximum laboratory density. The specifications also required that the thickness of the completed mat be within ¼ in. of the design thickness, and that the surface smoothness be within ⅛ in. in 10 ft.

The pilot studies included constructing the asphaltic concrete directly on base, subbase and embankment soil. The mixture was transferred from trucks on the outside shoulder to the paving machine with a "Multipurpose Gradall." The Gradall was similar to a truck crane except that it had a telescoping boom with bucket attached.

During the pilot studies, alterations were made on the paving machine. A plate was added to confine the material in the hopper, the tracks were extended to provide additional bearing area, and a sprinkling system was provided to keep the metal tracks wet.

A conventional pressure distributor was modified to permit priming a 25 ft width in the center of the roadbed from the outside shoulder. The spray bar was offset to the side of the truck and could be raised or lowered and moved in or out by hydraulic controls on the platform of the distributor.

The study of rolling weights and temperatures determined that three sets of rollers were needed to compact the asphaltic mixtures on the various thicknesses of subbase and base. The roller weights and rolling temperatures are given in Table 53. The thicknesses of subbase plus base were used only as a guide in selecting the set of rollers for each structural section. When there was an indication that a set of rollers was damaging the surfacing, they were replaced with the next lighter set or the number of passes was reduced.

Usually, the required density was obtained with one pass of the 3-wheel roller followed by eight passes of the pneumatic-tired roller. The tandem roller was used only to remove the marks left by the 3-wheel and pneumatic-tired rollers.

A steel bristle broom drag was attached to the paver during placement of the surface course to obliterate any tearing caused by the screed.

A tack coat of emulsified asphalt (SS-1) was used to facilitate bonding of the surface course to the binder. It was diluted with water and applied at a rate of 0.15 gal per sq yd. This resulted in a residue of about 0.02 gal of asphalt per square yard.

TABLE 53

ROLLING WEIGHTS AND TEMPERATURES

Roller Set	Roller Weight (lb per in.-width)			Section[1] Thickness (in.)
	Three-Wheel	Pneu-matic-Tired[2]	Tan-dem	
Heavy	300	300	250	15+ (also all 9″ base sections)
Intermediate	214	250	190	8 to 15
Light	180	200	120	8 or less
MAT ROLLING TEMPERATURE (°F)				
	250-275	190-220	——[3]	

[1] Subbase plus base.
[2] Based on 9-in. tire tread, inflation pressure was 75 psi.
[3] While mat was still workable but had cooled sufficiently to prevent shoving.

Figure 96. Constructing initial layer of binder course; second machine in position to pave other lane.

Construction on test tangents was always started at the west end of a loop. In general, the binder course was completed for the entire tangent length before placing the surface course. Construction within a tangent was started with the initial layer of binder course on those sections having the greatest total thickness of surfacing. Construction proceeded to the thinner surfacing sections so that each succeeding layer of binder course would extend through as many sections as possible and so that the surface course would extend throughout the entire length of the tangent.

Priming on test tangents also was started with the structural sections having the greatest thickness of surfacing. It was performed so that each section was covered within a period of 24 to 72 hr after priming. The prime coat (MC-1) was applied at a rate of about 0.2 gal per sq yd.

Figure 97. Paving shoulders on flexible tangent.

The surfacing was constructed in lane widths, using two paving machines. While construction operations were being performed in one lane, the other machine was being positioned in the opposite lane so that the crew could move back and immediately start spreading in that lane. Sufficient material was kept on hand to insure a continuous spreading operation throughout a test section. No delays in spreading were allowed except in transition areas. An additional 3/16 in. per in. was added to the spreading thickness to allow for compaction.

The paving machine always entered at transitions betwen sections. Plates of steel or wood were placed over the primed surface in the transition to protect it from damage by the turning movements of the machine. As previously mentioned, the initial layer of binder course was placed on the structural sections in a tangent having the greatest thickness of surfacing. This required bypassing sections with thinner surfacing thickness. In such cases, the machine was either moved over the primed surface of the intermediate sections on 2-ft

TABLE 54

RECORD OF ASPHALTIC CONCRETE CONSTRUCTION

Construction Block No.	Const. Sequence [1]	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6
F–1	1	9-15-58	9- 8-58	7- 2-58	8- 6-58	8-22/23-58	7-24/25-58
	2	9-20-58	9-10-58	7-7/8-58	8-25/26-58	8-30-58	7-29-58
	3	9–22/23-58	9-12-58	7-9/10-58	8-26/27-58	9-2/3-58	8-12/20-58
	4	9-24-58	9-15-58	8-22/23-58	8-29-58	9- 4-58	8-23-58
F–2	1	9-15/19-58	9-8/9-58	7-2/7-58	8-6/7-58	8-23/25-58	7-25-58
	2	9-20-58	9-10-58	7-7/8-58	8-25/26-58	8-30-58	7-29, 8-11-58
	3	9–22/23-58	9-12-58	7-9/10-58	8-26/27-58	9-2/3-58	8-12/20-58
	4	9-24-58	9-15-58	8-22-58	8-29-58	9- 4-58	8-23-58
F–3	1	9-19/20-58	9-9/10-58	7- 7-58	8- 7-58	8-28-58	7-25/28-58
	2	9-20/22-58	9-10-58	7- 7-58	8-25/26-58	8-30-58	8-11-58
	3	9-23-58	9-12-58	7-10-58	8-26/27-58	9- 3-58	8-12/20-58
	4	9-24-58	9-15-58	8-22-58	8-29-58	9- 4-58	8-23/25-58
F–4	1		9-10-58	7- 8-58	8-7/8-58	8-20-58	7-28-58
	2		9-10-58	7-8/9-58	8-25/26-58	8-30-58	8-11-58
	3		9-12-58	7-9/10-58	8-26/28-58	9- 3-58	8-12/20-58
	4		9-15-58	8-22-58	8-29-58	9- 4-58	8-23/25-58
F–5	1		9-10-58	7- 8-58	8-8/11-58	8-28/29-58	7-28/29-58
	2		9-12-58	7-8/9-58	8-25/26-58	8-30/9-2-58	8-11-58
	3		—	7-9/10-58	8-27/28-58	9-2/6-58	8-12/21-58
	4		9-20-58	7-23, 8-22-58	8-29-58	9-4/9-58	8-25-58
F–6	1		9-10/11-58	7-19/21-58	8-11/13-58	8-29/30-58	7-29-58
	2		9-12-58	7-24, 8-4-58	8-25-58	9- 2-58	8-13-58
	3		—	7-26/29-58	8-26/28-58	9-3/6-58	8-16/21-58
	4		9-20-58	8-7/8-58	8-29-58	9- 9-58	8-25-58
F–7	1		9-11-58	7-21/23-58	8-13-58	8-30/9-2-58	7-29-58
	2		9-12-58	7-25, 8-4-58	8-25/26-58	9- 4-58	8-13-58
	3		—	7-26-58	8-27/28-58	9-6/8-58	8-16/21-58
	4		9-20-58	8-7/8-58	8-29/30-58	9- 9-58	8-25-58
F–8	1			7-23-58	8-14-58	9-2/3-58	7-29/8-4-58
	2			7-25, 8-4-58	8-25/26-58	9- 4-58	8-13-58
	3			7-26/29-58	8-27/28-58	9-6/8-58	8-14/22-58
	4			8- 8-58	8-30-58	9- 9-58	8-25/26-58
F–9	1			7-24-58	8-14-58	9- 3-58	8- 4-58
	2			7-25, 8-4-58	8-26-58	9- 4-58	8-13-58
	3			8- 4-58	8-28-58	9-6/8-58	8-14/22-58
	4			8-11-58	8-30-58	9- 9-58	8-25/26-58
F–10	1			7-24-58	8-14/18-58	9- 3-58	8-4/5-58
	2			7-25-58	8-26-58	9- 4-58	8-13-58
	3			8- 4-58	8-28-58	9-6/8-58	8-14/22-58
	4			8-11-58	8-30-58	9- 9-58	8-25/26-58

[1] Construction sequence 1 is subgrading for surfacing, sequence 2 is application of prime coat, sequence 3 is binder course construction, sequence 4 is surface course construction.

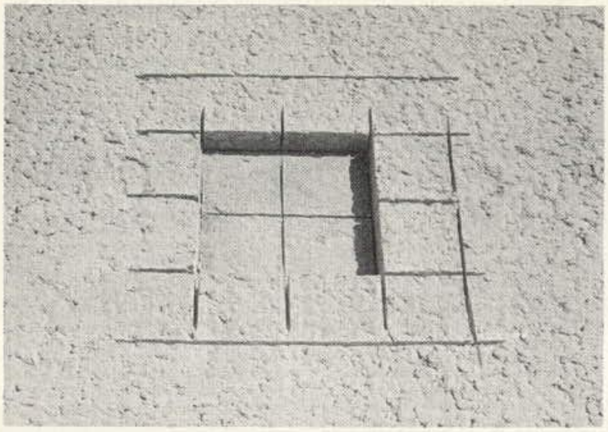


Figure 98. Sampled location, binder course.

wide planks of ¾-in. plywood, or was removed at the transition and moved down the inside shoulder.

Compaction of the mixture on any one tangent always required at least two of the sets of rollers listed in Table 53. The inside shoulder of a tangent was used for moving the sets of rollers from one section to another and for storage of the sets not in use.

Table 54 is a summary of construction by blocks of the asphaltic concrete surfacing. The table also includes the dates of subgrading and applying the prime coat.

5.2.2 *Bituminous Surface Treatment Construction*

The construction specifications for the bituminous surface treatment required that the bituminous material be applied at a rate of 0.20 to 0.25 gal per sq yd for the first cover coat, and at 0.25 to 0.30 for the second cover and the

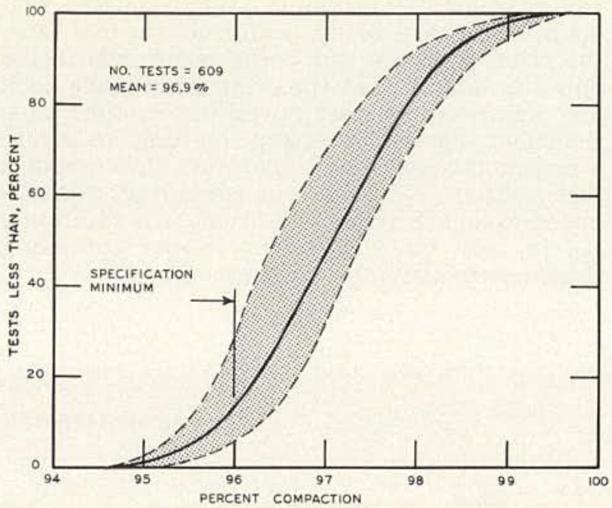


Figure 99. Binder course compaction.

seal coat. The aggregates were applied at a rate of 15 to 25 lb per sq yd for each cover coat and the seal coat.

The bituminous material was applied from the outside shoulder with the modified pressure distributor. The aggregates were spread with a standard chip spreader. The load in the truck was limited to an amount sufficient to spread the length of a structural section. The aggregates were rolled with three passes of a pneumatic-tired roller at 200 lb per in. of tire tread width followed by three passes of a tandem roller at 120 lb per in. of roller width. At the locations where the surface treatment was placed directly on the subbase, the aggregates were spread by hand and were rolled with a ½-ton tandem roller.

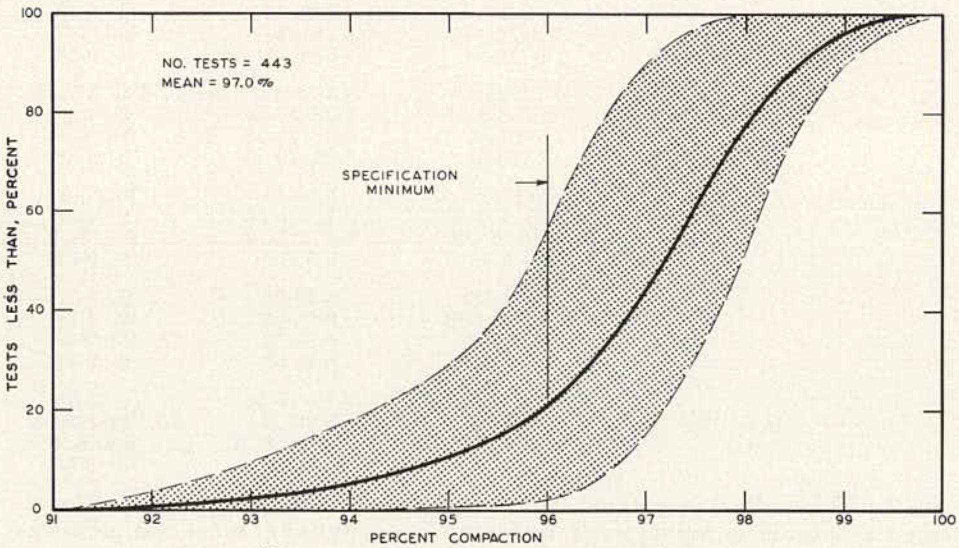


Figure 100. Surface course compaction.

Figure 101. Device on paver for following guide wire.

The surface treatment was applied to the twelve structural sections on Loop 2 during the first week of October 1958.

5.3 CONSTRUCTION CONTROL

The temperature of each truck load of mixture was recorded at the plant and on the grade immediately before spreading. Temperatures were also recorded prior to breakdown with the 3-wheel roller and prior to compaction with the pneumatic-tired roller. Each truck load of mixture placed in a test tangent had its location recorded by stations and by lift number.

Maximum laboratory density was determined from Marshall specimens. Two samples were taken from separate batches in a single truck for each 100 tons of material, and two specimens were made from each sample. The densities of the specimens were averaged to determine the maximum laboratory density for each day of production.

One sample was taken for field density determination from each layer of asphaltic mixture of each test section. The samples were approximately 7 in. square and were quartered. The field density of the layer was recorded as the average of the individual tests on two of the four quarters of each sample.

All samples for field density were taken from the center of the lane, and samples from the surface course were taken only in the transition areas.

Compaction control data of the asphaltic concrete are summarized in Table 55 and presented graphically in Figures 99 and 100. The shaded area in each of the figures represents the limits of the distribution curves for the individual loops.

It was necessary to reduce the amount of compactive effort to prevent cracking the surfacing on sections having thin combinations of subbase and base. This was especially true of the surface course mix and accounts for the lower surface course densities on Loops 1 and 2.

The guide wires for controlling thickness of the surfacing were set along each edge of the pavement 2 in. above the theoretical grade and adjusted to conform to the primed surface. During the placement of each layer, the thickness along the edge of the pavement was controlled from the wire. The thickness at the center of the pavement was controlled by measuring the loose thickness of the layer. In addition, the required loose thickness of each succeeding layer at the pavement centerline was painted on the surface of the previous course at 25-ft intervals in each section. These thicknesses were determined by measuring the distance from a string stretched across the guide wires to the surface of the previous layer.

Compensation for settlement of the primed surface was made by adjusting the wires after the first layer of asphaltic mixture was placed and compacted. Two-inch square plates were set on the primed surface at 1, 6, and 11 ft on each side of the centerline at 25-ft intervals along each section. The amount of adjustment was determined by measuring the distance from the string to the plates before placing any material and again after the first layer was placed and compacted.

TABLE 55

ASPHALTIC CONCRETE COMPACTION DATA
(Obtained from Data System 2142)

Loop	Laboratory Data: Max. Lab. Density (pcf)	Laboratory Data: Voids (%): Tot. Val.	Laboratory Data: Voids (%): Filled	Field Data: No. Tests	Field Data: Density (pcf): Mean	Field Data: Density (pcf): Std. Dev.	Field Data: Voids (% tot. vol.): Mean	Field Data: Voids (% tot. vol.): Std. Dev.	Field Data: Voids (% filled): Mean	Field Data: Voids (% filled): Std. Dev.	Field Data: % Max. Lab. Den.: Mean	Field Data: % Max. Lab. Den.: Std. Dev.	Field Data: Tests Within Spec.[1] (%)
(a) BINDER COURSE													
1	152.7	5.35	65.3	64	148.7	1.45	7.84	0.87	55.8	3.16	97.3	0.98	92.2
2	153.9	4.63	68.7	12	148.6	0.67	7.92	0.42	55.3	1.43	96.5	0.41	100
3	153.6	4.69	68.4	86	148.5	1.52	7.90	0.94	55.6	3.48	96.6	0.97	74.1
4	153.2	4.94	67.5	128	149.5	1.49	7.31	0.87	58.0	3.35	97.5	0.84	97.7
5	153.4	4.95	67.0	128	148.5	1.38	8.02	0.88	54.9	3.24	96.8	0.89	85.2
6	153.6	4.77	68.0	192	149.3	1.77	7.46	1.09	57.2	3.84	97.2	1.07	88.0
All	153.4	4.87	67.6	609	149.0	1.60	7.66	0.99	56.5	3.49	96.9	1.00	88.2
(b) SURFACE COURSE													
1	151.2	3.64	77.6	64	144.4	2.52	8.00	1.61	60.4	4.89	95.5	1.68	50.0
2	150.8	4.07	75.0	44	145.3	2.41	7.58	1.52	61.2	4.88	96.3	1.45	68.2
3	151.8	3.56	77.7	84	147.4	2.33	6.32	1.43	66.1	5.36	97.2	1.57	81.0
4	152.1	2.93	81.5	84	147.8	1.70	5.68	1.05	69.0	4.15	97.2	1.06	91.7
5	151.3	3.67	77.2	84	146.7	1.92	6.60	1.09	64.9	3.87	97.0	1.16	83.3
6	151.3	3.81	76.3	84	148.2	1.31	5.76	0.82	67.7	3.18	98.0	0.85	98.8
All	151.5	3.57	77.8	443	146.8	2.40	6.51	1.49	65.4	5.29	97.0	1.50	81.1

[1] Specifications not less than 96 percent of maximum laboratory density.
Void computations based on apparent specific gravity of aggregates, AASHO Designation: T84-57 and T85-45.

TABLE 56

SUMMARY OF THICKNESS MEASUREMENTS OF ASPHALTIC CONCRETE SURFACING
(Obtained from Data System 2150)

Design Thickness (in.)	No. Measurements	Mean (in.)	Percent of Measurements		
			Within Tolerances [1]	Below Tolerances	Above Tolerances
1	64	1⅛	81.3	0.0	18.7
2	175	2	91.5	5.1	3.4
3	527	3	96.6	2.1	1.3
4	323	4	95.7	2.5	1.8
5	233	5	96.6	1.3	2.1
6	62	5⅞	88.7	11.3	0.0

[1] Specified tolerances, ± ¼ in.

TABLE 57

CHARACTERISTICS OF RECOVERED ASPHALT CEMENT
(Summary of 127 tests)
(Obtained from Data System 2148)

Characteristic	Mean Value	Test Designation
Penetration at 77F, 100 g, 5 sec	57	AASHO T49–53
Ductility at 77F (cm)	150±	AASHO T51–44
Softening point, ring and ball method (°F)	125	AASHO T53–42
Ash (%)	0.55	AASHO T111–42

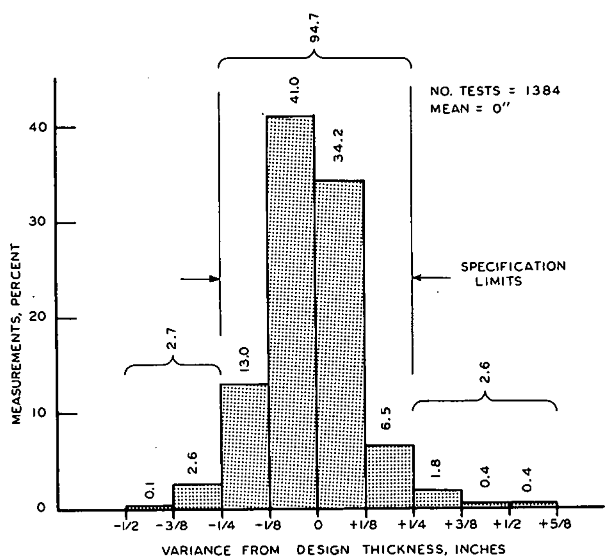


Figure 102. Asphaltic concrete surfacing thickness.

The remaining layers were placed and compacted, and final measurements were taken to the surface at the plate locations for determining the total thickness of the surfacing. A summary of these measurements is given in Table 56. Each measurement represents the mat thickness at a given location and is the average of six individual measurements taken across the width of the pavement.

Figure 102 shows the variation from design thickness for the measurements on all thicknesses of asphaltic concrete. These measurements were recorded to the nearest 1/16 in., and 94.7 per cent of all measurements were within the specified tolerances, with 2.7 percent below and 2.6 above.

The entire area of the completed surface was checked with a 10-ft straightedge. All surface variations in excess of ⅛ in. in 10 ft were corrected with a power grinding machine.

5.4 SUPPLEMENTARY TESTS

Samples of the binder and surface course for Abson recovery tests (AASHO Designation: T170–55) were obtained during the production of the two mixtures. The samples were approximately 5 months old at the time of testing. The results of these tests are summarized in Table 57. The distribution of the penetration test results is shown in Figure 103.

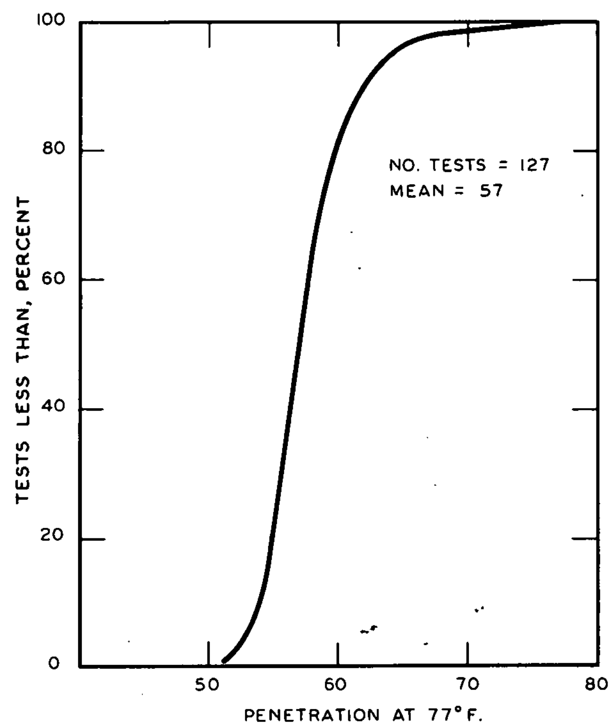


Figure 103. Penetration at 77 F on recovered asphalt cement.

Comparing the values in the table with those obtained on samples the asphalt cement from the storage tanks at the refinery (Table 44, Section 5.1), shows that the mean penetration value had dropped from 91 to 57, that there was no appreciable change in ductility, and that the mean ash content of the recovered asphalt increased by 0.5 percent.

The spot test on the recovered asphalt was reported as positive. However, spot tests on the recovered asphalt by the Bureau of Public Roads and other laboratories indicated a negative value. The spot test was determined by the method of AASHO Designation: T102–57, but the difference in results could be due to the interpretations of the test and to the method of centrifuging the solution. If all the mineral were not removed during centrifuging, it would result in a higher ash content and could account for the positive spot test.

Samples of the aggregates and asphalts cement were furnished to the Bureau of Public Roads for testing. The results of some of the tests are summarized in Appendix D. Also included are tests on laboratory-prepared samples of the binder and surface course mixtures.

Chapter 6

Surfacing—Rigid Pavement

The surfacing on rigid pavements consisted of reinforced and non-reinforced portland cement concrete. This chapter describes the materials, the proportioning and the construction procedures for the surfacing. It outlines the specification requirements for both materials and construction, and includes summaries of data illustrating construction control.

6.1 MATERIALS AND MATERIALS CONTROL

The portland cement concrete contained a coarse aggregate, a natural sand, Type I portland cement, water and an air-entraining agent. The coarse aggregate was obtained in two sizes, Size A (2½-in. maximum) and Size B (1½-in. maximum). Both sizes were used in the concrete for pavements 5 in. and greater in thickness. Only the Size B material was used for the 2½- and 3½-in. thick pavements.

The specifications permitted the use of gravel or crushed stone for the coarse aggregates, and required that both sizes be obtained from a single source and plant. The fine aggregate was required to be a natural sand, non-reactive in character, obtained from one source and plant.

The specified gradation limits for the aggregates were considered only as maximum and minimum values, and the contractor was required to submit gradation formulas consisting of single percentages for the amounts of material passing each of the sieves. The specifications included plus and minus tolerances for variations from the approved formulas. The specified requirements and approved gradation formulas are given in Table 58.

The cement was required to be Type I from one source and one homogeneous lot of material, and manufactured in one continuous burning and grinding. The temperature of the cement at the time of loading at the mill was not to exceed 165 F.

The specifications for the mixing water and water used for curing concrete limited the acidity, alkalinity and organic matter content (Table 59).

The mix designs were based on a fixed cement factor of 1.50 bbl (6 bags) per cu yd, a maximum water content of 5.3 gal per bag of cement, and about a one-third sand-to-total-aggregate ratio. It was required that the cement, aggregates, and water be proportioned by weight to produce a workable, plastic concrete having a minimum compressive strength of 3,500 psi and a minimum modulus of rupture of 550 psi at the age of 14 days. The method specified for determining compressive strength was AASHO Designation: T22–57, except that the specimens were 6 in. in diameter and 12 in. long. The method specified for determining the modulus of rupture was AASHO Designation: T97–57 (third point loading), the specimens were 6- by 6- by 30-in. beams.

The air-entraining agent was added during mixing in amounts designed to obtain an air content within the specified limits of 3 to 6 percent of the volume of the concrete. The consistency of the concrete was required to be such that the slump would be not less than 1½ in. nor more than 2½ in. The proportioning data for both concrete mixes are given in Table 60.

The coarse aggregates were uncrushed gravel composed mainly of dolomitic and argillaceous limestone. These aggregates and the natural siliceous sand were obtained from a deposit at Sheridan, Ill., 20 mi northeast of the project.

During batching operations, samples of the aggregates were taken as the materials were being discharged into the weigh hopper. The samples were taken at an approximate rate of one per hour of production for the two sizes of coarse aggregates and one per two hours for the sand. A summary of the gradations run on these samples is given in Table 61. Segregation of the coarse aggregates made it necessary to split the Size B aggregate into two sizes. The summary included in the table for the gradation of the Size B aggregate was obtained by combining the individual gradations of the two sizes for each test.

The bulk saturated specific gravity was 2.68 for the sand and Size A coarse aggregate, and 2.67 for Size B. Soundness and abrasion tests on the coarse aggregates indicated an average loss ranging from 25 to 30 and 10 to 12 percent respectively.

The Type I portland cement was obtained at Oglesby, Ill., 8 mi southwest of the west end of the project. The cement used on test tangents was produced in the spring of 1958. The total production of the mill for several days was sampled and tested for compliance with the specifications and placed in designated silos. The cement was again sampled after loading

TABLE 58

PORTLAND CEMENT CONCRETE AGGREGATE SPECIFICATION REQUIREMENTS

(a) GRADATION [1]
(Percent Passing Sieve)

Sieve	Coarse Aggregate				Fine Aggregate	
	Size A		Size B			
	Spec. Limits	Formula and Tolerance	Spec. Limits	Formula and Tolerance	Spec. Limits	Formula and Tolerance
2½ in.	100	100				
2 in.	90-100	90-100				
1½ in.	35-70	62 ± 7	100	100		
1 in.	0-15	10 ± 5	90-100	90-100		
½ in.	0-5	0-5	30-50	38 ± 5		
⅜ in.					100	100
No. 4			0-10	0-10	95-100	95-100
No. 8					80-95	85 ± 5
No. 16					45-80	67 ± 4
No. 30					25-60	46 ± 4
No. 50					10-30	13 ± 3
No. 100					0-10	3 ± 2

(b) DELETERIOUS SUBSTANCES

Substance	Max. Percent by Weight [2]		Determined by Test Designation
	Coarse Aggregate	Fine Aggregate	
Soft and unsound fragments	5		ASTM C235–57T
Coal and lignite (also shells for fine)	1	1	AASHO T113–57
Clay lumps	0.25	1	AASHO T112–56
Materials finer than No. 200 sieve	1	3	AASHO T11–49
Other deleterious or injurious material	1		
Conglomerate and cement particles		1	
Thin or elongated pieces (5 times) average thickness	15		

(c) OTHER REQUIREMENTS

Characteristic	Requirement		Determined by Test Designation
	Coarse Aggregates	Fine Aggregates	
Percentage of wear (Los Angeles method)	Not to exceed 35	—	AASHO T96–56
Soundness (5 cycles sodium sulfate)	Not to exceed 15	Not to exceed 10	AASHO T104–57

[1] AASHO Designation: T27–46.
[2] The combined amounts of the first six items shall not exceed 4 percent for the fine aggregate and 5 percent for each coarse aggregate.

TABLE 59

WATER SPECIFICATION REQUIREMENTS

Determination	Spec. Requirement
Acidity and alkalinity:	
Acidity	0.1 normal NaOH 2 ml max.[1]
Alkalinity	0.1 normal HC 10 ml max.[1]
Total solids:	
Organic	0.02 percent max.
Inorganic	0.30 percent max.
Sulfuric anhydride, SO_3	0.04 percent max.
Alkali chloride as sodium chloride	0.10 percent max.

[1] To neutralize 200-ml sample.

TABLE 60

PORTLAND CEMENT CONCRETE PROPORTIONING DATA [1]

Characteristic	For Pavement Thickness	
	5 In. and Greater	2½ and 3½ In.
Total mixing water (gal)	4.8	4.9
Absolute volume (cu ft):		
Sand	1.02	1.08
Coarse aggregate	2.16	2.09
Mortar (cu ft)	2.34	2.41
Yield (cu ft)	4.5	4.5
Proportioning weights (lb):		
Coarse aggregate Size A	180.5	—
Coarse aggregate Size B	179.9	348.0
Sand	170.5	180.5

[1] Per bag of cement.

into railroad cars for shipment to the job site. Tables 62 and 63 summarize the results of tests on the samples obtained after loading. All tests were performed according to the latest ASTM methods (as of 1958) in a temperature and humidity controlled room.

Mixing water was obtained from a well at the batching plant. Tests on samples of the water indicated that it met all the requirements of the specifications except for alkalinity. The specifications set a maximum of 10 ml of 0.1 normal HCl to neutralize a 200 ml sample, but 15 ml were required. However, the water was approved for use since it had a pH value of 8.1,

TABLE 61

SUMMARY OF GRADATION TESTS ON P.C.C. AGGREGATES
(Obtained from Data System 2233)

Sieve	Gradation Formula and Tolerances	Mean Percent of Material Passing	Standard Deviation	Percent of Tests		
				Within Tolerances	Above Tolerances	Below Tolerances
(*a*) COARSE AGGREGATE SIZE A (170 TESTS)						
2½ in.	100	100	—	100	0.0	0.0
2 in.	90-100	96.3	3.45	94.1	0.0	5.9
1½ in.	62 ± 7	63.5	6.11	77.6	16.5	5.9
1 in.	10 ± 5	10.6	3.18	92.9	5.3	1.8
½ in.	0-5	3.8	2.14	84.6	15.4	0.0
(*b*) COARSE AGGREGATE SIZE B (171 TESTS)						
1½ in.	100	100	—	100	0.0	0.0
1 in.	90-100	94.1	1.30	99.4	0.0	0.6
½ in.	38 ± 5	37.9	1.65	99.4	0.6	0.0
No. 4	0-10	1.5	0.78	100	0.0	0.0
(*c*) P.C.C. SAND (80 TESTS)						
⅜ in.	100	100	—	100	0.0	0.0
No. 4	95-100	99.0	0.97	98.7	0.0	1.3
No. 8	85 ± 5	84.1	1.55	100	0.0	0.0
No. 16	67 ± 4	67.0	1.83	98.7	1.3	0.0
No. 30	46 ± 4	45.4	1.51	100	0.0	0.0
No. 50	13 ± 3	12.3	0.73	100	0.0	0.0
No. 100	3 ± 2	2.7	0.46	100	0.0	0.0

TABLE 62

PHYSICAL PROPERTIES OF TYPE I PORTLAND CEMENT
(Obtained from Data System 2234)

Silo No.	No. Tests	Autoclave Expansion (%)	Time of Set: Initial (hr)	Time of Set: Initial (min)	Time of Set: Final (hr)	Time of Set: Final (min)	Fineness, Blaine (sq cm/g)	Compressive Strength (psi): 3 Day	7 Day	28 Day	90 Day	Air (%)
16	9	0.23	3	10	5	40	3407	3343	4679	6228	6783	5.7
8	9	0.21	3	08	5	36	3432	3063	4344	5600	6287	7.2
10	13	0.23	3	25	5	50	3408	3063	4413	5829	6233	6.8
7	13	0.21	3	16	5	46	3406	3113	4474	5936	6179	8.2
11	12	0.20	3	36	5	49	3432	3069	4471	5877	6264	7.1
13	12	0.18	3	08	5	09	3398	3262	4760	5892	6297	6.7
15	9	0.22	3	18	5	18	3426	3429	5067	5855	6009	6.5
17	9	0.21	3	10	5	30	3400	3369	4946	5774	5879	7.6
All	86	0.21	3	17	5	35	3413	3200	4624	5875	6241	6.9

and the results of strength tests were almost identical to those obtained with distilled water. The results of the tests on the mixing water are given in Table 64.

Darex AEA was used as the air-entraining agent. It had a specific gravity of 1.022 and 11.4 percent non-volatiles.

The dowel bars for contraction joint assemblies were intermediate grade billet-steel conforming to AASHO Designation: M31–54. The assemblies were coated with two applications of coal-tar base mill coating at the fabrication plant. The tie bars for the longitudinal center-joint assemblies were structural and intermediate grade billet-steel also conforming to the previous AASHO specification. The reinforcement was electric-welded steel wire fabric conforming to AASHO Designation: M55-55.

The batching plant was located at the center of the project. The stockpile areas were covered with a bituminous mixed-in-place gravel mat. The aggregates were stockpiled by a crane equipped with a clam-shell bucket. A conveyor belt was used to place the aggregates in the various compartments of the batching bin. A rubber-tired front-end loader moved the materials from the stockpiles to the hopper of the conveyor belt.

The aggregate batching bin contained four compartments with a weigh hopper beneath the

TABLE 63

CHEMICAL COMPOSITION OF TYPE I PORTLAND CEMENT
(Obtained from Data System 2234)

Silo No.	No. Tests	Insol. Residue (%)	Ignition Loss (%)	Sulfuric Anhydride (%)	Magnesia, Mgo (%)
16	9	0.19	0.99	2.49	1.82
8	6	0.17	1.46	2.54	1.79
10	12	0.18	1.22	2.41	1.78
7	13	0.15	2.05	2.42	1.75
11	12	0.16	2.48	2.36	1.75
13	12	0.17	2.11	2.27	1.74
15	6	0.14	2.05	2.35	1.79
17	9	0.19	1.58	2.43	1.75
All	79	0.17	1.78	2.40	1.77

TABLE 64

TEST RESULTS ON MIXING WATER

(a) PHYSICAL TESTS OF CEMENT SPECIMENS

Comparison of well with distilled mixing water.

	Well Water	Distilled Water
Time of set (hr, min):		
Initial	3, 45	3, 30
Final	6, 0	6, 0
Autoclave expansion (%)	0.17	0.18
Compression strength (psi):		
2-in. cube, 3 days	2600	2792
2-in. cube, 7 days	3367	3833
Tensile strength (psi):		
Sand briquets, 3 days	335	367
Sand briquets, 7 days	437	437

(b) CHEMICAL TESTS OF WELL WATER

Appearance, clear and transparent	Total solids:
Reaction to:	Organic, 0.01 percent
Litmus, slightly alkaline	Inorganic, 0.03 percent
Methyl orange, alkaline	Sulfuric anhydride, 0.001 percent
Phenolphthalein, none	Alkali chloride, trace
Tests for sugars, negative	Alkalinity,[1] 15.0 ml

[1] 0.1 normal HCl for 200-ml sample.

bin equipped to weigh the fine aggregate and the individual sizes of coarse aggregates cumulatively.

The weigh hopper beneath the cement silo was completely enclosed and constructed so as to eliminate the accumulation of tare material and leakage through the discharge boot. The cement scales were separate from the scales used for weighing the aggregates.

In the early fall of 1957, the plant was set up and placed in operation for paving the pilot sections and rigid pavement turnarounds. Considerable difficulty was encountered with segregation of the coarse aggregates. Tests on samples obtained from the batching bin indicated that the segregation was principally in the fraction of material passing the 1½-in. sieve for Size A aggregate and the ½-in. sieve for Size B.

Segregation of the Size A aggregate was eliminated by using a different method of stockpiling. Each truck load was distributed in a thin layer over the entire area of the stockpile, and the height of the stockpile was limited to approximately 6 ft.

To eliminate segregation in the Size B aggregate, it was necessary to separate the material into two sizes at the source and to proportion them separately during batching operations. The material retained on a ⅝-in. screen (termed Size B-1) constituted 70 percent of the weight of Size B aggregate and the material passing the screen (termed Size B-½) constituted 30 percent.

In addition to the gradation analyses conducted on samples obtained from the batching bin, tests for control of batching plant operations included stockpile moisture determinations and stockpile gradations.

The moisture content of the various sizes of aggregates was continually checked. Stockpile moisture contents were determined by a water-immersion method on samples of each size of aggregate taken at a rate of about one for every hour of production. Stockpile gradations were determined from two composite samples from each 3 ft of stockpile placement for each size of aggregate. Specific gravity determinations were made on the various sizes as routine checks on previous determinations made on samples taken at the source.

The cement, coarse aggregates, sand and water were proportioned in 37.4-cu ft batches. The cement and aggregates were delivered to the mixer in 4-compartment batch trucks. A closed waterproof container was provided in each compartment for the cement.

6.2 CONSTRUCTION PROCEDURES

The specifications required that all equipment-crossing be made within the transition areas between construction blocks, and prohibited the paving mixer and batch trucks from operating within the center 24-ft portion of the roadbed.

The portland cement concrete was mixed in a 34-E dual-drum paving mixer equipped with a boom and bucket. The specifications required that the total mixing time be not less than 60 sec. A portion of the mixing water had to enter the drum in advance of the cement and aggregates, and all the water had to be added within the first 15 sec. The air-entraining agent was introduced into the stream of mixing water before all the water entered the drum, and the dispenser was equipped with a device to indicate visually when the supply was running low.

The concrete was spread mechanically, and the operation was required to be continuous for the full length of a structural section. In case of an emergency, the specifications permitted construction joints to be formed within structural sections but only at the location of a contraction joint. On one occasion it was necessary to form a construction joint within a structural section. All other construction joints were formed at contraction joints separating structural sections from transitions or within transitions. All construction joints were formed by placing header boards over the dowel-bar assemblies.

The wire fabric for reinforced pavements 5 in. and greater in thickness was placed by the double strike-off method. That for the 2½- and 3½-in. pavements was set in place prior to placing the concrete.

Standard methods were used to place and finish the concrete. The sequence of operations included placing and spreading, strike-off and consolidation, longitudinal floating, straightedging, belting, edging, and final finish with a burlap drag. The wetted-straw method was used to cure the concrete.

The specifications required that the thickness of the completed pavement be within ¼ in. of the design thickness, and that the surface smoothness be within ⅛ in. in 10 ft.

Pilot studies were conducted mainly to determine a satisfactory method to support the wire fabric in the two thinner slabs and to test the double strike-off method for fabric placement in the 5-in. reinforced pavement.

It was determined that the fabric in the two thinner reinforced slabs could be satisfactorily supported by metal chair assemblies. Reinforcement bar chairs of No. 8 gage steel wire were tack-welded on 12-in. centers to No. 16 gage metal sand plates, 9.5 ft long and 3 in. wide. The sand plates were metal strips which supported assemblies on the subgrade. The assemblies, placed on the subgrade parallel to the centerline of the pavement, were spaced transversely at 18-in. centers for the 2½-in. pavement and at 24-in. centers for the 3½-in. pavement. The fabric was tack-welded to alternate chairs of the assemblies. It also was

Figure 104. Placing fabric in 2½- and 3½-in. pavements on Loop 2.

TABLE 65

FORMS FOR P.C.C. PAVEMENT

Pavement Thickness (in.)	Form Height (in.)
2½	3½
3½	3½
5	5
6½	7
8	9
9½	10
11	12
12½	12

determined that the tie-bar assemblies for the center joint of the 5-in. reinforced pavement had to be installed after the initial strike-off of the concrete to prevent them from being displaced. The metal sand plates on these assemblies were removed, and the assemblies were held in position with small steel pins.

Paving on test tangents was done by two separate outfits with identical pieces of equipment. One outfit paved the tangent on Loops 1, 2, 3 and 4, and the other paved on Loops 5 and 6. Paving within each loop was started at the west end of the tangent.

New forms were provided for all thicknesses of test pavements. Forms for the thinner

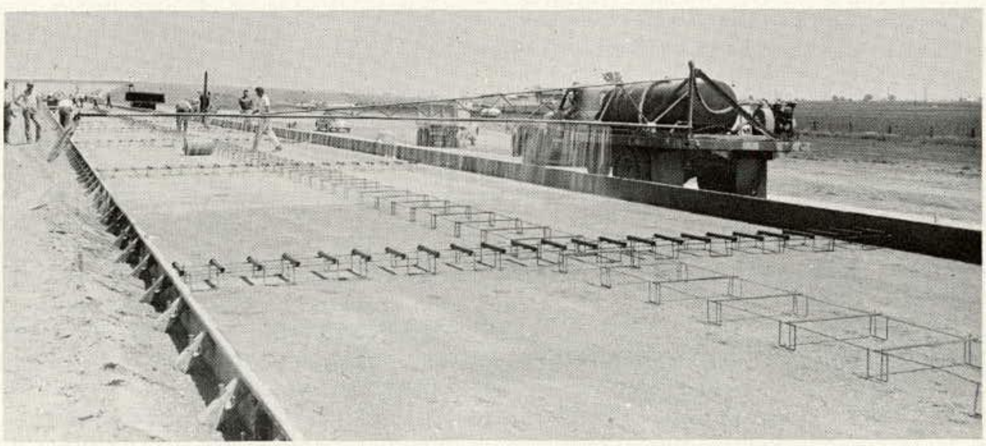


Figure 105. Wetting subgrade ahead of paving; transverse and longitudinal joint assemblies in place.

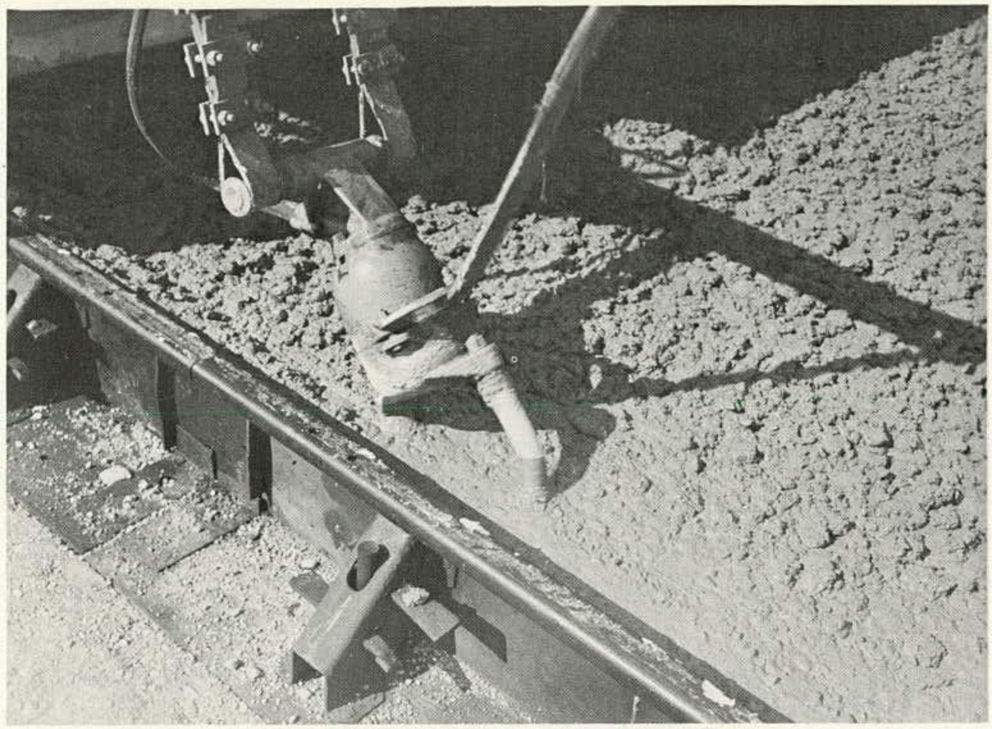


Figure 106. Vibrator on concrete spreader.

pavements had to be specially fabricated. The specifications required that the forms have a base width equal to or greater than the height, but not less than 8 in.

Six heights of forms were used for the eight thicknesses of concrete pavement (Table 65). These heights insured that the maximum depth of cutting below the bottom of the slab did not exceed 1 in., and building up of forms was required only for the 12½-in. pavement.

After subgrading, the tie-bar and dowel-bar assemblies for the longitudinal and transverse joints were positioned on the subgrade and anchored with steel pins. (As previously mentioned, the tie-bar assemblies for the 5-in. reinforced concrete pavements had to be set in place after the initial strike-off for the fabric.) Each dowel-bar was set parallel to the form grade and coated with a medium curing liquid asphalt (MC-1).

The concrete was finished by the non-vibratory method. Internal vibration was required along both side forms and along the longitudinal and transverse joint assemblies. The spreader was equipped with a vertically-adjustable horseshoe-type internal vibrator mounted along each side form and along each side of the longitudinal joint assembly. Hand-operated spud-type vibrators were used along the transverse joint assemblies.

After depositing and spreading, the concrete was struck off and consolidated with at least two passes of a finishing machine. It was further smoothed and consolidated by a mechanical longitudinal float. The surface was checked with a 10-ft straightedge. After most of the water sheen had disappeared, it was belted with one application of a mechanical belt. This was followed by edging and two passes of a double-thickness burlap drag.

When the concrete had attained a sufficient set, it was covered with two layers of wetted burlap, kept saturated until removed. The morning following placement, the forms and burlap were removed and the surface and edges of the pavement were covered with a layer of clean straw. The straw was saturated with water and kept wet for the first three days. It was again saturated on the morning of the fourth day, and was left in place until test beams indicated that the concrete had attained a flexural strength of at least 550 psi.

All transverse joints were sawed with a 4-blade machine using diamond blades and operating on the forms. These joints were sawed during the morning following the placement of the concrete prior to removing the forms and burlap. The longitudinal center-joints were sawed with a single-blade (diamond blade) machine after curing was completed and the straw cover removed.

Immediately after a joint was sawed, it was cleaned and jute roving was placed in the sawed groove. The jute roving was removed after the shoulders on a tangent were completed and the grooves were cleaned and sealed with a cold-applied compound.

A record of construction by blocks on the

Figure 107. Paving on test tangent.

rigid tangent of each test loop is given in Table 66, which also includes the dates of subgrading for paving and of removal of the straw cover.

6.3 CONSTRUCTION CONTROL

The testing program for construction control of the concrete pavement included tests for air content, slump and yield of the plastic concrete, as well as compressive and flexural strengths of the hardened concrete.

In addition, a record was made of the location in the pavement of the four batches of concrete in each load of material. The air temperature was recorded three times daily, and the temperature of the cement was taken in one of the four batches in each truck. The temperature of the plastic concrete was recorded during placement of each batch from which beams and cylinders were made for 14-day strength tests.

Tests for air content and slump were made on every batch from which beams and cylinders were molded for 14-day strength tests, and on intermediate batches so that the rate of testing was one for every 10 to 20 min of concrete placement on a tangent. Yield determinations also were made on the batches of concrete from which these beams and cylinders were molded.

Two beams and two cylinders were molded from two randomly-selected batches of concrete for each 120-ft structural section; one beam and two cylinders from one batch, one beam from the other. For each 240-ft structural section, four beams and four cylinders were made from four separate batches; one beam from each of two of the batches, and one

Figure 108. Placing straw for curing concrete.

Figure 109. Sawing transverse contraction joints.

Figure 110. Testing plastic concrete.

Figure 111. Preparing concrete test specimens.

TABLE 66

RECORD OF PORTLAND CEMENT CONCRETE PAVEMENT CONSTRUCTION

Const. Block No.	Const. Sequence [1]	Loop 1	Loop 2	Loop 3	Loop 4	Loop 5	Loop 6
R–1	1	6–30/7–1, 7–7–58	6–26/27–58	5–16/17–58	6–6, 6–16–58	6– 7–58	5–19/20–58
	2	7–7/9–58	6–27–58	5–19/20–58	6–17–58	6–17–58	5–21/22–58
	3	7–17–58	6–8/10–58	6–17–58	6–27–58	6–27–58	6–17/20–58
R–2	1	7–7/8–58	6–27–58	5–20, 26–58	6–16/17–58	6–16/17–58	5–20/21–58
	2	7–9–58	6–28–58	5–20, 6–6–58	6–17–58	6–17/18–58	5–23–58
	3	7–17/21–58	7–10–58	6–17–58	6–27–58	6–27/28–58	6–20/23–58
R–3	1	7–8/9–58	6–27/28–58	5–20–58	6–17–58	6–17–58	5–21/23–58
	2	7–9/10–58	6–28/30–58	5–21–58	6–18–58	6–18–58	5–23/26–58
	3	7–21/22–58	7–10–58	6–17–58	6–27/28–58	6–30–58	6–23–58
R–4	1		6–28–58	5–21–58	6–18–58	6–18–58	5–23/26–58
	2		6–30–58	5–22/23–58	6–19–58	6–19–58	5–26/27–58
	3		7–10–58	6–19–58	6–28–58	6–30–58	6–23–58
R–5	1		6–30/7–1–58	5–22/24–58	6–18/19–58	6–19–58	5–26/27–58
	2		6–30/7–2–58	5–23/26–58	6–20–58	6–19/20–58	5–27/28–58
	3		7–10/11–58	6–19–58	6–28–58	6–30–58	6–23–58
R–6	1		7– 1–58	5–26/28–58	6–20–58	6–19/20–58	5–27/28–58
	2		7– 2–58	5–27/6–4–58	6–20/23–58	6–20/23–58	5–28/29–58
	3		7–11/16–58	6–19–58	6–28–58	6–30–58	6–23/24–58
R–7	1		7– 2–58	5–28/29–58	6–20/21–58	6–20/21–58	5–28/6–3–58
	2		7– 2–58	6–4/5–58	6–23–58	6–23–58	5–29/6–4–58
	3		7–16–58	6–19–58	7– 1–58	6–30–58	6–24–58
R–8	1			5–28/6–5–58	6–21/23–58	6–23–58	6–3/4–58
	2			6– 5–58	6–23–58	6–23/24–58	6– 4–58
	3			6–20–58	7– 1–58	6–30/7–2–58	6–26–58
R–9	1			6– 5–58	6–23/24–58	6–24–58	6– 4–58
	2			6– 6–58	6–24/26–58	6–24–58	6–4/5–58
	3			6–20–58	7– 1–58	7– 2–58	6–26–58
R–10	1				6–24–58	6–24/26–58	6–4/5–58
	2				6–26–58	6–26–58	6– 5–58
	3				7– 2–58	7– 2–58	6–26–58

[1] Construction sequence 1 is subgrading for paving; 2 is paving; 3 is removal of straw cover.

TABLE 67

SUMMARY OF TEST RESULTS ON PLASTIC CONCRETE [1]
(Obtained from Data System 2230)

Loop	Slump [2] (in.)			Air Content [3] (%)			Cement Factor [4] (bags/cu yd)			Water-Cement Ratio [5] (gal/bag)		
	No. Tests	Mean	Std. Dev.	No. Tests	Mean	Std. Dev.	No. Tests	Mean	Std. Dev.	No. Tests	Mean	Std. Dev.
(a) 2½-IN. MAXIMUM SIZE AGGREGATE												
1	49	2.7	0.47	48	3.4	0.32	16	6.11	0.058	15	4.76	0.169
2	31	2.7	0.92	29	4.0	0.65	20	6.08	0.054	20	4.91	0.118
3	151	2.7	0.82	149	4.0	0.44	67	6.04	0.043	67	4.77	0.139
4	152	2.1	0.68	149	3.6	0.46	98	6.09	0.049	98	4.69	0.099
5	176	2.3	0.54	175	3.6	0.33	97	6.10	0.039	96	4.54	0.092
6	214	2.6	0.63	211	3.7	0.47	95	6.10	0.037	95	4.57	0.102
All	773	2.5	0.70	761	3.7	0.49	393	6.08	0.048	391	4.65	0.152
(b) 1½-IN. MAXIMUM SIZE AGGREGATE												
1	5	2.6	0.14	5	3.9	0.67	4	6.10	0.033	4	4.72	0.127
2	50	2.8	0.78	49	4.0	0.68	39	6.09	0.045	39	4.84	0.145
3	49	2.6	0.78	48	4.4	0.64	24	6.05	0.046	24	4.76	0.077
All	104	2.7	0.76	102	4.2	0.69	67	6.07	0.048	67	4.81	0.130

[1] 2½-in. maximum size aggregate for pavements 5 in. and greater in thickness; 1½-in. maximum size for 2½- and 3½-in. pavements.
[2] AASHO Designation: T119–42.
[3] AASHO Designation: T152–53.
[4] Computed on basis of measured yield per batch and measured cement per batch.
[5] Computed on basis of actual water added at mixer, corrected for free and absorbed moisture in aggregates.

TABLE 68

SUMMARY OF TEST RESULTS ON HARDENED CONCRETE
(Obtained from Data System 2230)

Loop	Flexural Strength [1], 14 Days			Compressive Strength [2], 14 Days		
	No. Tests	Mean (psi)	Std. Dev. (psi)	No. Tests	Mean (psi)	Std. Dev. (psi)
(a) 2½-IN. MAXIMUM SIZE AGGREGATE						
1	16	637	46	8	3599	290
2	20	648	37	9	3603	281
3	71	630	44	38	3723	301
4	96	651	38	48	4062	288
5	96	629	28	48	4196	388
6	99	628	51	48	3963	325
All	398	636	45	199	3966	376
(b) 1½-IN. MAXIMUM SIZE AGGREGATE						
1	4	676	65	2	4088	162
2	39	668	44	19	4046	295
3	24	667	47	14	3933	440
All	67	668	46	35	4004	352

[1] AASHO Designation: T97–57 (6x6x30-in. beams).
[2] AASHO Designation: T22–57 (6-in. dia. x 12-in. long cylinders).

beam and two cylinders from each of the other two. These specimens were used to determine the 14-day flexural and compressive strengths of the concrete in the test pavements.

Three additional concrete beams for each paving outfit were molded per day for flexural strength tests to determine the required length of curing time.

Table 67 is a summary of the results of tests on the plastic concrete for slump, air content, cement factor and water-cement ratio. Figures 112 and 113 show the distribution of the results of all tests for slump and for air content.

The distribution of slump tests for both mixes was centered about the upper specification limit (Fig. 112). This was purposely done to reduce the difficulties encountered in finishing the concrete and to eliminate the necessity for sprinkling the surface during the floating operation. If the upper specification limit were extended from 2½ to 3 in., the percentage of tests within the specifications would be 84.3 for the mix having 2½-in. maximum size aggregate and 77 for the other.

Table 68 and Figures 114 and 115 contain data for the 14-day flexural and compressive strength tests on the concrete used in test pavements. Each test for flexural strength represents the average of two breaks on one beam, while each compressive strength test is the average of the strength obtained from each of two cylinders in a set.

The thickness of the portland cement concrete pavement was determined from elevations of the completed subgrade and of the top of the completed pavement. Elevations were taken at the centerline and at 4, 8 and 12 ft on each side at each end of a structural section, and at 20-ft intervals for the 120-ft sections and 40-ft intervals for the 240-ft sections. The thickness measurements are summarized in Table 69. Each measurement represents the average of the seven thicknesses determined at a given cross-section.

Variations from design thickness for all thicknesses are shown in Figure 117. Measurements were recorded to the nearest ⅛ in. and 93.8 percent of all measurements were within the specified tolerances, with 2.7 percent below and 3.5 above.

The surface of the completed pavements was tested with a 10-ft straightedge. All variations in excess of ⅛ in. in 10 ft were corrected by a power grinding machine.

6.4 SUPPLEMENTARY TESTS

Concrete beams and cylinders were molded during the paving operations on the test tangents for determining flexural and compressive strengths at the ages of 3, 7 and 21 days; 3 and 12 months; and 2 years. A set of specimens consisting of six beams and twelve cylinders were made for every 5 days of paving for each of the two paving outfits. The concrete for all specimens of any one set was obtained from a single batch. The specimens were cured in moist sand and tested by the methods described in Section 6.3 for the 14-day flexural and compressive strengths. One beam and two cylinders from each set of specimens were tested at each designated age. The results of these tests are summarized for each mix design in Table 70. Each test for flexural strength represents the average of two breaks on one beam. Each test for compressive strength was obtained by averaging the results of individual tests on two cylinders from one set of specimens.

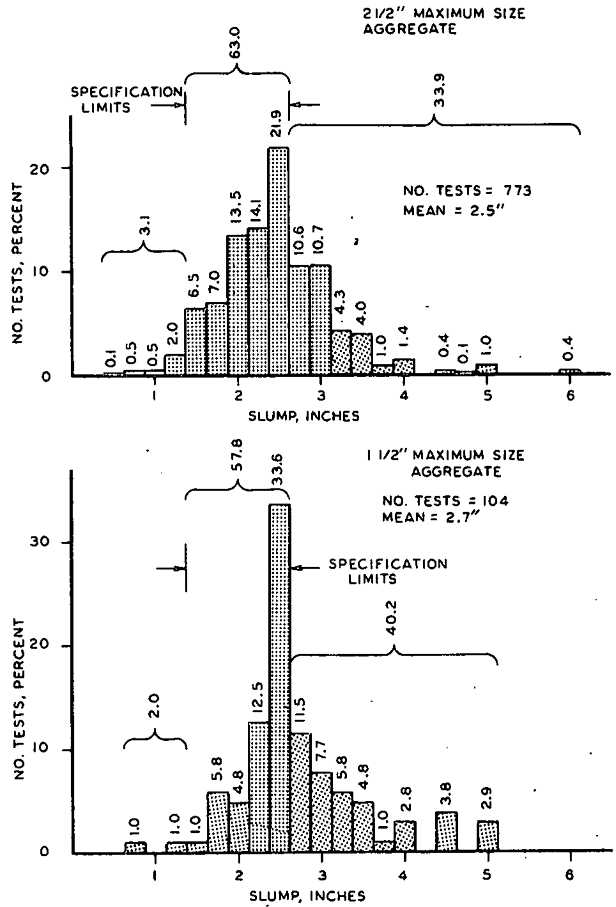


Figure 112. Slump.

TABLE 69

SUMMARY OF THICKNESS MEASUREMENTS OF P.C.C. PAVEMENT
(Obtained from Data System 2239)

Pavement Design Thickness (in.)	No. Measurements	Mean Thickness (in.)	Percent of Measurements		
			Within Tolerances[1]	Below Tolerances	Above Tolerances
2½	58	2 9/16	94.8	0.0	5.2
3½	103	3½	91.3	1.9	6.8
5	208	4 15/16	92.8	5.8	1.4
6½	188	6½	93.6	4.3	2.1
8	227	8	94.3	3.5	2.2
9½	184	9 9/16	96.8	0.5	2.7
11	119	11 1/16	94.4	0.0	5.6
12½	59	12 9/16	93.2	0.0	6.8

[1] Plus or minus ¼ in.

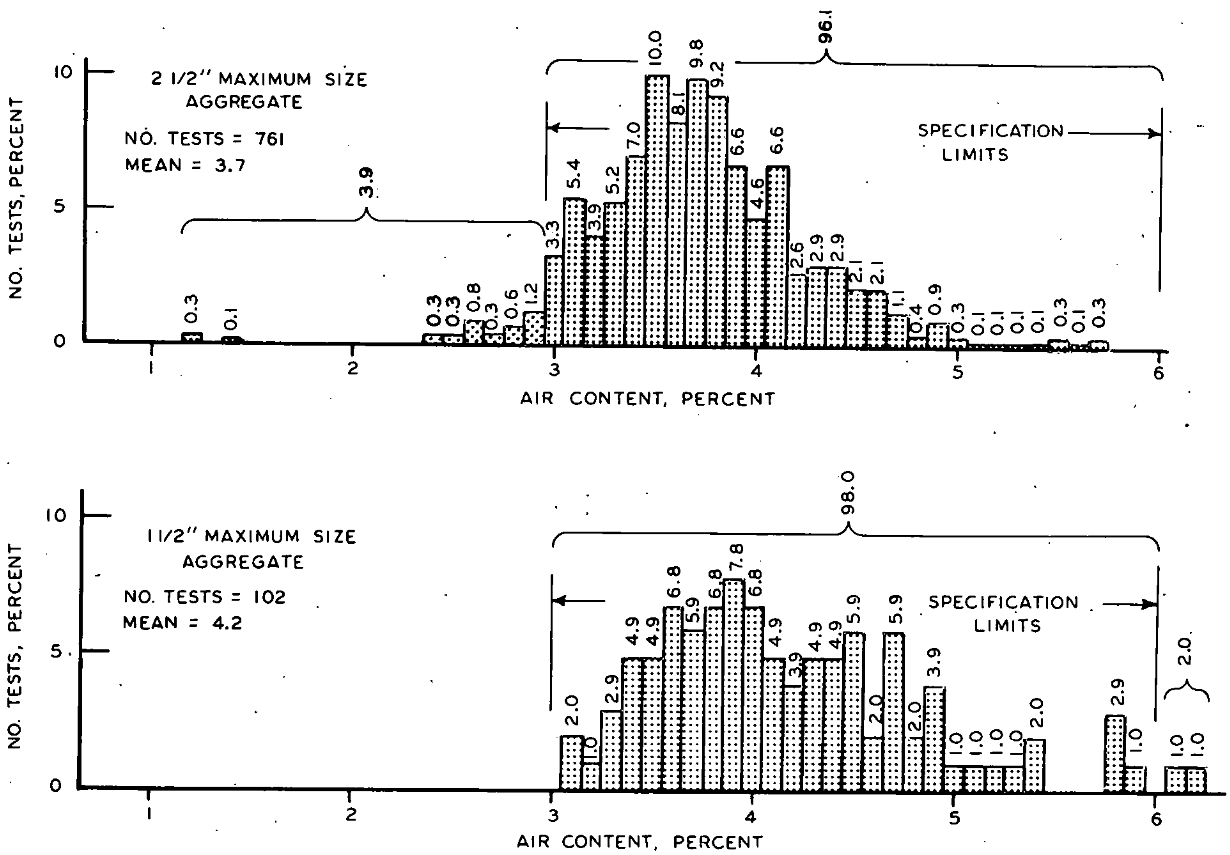


Figure 113. Air content.

TABLE 70

SUMMARY OF STRENGTH TESTS
(Obtained from Data System 2231)

Age at Testing	Flexural Strength (psi)			Compressive Strength (psi)		
	No. Tests	Mean	Std. Dev.	No. Tests	Mean	Std. Dev.
	(a) 2½-In. Maximum Size Aggregate					
3 days	11	510	23	11	2670	784
7 days	11	620	34	11	3560	396
21 days	11	660	51	11	4130	397
3 months	11	770	66	11	4680	487
1 year	11	790	61	11	5580	509
2 years	11	787	66	11	5818	328
	(b) 1½-In. Maximum Size Aggregate					
3 days	12	550	37	12	2860	809
7 days	12	630	35	12	3780	289
21 days	12	710	53	12	4250	365
3 months	12	830	41	12	4930	528
1 year	10	880	53	12	5990	379
2 years	12	873	48	12	6155	373

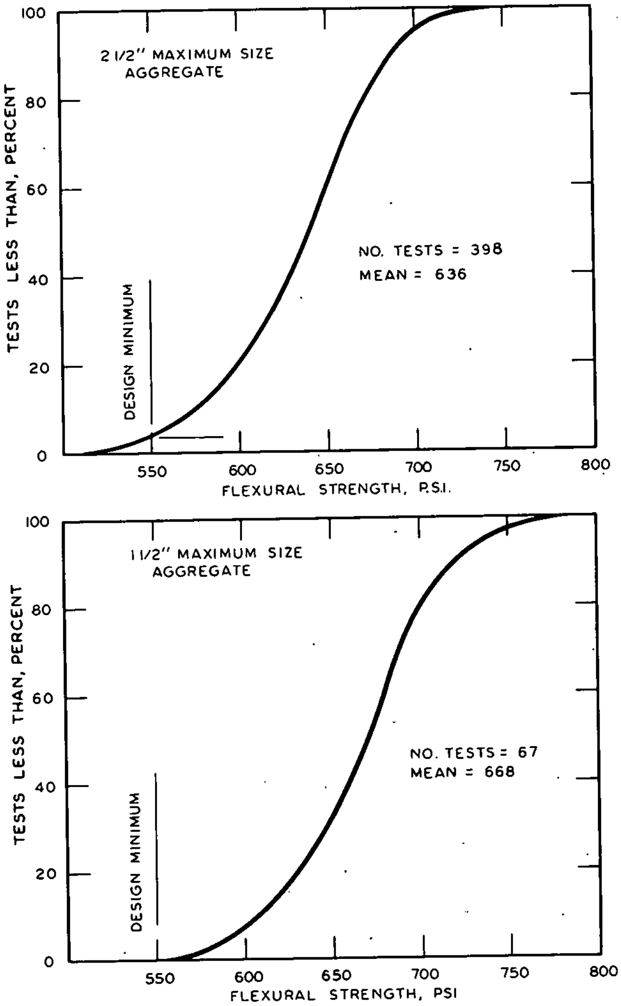


Figure 114. Distribution of 14-day beam strengths.

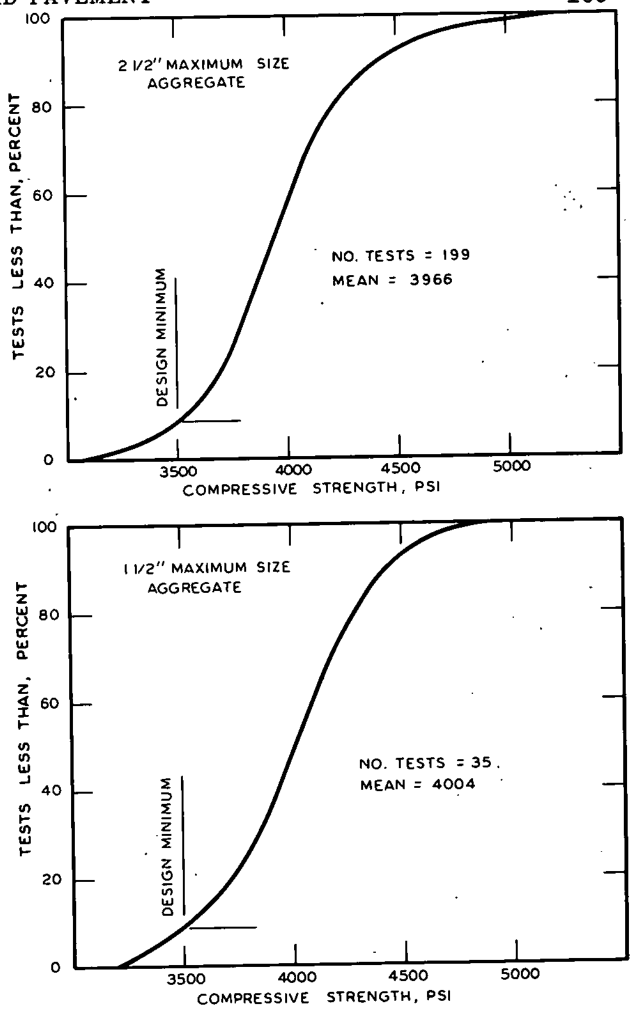


Figure 115. Distribution of 14-day cylinder strengths.

The cores taken from the pavements were tested to determine the air content of the hardened concrete. These tests were performed by the central laboratory of the Illinois Division of Highways in a high-pressure air meter* at 5,000-psi pressure. The cores were dried in an oven at approximately 300 F for 72 hr and then immersed in water for 48 hr prior to testing. The unit weight, absorption and air content were determined for 166 cores. The results of these tests are summarized for each mix design in Table 71.

The tests on the cores indicated that the air content of the hardened concrete was uniform throughout for both mix designs—3.2 percent for the mix containing the larger size aggregate and 3.1 percent for the other. These values, however, were somewhat smaller than those obtained for the plastic concrete. As shown in Table 67 of Section 6.3, tests on samples of the plastic concrete indicated an air content of 3.7 percent for the mixture containing the 2½-in.

* Lindsay, J. D., "Illinois Develops High Pressure Air Meter for Determining Air-Content of Hardened Concrete." *HRB Proc.*, 35: 424–435 (1956).

TABLE 71

TESTS ON CORES FROM PAVEMENT
(Obtained from Data System 2240)

Loop No.	No. Tests	Weight (pcf)	Absorption (%)	Air-Content (%) Mean	Air-Content (%) Std. Dev.
(a) 2½-In. Maximum Size Aggregate					
1	8	151.1	4.66	3.2	0.71
2	7	151.4	4.78	3.1	0.50
3	25	150.7	4.83	3.4	0.53
4	34	151.0	4.74	3.0	0.53
5	34	150.8	4.69	3.3	0.52
6	34	151.0	4.45	3.1	0.40
All	142	150.9	4.67	3.2	0.52
(b) 1½-In. Maximum Size Aggregate					
1	2	151.0	5.17	3.0	1.13
2	13	151.7	4.80	3.0	0.36
3	9	150.8	4.75	3.5	0.44
All	24	151.4	4.81	3.1	0.50

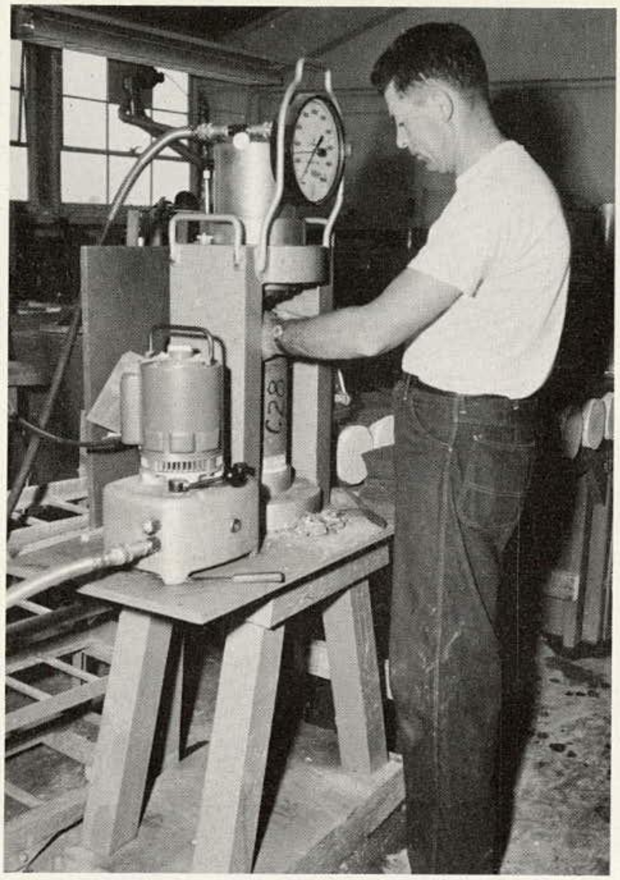


Figure 116. Testing concrete cylinders.

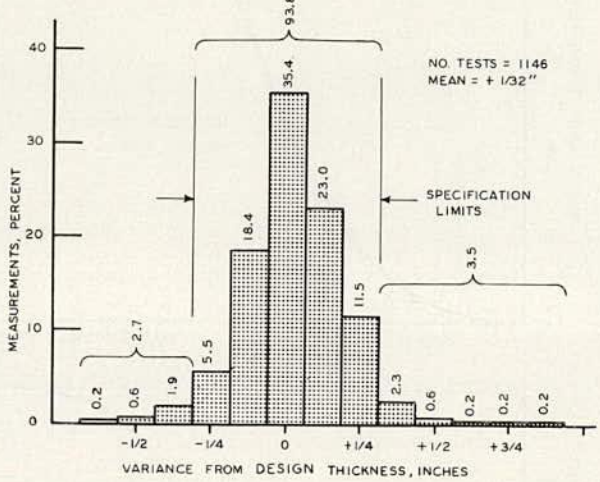


Figure 117. Portland cement concrete pavement thickness.

maximum size aggregate and 4.2 percent for the other.

Tests conducted by the Bureau of Public Roads on the materials used in the concrete are summarized in Appendix D. Included are chemical analysis of the cement, physical properties of the aggregates, lithological analysis of the coarse aggregates and alkali-reactivity of the cement and aggregates.

Chapter 7

Test Bridges

The bridge research program was planned as a series of case studies. It included bridge type and stress level as the major design variables. A general description of the test bridges is included in Section 1.3.4. This chapter details each bridge type. It outlines the specification requirements and procedures for material and construction control, and includes summaries of test data giving physical properties of the materials and as-constructed characteristics of the test bridges.

7.1 BRIDGE DETAILS

7.1.1 Substructures and Approach Slabs

The four bridges at each of the four locations (Fig. 4, Section 1.1.2) were supported on a common concrete substructure consisting of two abutments and one pier on spread footings. The piers and end abutments for all bridge structures were identical in all major details. The dimensions of the substructures are shown in Figures 118 and 119. They were dictated in part by the necessity of access to the underside of the bridges.

The footings were located on a hard clay till—either gray, mottled, or brown in color. The unconfined compressive strength of the foundation soils varied from 5 to 10 tons per sq ft. The design of the footings was based on a maximum toe pressure of 1½ tons per sq ft.

Steel bearings supported the superstructures of the test bridges. Fixed bearings were placed on the center pier and expansion rockers on the end abutment. A 7- by 16- by ⅛-in. Fabreeka pad provided distribution of the load from the bearing to the substructure. Details of the bearings are shown in Figure 120.

Bridge approach slabs of portland cement concrete were constructed at each site. They were 20 ft long, heavily reinforced with deformed bars, and supported at one end by the abutments. The slab at the traffic approach end of each site was 28 ft wide and the one at the traffic runoff end was 24 ft. wide. Details of the bridge approach slabs are shown in Figure 122.

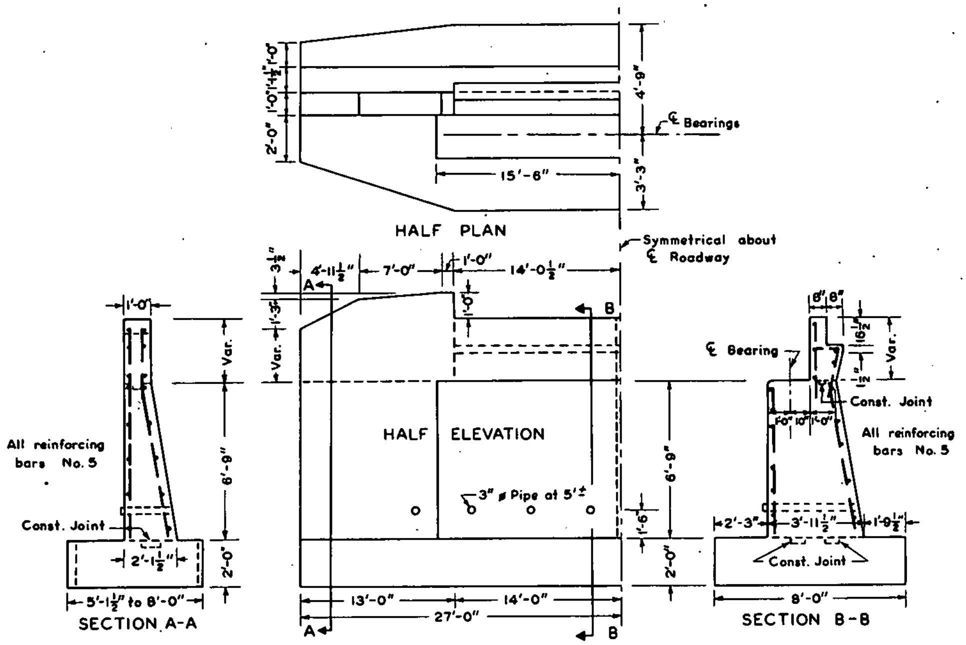


Figure 118. Typical abutment.

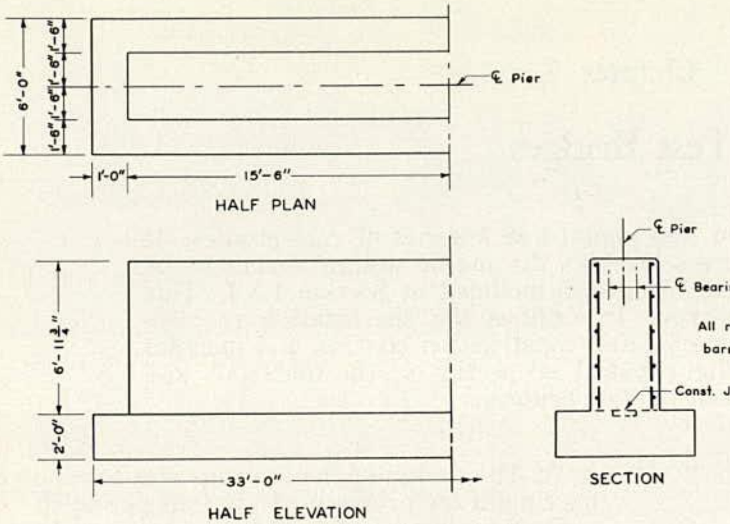


Figure 119. Typical pier.

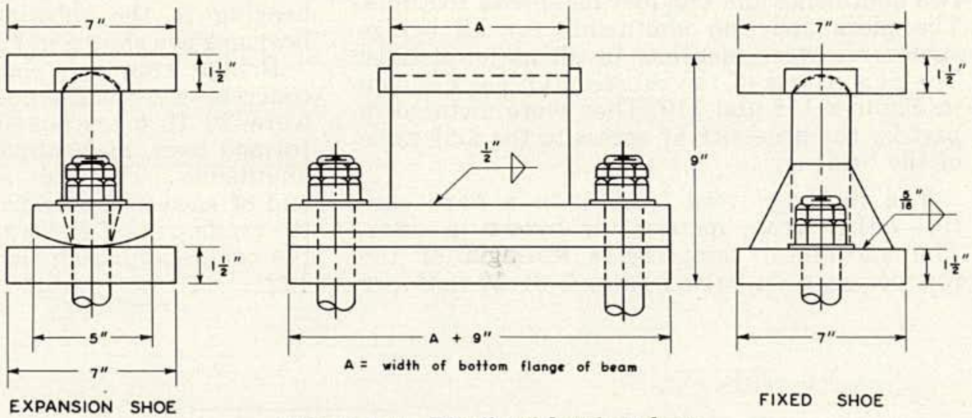


Figure 120. Details of bearing shoes.

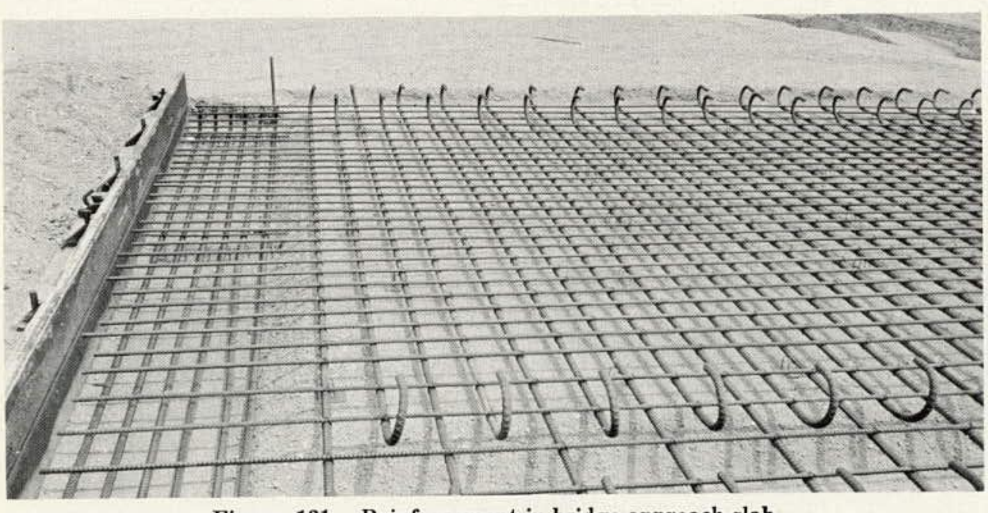


Figure 121. Reinforcement in bridge approach slab.

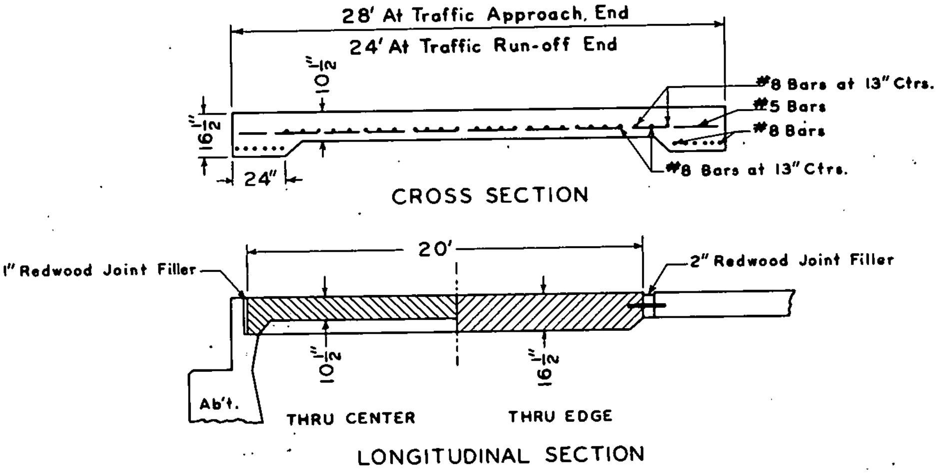


Figure 122. Bridge approach slabs.

7.1.2 Steel Beam Bridges

The steel beam bridges were located at the beginning of the rigid pavement tangents on Loops 5 and 6. Bridges designated 1A, 1B, 2A and 2B were located on Loop 5 and bridges designated 3A, 3B, 4A and 4B on Loop 6. Bridges 9A and 9B were replacements for bridges 4A and 4B, respectively. The location of each bridge with respect to the traffic lanes is given in Table 72. Bridges 1A, 2A, 3A and 4A were located nearest the turnaround and were first in line relative to the direction of test traffic.

Table 72 also gives information pertaining

TABLE 72

STEEL I-BEAM BRIDGES

Bridge No.	Location		Bridge Type	Desired Design Stress (psi)	Beam Size	Length of Cover Plate	
	Loop	Lane [1]				Bottom	Top
1A	5	1	Noncomposite with cover plates	27,000	18WF55	20'–6"	0
1B	5	1	Noncomposite without cover plates	35,000	18WF50	0	0
2A	5	2	Noncomposite without cover plates	35,000	18WF55	0	0
2B	5	2	Composite with cover plates	35,000	18WF50	14'–0"	0
3A	6	1	Noncomposite without cover plates	27,000	21WF62	0	0
3B	6	1	Composite with cover plates	27,000	18WF60	18'–6"	0
4A	6	2	Noncomposite with cover plates	35,000	18WF60	19'–0"	0
4B	6	2	Noncomposite with cover plates	35,000	18WF60	19'–0"	0
9A [2]	6	2	Noncomposite with cover plates	27,000	18WF96	17'–0"	17'–0"
9B [3]	6	2	Noncomposite with cover plates	27,000	18WF96	17'–0"	17'–0"

[1] Lane 1 is inside, Lane 2 is outside lane.
[2] Replacement for bridge 4A.
[3] Replacement for bridge 4B.

to the ten steel I-beam bridges. There were three types of steel beam bridges: noncomposite without cover plates, noncomposite with cover plates and composite with cover plates. Details of the noncomposite and the composite bridges are shown in Figures 123 and 124, respectively.

The slab reinforcement was the same for all ten bridges. It consisted of a layer of longitudinal and transverse deformed bars placed at the top and bottom of the slab. The minimum concrete cover over the steel was 1½ in. (Figs. 123 and 124). The longitudinal reinforcement in each layer was No. 3 bars spaced at 1-ft 8-in. centers and the transverse reinforcement was No. 5 bars at 5-in. centers.

The ends of the slabs of all steel bridges were thickened (Fig. 123) and were in contact with the top surface of the end diaphragms.

In the noncomposite bridges, 3-in. wide concrete haunches were constructed flush with the bottom of the top flanges to provide lateral support to the beams. The top surface of the beams in contact with the slab was coated with a mixture of graphite and linseed oil to inhibit formation of bond.

In the composite bridges, the bottom of the slab was flush with the top surface of the beams. Composite action between the slab and beams was obtained with channel shear connectors welded to the top flanges.

The beams were rolled wide flange steel I-beams. The three beams in each bridge were spaced at 5-ft 0-in. centers (Figs. 123 and 124). The beams for bridge 3A were twenty-one inches deep, while all other steel bridges had 18-in. beams. The beam weights ranged from 50 to 96 lb per linear foot. Table 72 gives the size and weight of the beams for each bridge. The beams for bridge 1B were cambered 3 in. at midspan, and those for the other bridges were cambered 2 in.

The beams for five of the original structures had cover plates on the bottom flange. The beams of bridges 9A and 9B had cover plates on both top and bottom flanges. All cover plates were 6- by 7/16-in. steel plates welded to the I-beam flanges with longitudinal fillet welds

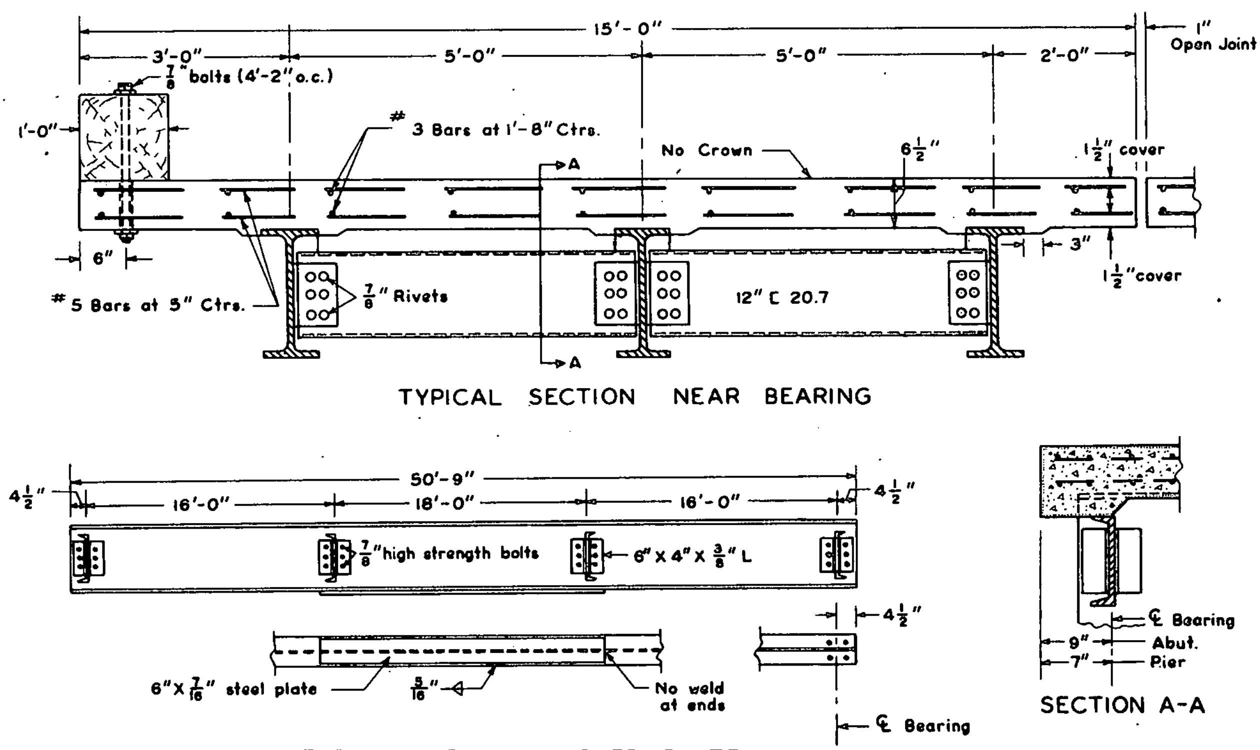


Figure 123. Details of noncomposite steel I-beam bridges.

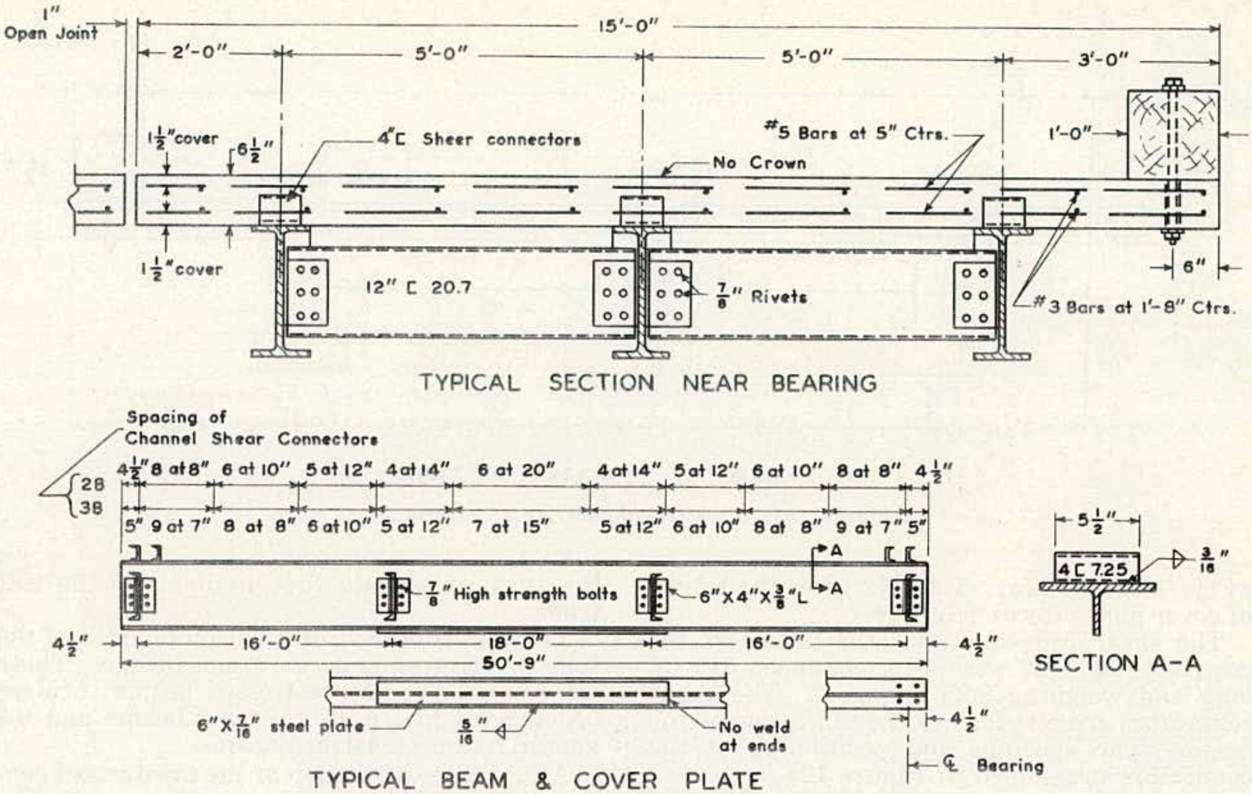


Figure 124. Details of composite steel I-beam bridges.

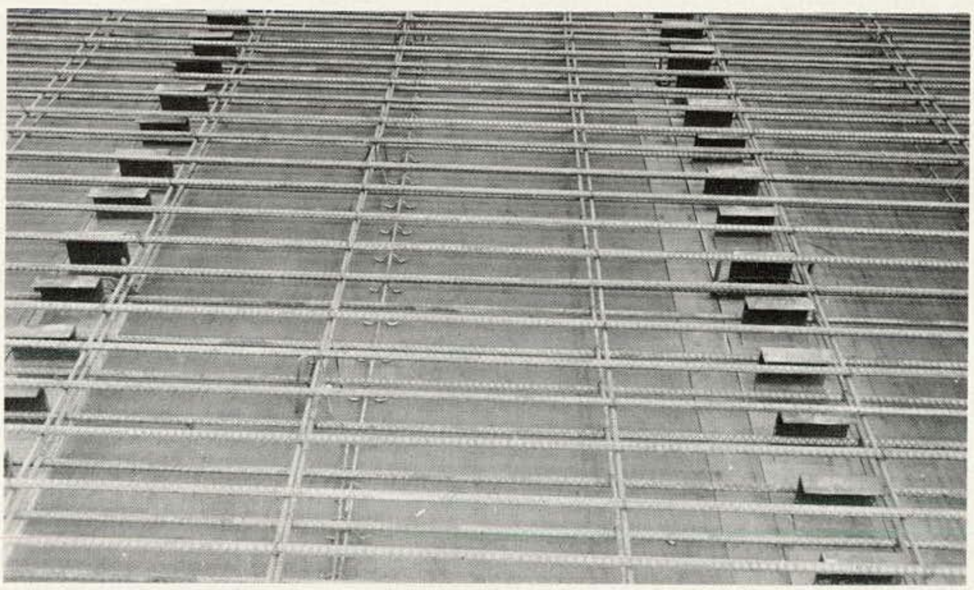
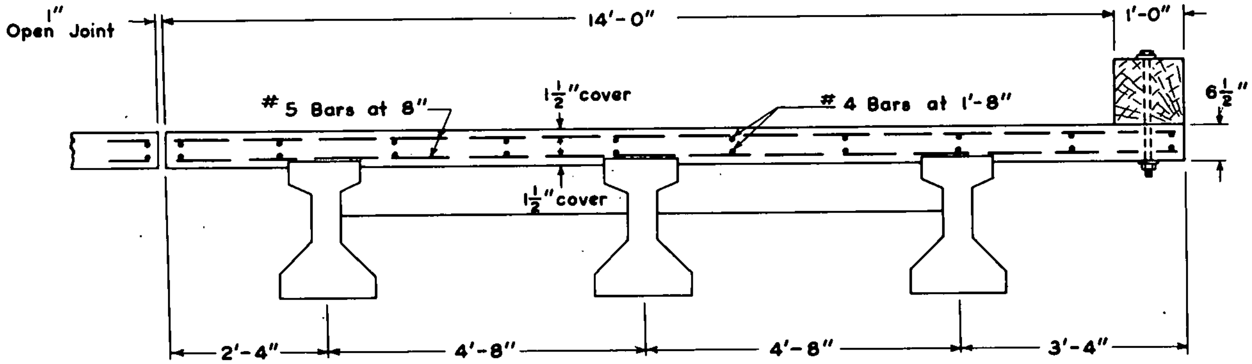


Figure 125. Slab reinforcement and channel shear connectors in composite steel beam bridges.



TYPICAL CROSS-SECTION

Figure 126. Prestressed concrete beam bridges.

(Figs. 123 and 124). Table 72 gives the length of cover plates for each bridge.

The shear connectors for the beams of the composite bridges were 4-in. channels, 5½ in. long and weighing 7.25 lb per ft. The shear connectors were welded to the top flange of the beams. The spacings and positioning of the connectors are shown in Figure 124.

All ten bridges were provided with diaphragms near the third points and at the ends. The diaphragms were fabricated of 12-in. channels and were bolted to the beams with ⅞-in. high-strength bolts.

7.1.3 Prestressed Concrete Beam Bridges

The test bridges constructed with precast prestressed concrete beams were located at the beginning of the flexible pavement tangent (east end) on Loop 5. Bridges 5A and 5B were in the outside lane and 6A and 6B were in the inner lane. Bridges 5B and 6B were nearest to the turnaround and first in line for the test traffic.

Table 73 contains information on each of the four prestressed concrete beam bridges. There were two types of prestressed beams: bridges 5A and 5B had post-tensioned beams and 6A and 6B had pretensioned beams.

A typical cross-section of the prestressed concrete beam bridges is shown in Figure 126. The three beams of each bridge were spaced at 4-ft 8-in. centers.

The reinforcement in the concrete slab of each bridge consisted of a layer of deformed bars placed in the top and bottom of the slab with a minimum concrete cover of 1½ in. The transverse reinforcement in each layer was No. 5 bars on 8-in. centers and the longitudinal reinforcement was No. 4 bars on 1-ft. 8-in. centers. The bottom of the slabe was constructed approximately 1 in. below the top of the concrete beams.

TABLE 73

PRESTRESSED CONCRETE BEAM BRIDGES

Bridge No.	Location		Bridge Type	Initial Prestressing			
	Loop	Lane [1]		Desired Design Stress [2] (psi)	Force [3] (lb)	Elongation [4] (in.)	Number of Prestressing Units [5]
5A	5	2	Post-tensioned	800	47,000	1¾	4
5B	5	2	Post-tensioned	300	52,700	2	6
6A	5	1	Pretensioned	800	13,800	3⅞	16
6B	5	1	Pretensioned	300	14,800	4⅛	20

[1] Lane 1 is inside, Lane 2 is outside lane.
[2] Tension in bottom flange at midspan.
[3] Per prestressing unit.
[4] Elongation at each end for bridges 5A and 5B; total elongation for bridges 6A and 6B.
[5] Tendons of ten parallel wires in bridges 5A and 5B, seven-wire strands in bridges 6A and 6B

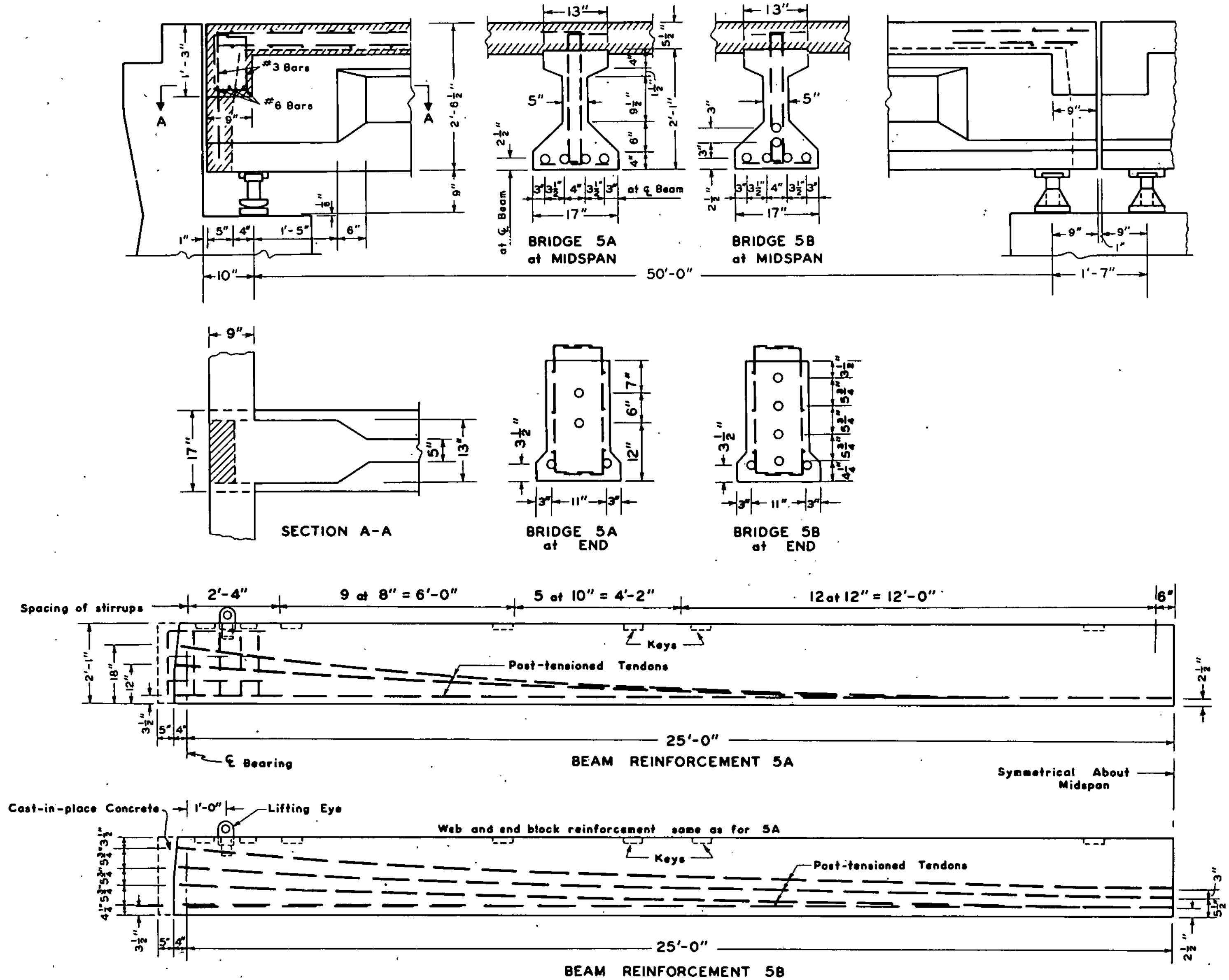


Figure 127. Post-tensioned beams.

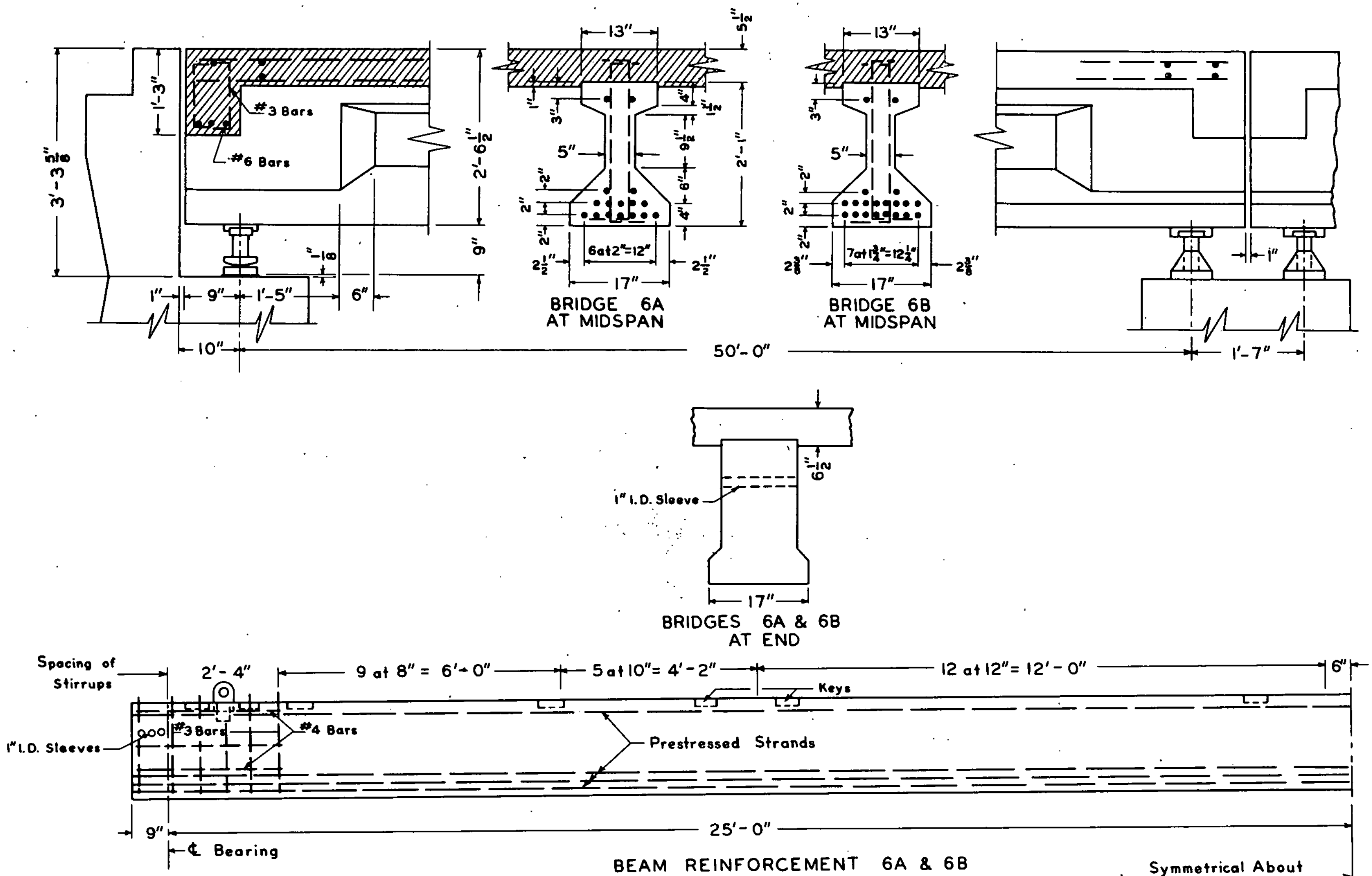


Figure 128. Pretensioned beams.

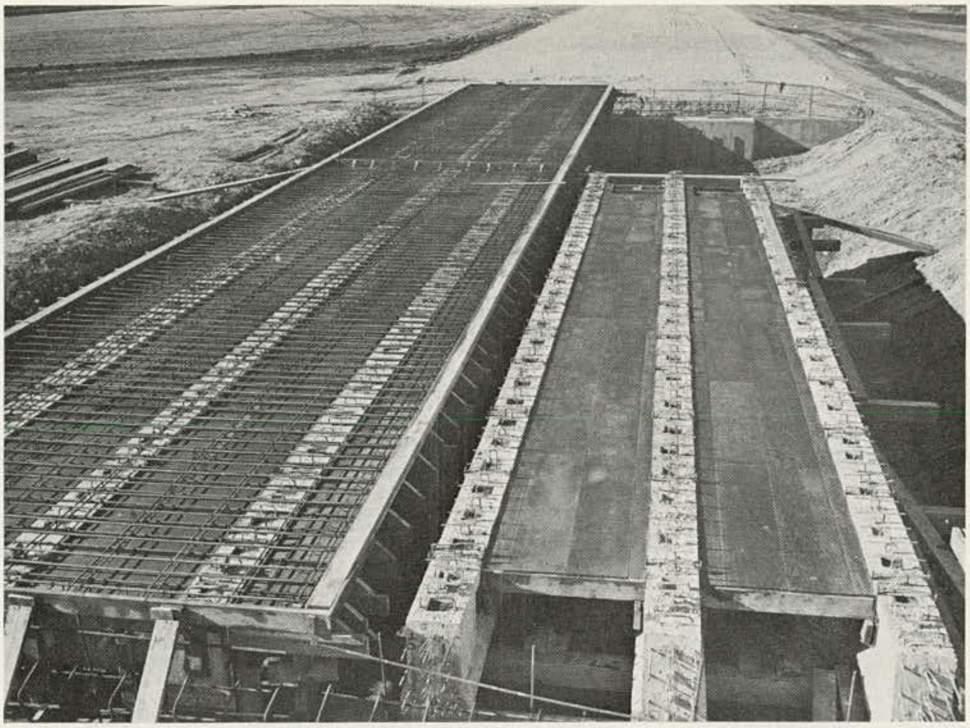


Figure 129. Prestressed concrete beam bridges showing slab reinforcement, recessed keys and extended stirrups.

Details of the post-tensioned and pretensioned beams are shown in Figures 127 and 128. The size and shape of the beams were the same for all bridges. The beams were concrete I-sections with a web thickness of 5 in., a top flange width of 13 in., a bottom flange width of 17 in. and an over-all height of 2 ft 1 in. The web reinforcement, end blocks, diaphragms and connection between the slab and beams also were the same for all bridges.

The web of the prestressed concrete beams was reinforced with No. 3 bar closed vertical stirrups extending above the beams. Recessed keys, 2 by 6 by 6 in., were provided at alternate spaces between the vertical stirrups. The extended stirrups and recessed keys provided

Figure 130. Erecting precast prestressed concrete beams.

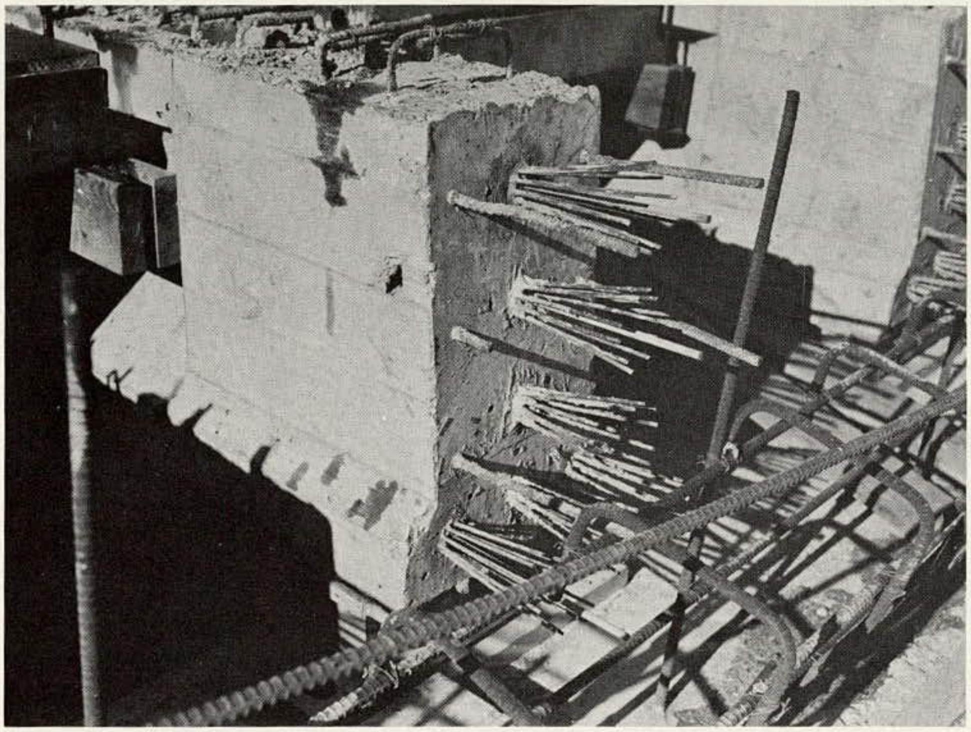


Figure 131. End of post-tensioned beam, Bridge 5B.

connection between the slab and the beams. The top surface of the beams was struck off and left in a rough condition.

All precast concrete beams had end blocks reinforced vertically with stirrups of No. 3 bars and horizontally with No. 4 bars (Figs. 127 and 128). The ends of the post-tensioned beams were covered with end caps.

Concrete diaphragms were provided at each end of the beams. They were reinforced with three No. 6 tension bars, and closed vertical stirrups of No. 3 bars placed at 10-in. centers. The diaphragms and end caps were cast monolithically with the slabs. To connect the diaphragms to the pretensioned beams, three tension bars were passed through 1-in. sleeves in the ends of the precast beams. For the post-tensioned beams, the end caps provided this connection.

The prestressing tendons in the post-tensioned beams consisted of ten 0.192-in. diameter wires enclosed in flexible metal conduits. Four tendons were used in each beam of bridge 5A and six in each of 5B. The tendons were anchored with Freyssinet cone anchorages and the space between the conduit and the prestressed wires was grouted. Table 73 gives the specified force and elongation for the initial prestressing of the tendons.

The prestressing steel of the pretensioned beams consisted of 7-wire strands of ⅜-in. nominal diameter. Sixteen parallel strands were used in each beam of bridge 6A and 20 in each beam of 6B.

Table 73 also gives the specified force and elongation for the initial prestressing of the strands in the beams of bridges 6A and 6B.

TABLE 74

REINFORCED CONCRETE BEAM BRIDGES

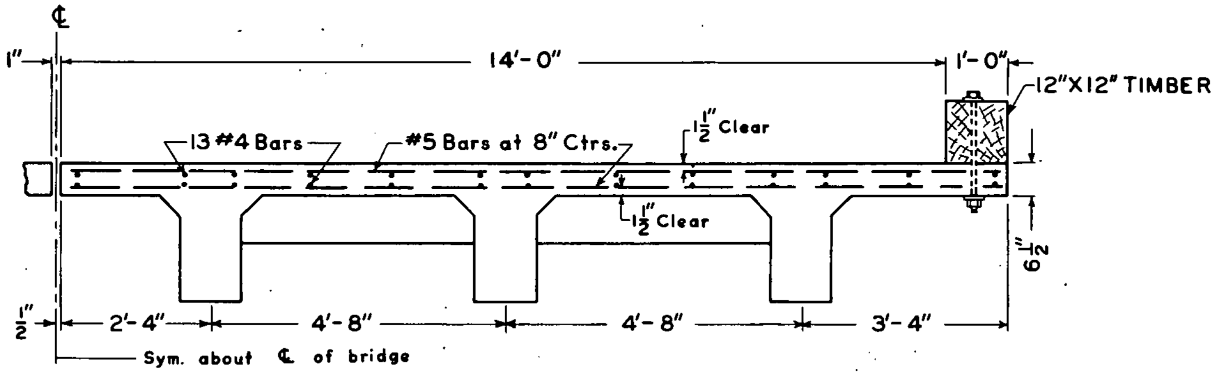
Bridge No.	Location		Desired Design Stress (psi)	Beam Reinforcement	
	Loop	Lane[1]		Bottom Row	Top Row
7A	6	2	40,000	3 No. 11	2 No. 9
7B	6	2	40,000	3 No. 11	2 No. 9
8A	6	1	30,000	3 No. 11	2 No. 9 1 No. 8
8B	6	1	30,000	3 No. 11	2 No. 9 1 No. 8

[1] Lane 1 is inside, Lane 2 is outside lane.

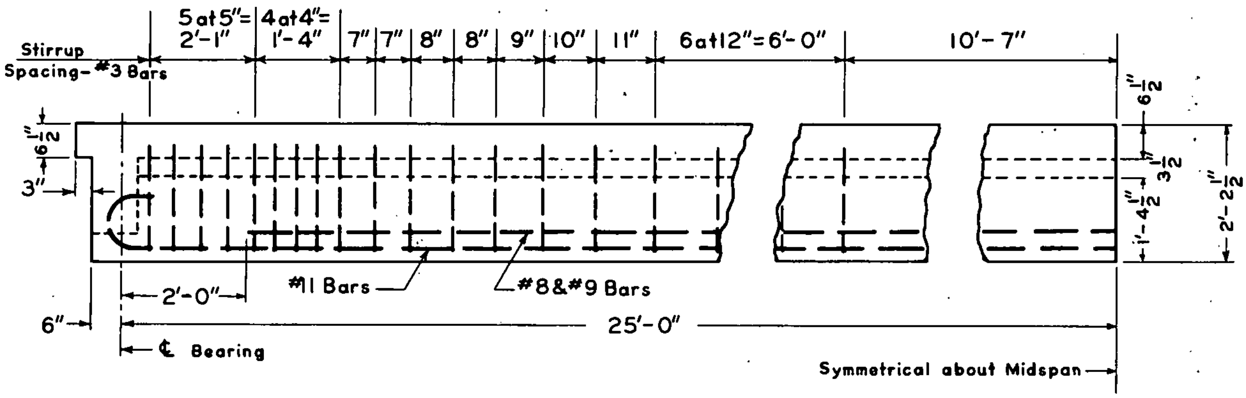
7.1.4 Reinforced Concrete Beam Bridges

The reinforced concrete beam bridges were of conventional T-beam construction. They were located at the beginning of the flexible pavement tangent (east end) on Loop 6. Bridges 7A and 7B were in the outside lane and 8A and 8B were in the inner lane. Bridges 7B and 8B were nearest to the turnaround and first in line for test traffic.

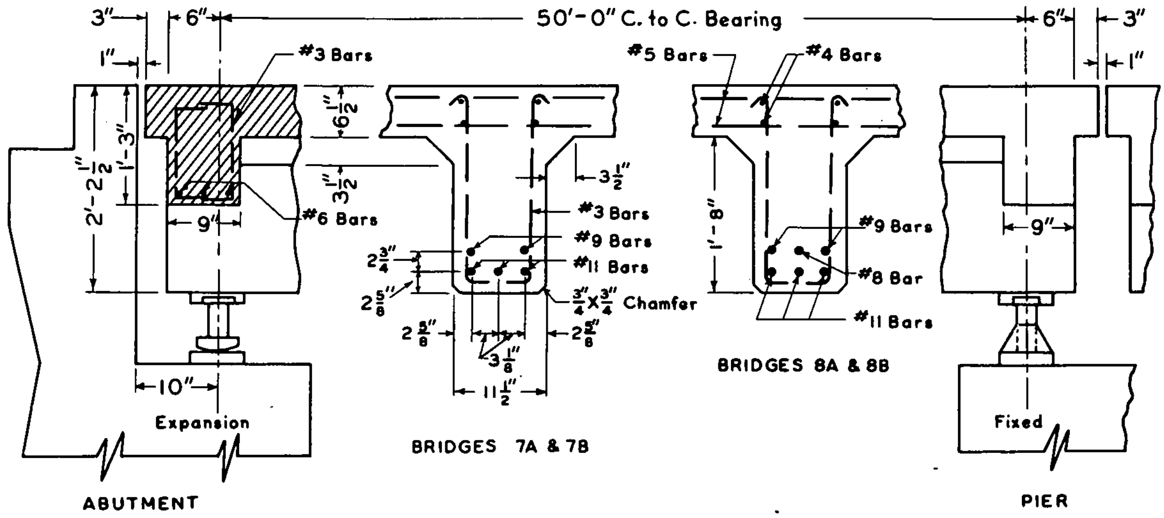
Table 74 gives the location, desired design stress and beam reinforcement for each bridge.



TYPICAL CROSS-SECTION



TYPICAL BEAM REINFORCEMENT



TYPICAL DETAILS

Figure 132. Details of reinforced concrete beam bridges.

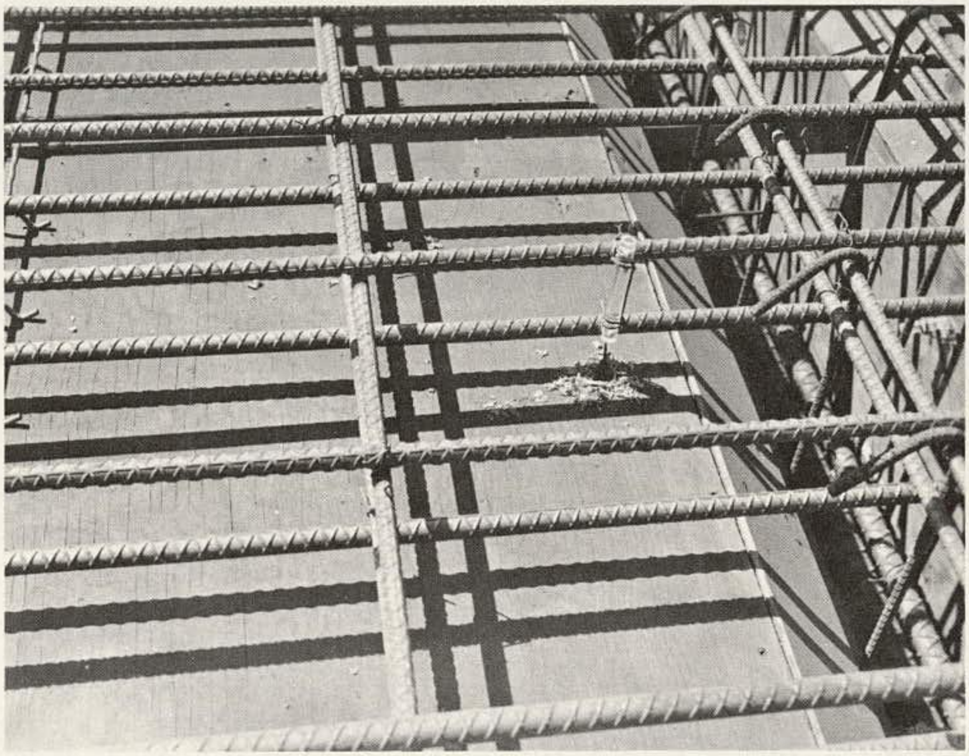


Figure 133. Reinforcement in concrete beam bridges; thermocouple on steel.

TABLE 75

RESULTS OF MILL TESTS OF STRUCTURAL STEEL

Heat No.[1]	Bridge	Beam[2]	Yield Point (psi)	Tensile Strength (psi)	Elongation (%)	Chemical Analysis (%)			
						C	Mn	P	S
74	1A 2A	1, 2, 3 1, 2, 3	40,260	66,340	27.7	0.20	0.65	0.007	0.040
90	1B 2B	1, 2, 3 1, 2, 3	38,190	65,580	30.7	0.21	0.71	0.012	0.040
83	3A	2, 3	41,260	67,490	29.0	0.21	0.62	0.008	0.026
93	3A	1	40,730	64,350	30.5	0.23	0.65	0.009	0.027
42	3B 4A 4B	1, 2, 3 1, 2, 3 2, 3	39,870	66,410	30.5	0.24	0.70	0.010	0.032
86	4B	1	38,660	64,750	30.0	0.20	0.67	0.009	0.030
58	Cover plates—all beams of 1A, 2B, 3B, 4A and 4B		38,080	63,920	31.5	0.23	0.41	0.008	0.030
36	9A 9B	1, 2, 3 1, 2, 3	37,935	64,345	29.9	0.24	0.68	0.012	0.023
85	Cover plates—all beams of 9A and 9B		36,120	62,414	27.9	0.22	0.42	0.014	0.032

[1] Last two digits from mill heat number.
[2] Beams of each bridge were numbered 1, 2 and 3 consecutively from north beam to south beam.

A typical bridge cross-section and details of the concrete beams are shown in Figure 132.

The slab reinforcement was the same for all four bridges. It consisted of two layers of longitudinal and transverse deformed bars placed within the slab so as to provide a minimum concrete cover over the steel of 1½ in. In each layer, the longitudinal reinforcement consisted of 13 No. 4 bars and the transverse reinforcement consisted of No. 5 bars spaced at 8-in. centers.

The three beams of each bridge were 11½ in. wide, 1 ft 8 in. deep below the bottom of the slab, and were spaced at 4-ft 8-in. centers. Concrete diaphragms, reinforced with three No. 6 tension bars and closed vertical stirrups of No. 3 bars on 10-in. centers, were provided at the ends of each bridge. The diaphragms were cast monolithically with the beams and the slab.

The concrete beams were reinforced in tension with two layers of deformed bars. Three No. 11 bars were used in the bottom layer and two No. 9 bars were used in the top layer of all concrete beams. In addition, one No. 8 bar was used in the top layer of each beam of bridges 8A and 8B. All bars had diamond-shaped deformations.

The web reinforcement consisted of vertical stirrups of No. 3 bars. The stirrups were omitted from the center portion of each beam (see Fig. 132).

7.2 MATERIALS AND MATERIALS CONTROL

7.2.1 Steel

The steel used in the test bridges included structural steel, prestressing steel and reinforcing steel. In general, the material requirements were representative of current practice. However, the specifications contained certain requirements aimed at obtaining steel with exceptionally uniform physical properties for the primary load-carrying elements.

Structural steel was required to comply with ASTM Designation: A7-55T. The structural steel for beams and cover plates was required to have physical properties (yield point, tensile strength and elongation) as close to the minimum specified values as possible but not below them. It was required that mill test reports showing the chemical analysis and physical properties be furnished for each heat used for beams and cover plates, and that each beam and cover plate be clearly marked to indicate its heat number, bridge number and location within the bridge.

Table 75 gives the results of the mill tests on the structural steel. The beams and cover plates for the ten steel beam bridges were rolled from nine separate heats. Except for 3A and 4B, the three beams in any one bridge were rolled from a single heat. All cover plates for the original steel beam bridges were rolled from a single heat. The cover plates for bridges 9A and 9B also were rolled from a single heat.

The prestressing steel for the post-tensioned beams was required to be high-tensile wire of 0.192-in. nominal diameter with a minimum yield strength at 0.2 percent offset of 200,000 psi. That for the pretensioned beams was required to be high-tensile 7-wire strand with a ⅜-in. nominal diameter and a 250,000-psi minimum ultimate strength of wire. The diameter of the wire was not permitted to vary by more than ± 0.003 in. from the nominal value and the elongation in 10 in. at rupture could not be less than 3 percent.

Results of the prestressing steel acceptance are given in Table 76. Two shipments of steel wire were used for the post-tensioned beams. The first shipment was used for two beams of bridge 5A and all three beams of 5B. The prestressing steel for beam 1 of bridge 5A was obtained from the second shipment. Two spools of wire strand were used for the pretensioned beams. All beams for bridge 6A were made with the strand from the first spool and those

TABLE 76

RESULTS OF ACCEPTANCE TESTS ON PRESTRESSING STEEL

Type	Bridge	Beams [1]	Wire Diameter (in.)	Yield Strength (psi)	Tensile Strength (psi)
Steel wire for post-tensioning tendons	5A 5B	2, 3 1, 2, 3	0.191	216,400	262,480
	5A	1	0.192	234,890	252,500
Steel strand (7 wire) for pretensioning ..	6A	1, 2, 3			273,220[2]
	6B	1, 2, 3			280,980[2]

[1] Beams of each bridge were numbered 1, 2 and 3 consecutively from beam nearest centerline of roadway to exterior beam.
[2] Based on cross-sectional area of 0.0799 sq in.

TABLE 77

RESULTS OF MILL TESTS ON REINFORCING STEEL

Heat No.[1]	Bar Size	Yield Point (psi)	Tensile Strength (psi)	Elongation (%)	Chemical Analysis (%)			
					C	Mn	P	S
89[2]	3	57,910	83,550	20.0	0.37	0.60	0.023	0.030
	4	56,670	87,270	18.0				
	5	46,610	75,300	19.0				
	6	50,390	83,850	18.0				
	8	55,600	82,450	22.0				
	9	53,820	81,950	21.0				
	11	49,600	79,250	24.0				
11[3]	3	48,100	71,230	25.0	0.40	0.46	0.017	0.020
47[3]	4	47,880	82,010	19.0	0.40	0.44	0.012	0.033
34[3]	4	45,260	74,320	23.0	0.35	0.47	0.010	0.024
09[4]	3	51,900	77,100	20.3	0.28	0.45	0.013	0.056
03[4]	5	48,050	72,050	23.4	0.32	0.43	0.018	0.056

[1] Last two digits from mill heat number.
[2] Bars for reinforced concrete beam bridges and for concrete slabs of all original bridges.
[3] Bars for prestressed concrete beams.
[4] Bars for slabs of bridges 9A and 9B.

for bridge 6B were made with that from the second.

Deformed bars of intermediate grade billet-steel conforming to the requirements of AASHO Designation: M31-56 (ASTM A15-54T) were specified for the concrete reinforcing bars of the test bridge superstructures. The deformations of the bars were required to conform to AASHO Designation: M137-55 (ASTM A305-53T). Furthermore, the bars for the reinforced concrete beam bridges, the slabs of the prestressed concrete beam bridges and the slabs of the original eight steel beam bridges were required to be rolled from a single heat. A mill test report giving the chemical analysis and physical properties of the heat was to be furnished.

Table 77 gives the results of the mill tests for the reinforcing steel. The table includes the mill test data for all concrete reinforcing bars used in the superstructures of all test bridges.

The specifications for the reinforcing bars in the substructures permitted structural grade conforming to ASTM A15-54T or rail steel conforming to ASTM Designation A16-54T.

7.2.2 Concrete

The specifications for the materials used in the concrete of the substructures were in accordance with the standard specifications of the State of Illinois, and material control was representative of normal construction practice. Class A concrete was used for the piers and for the portion of the abutments below the bridge seat. Class X concrete was used in abutments above the bridge seat. The design of the two mixes was identical except for the gradation of the coarse aggregate. The cement factor was 1.50 bbl per cu yd, the water-cement ratio was 4.9 gal per bag, and the aggregates were gravel and natural sand both obtained from Ottawa, Ill. The maximum size of the gravel was 2½ in. for the class A concrete and 1½ in. for the class X concrete.

The specification requirements for the control of the materials in the concrete of the superstructures were more rigid in an effort to obtain uniform physical characteristics in each bridge. Concrete mixes with 3,000-, 4,000-, and 5,000-psi nominal strengths were specified for the superstructures. The 3,000-psi concrete was used in the reinforced concrete beam bridges and in the slabs of the prestressed concrete bridges, the 4,000-psi concrete was used in the slabs of the steel beam bridges, and the 5,000-psi concrete was used in the prestressed beams. In the concrete for the slab of bridge 1B, steel punchings replaced by volume a portion of the coarse aggregate to increase the unit weight of concrete to approximately 160 lb per cu ft. The design criterion for this bridge was revised, and the steel punchings were added to increase the dead load.

The 3,000- and 4,000-psi concrete mixes were designed assuming a 10 percent coefficient of variation in 28-day compressive strengths and permitting 1 test specimen in 15 to fall below the specified strength. The 5,000-psi mix was the standard used by the State of Illinois for prestressed concrete bridge beams. The concrete mix data are summarized in Table 78.

The specifications required that the cement, aggregate and water be properly proportioned and mixed at the location of the work. The 3,000- and 4,000-psi concrete were proportioned and mixed at the project site, and the 5,000-psi concrete for the prestressed beams was produced at the site of the prestressing beds in Springfield, Ill.

TABLE 78

CONCRETE MIX DATA

Concrete Design Mix (psi)	Cement Factor (bags/ cu yd)	Total Water (gal/ bag)	Abs. Vol. of Agg. (cu ft/bag)			Mortar (cu ft/ bag)	Yield (cu ft/ bag)	Prop. Wt. of Agg. (lb/bag)		
			Sand	Coarse Agg.	Steel Punch-ings			Sand	Coarse Agg.	Steel Punchings
3,000	5.23	6.0	1.30	2.37	0.0	2.79	5.16	218	398	0
4,000	6.67	4.9	0.91	1.85	0.0	2.20	4.05	153	310	0
5,000	6.67	4.9	0.91	1.722	0.125	2.20	4.05	153	289	60.8
	7.00	4.6	1.01	1.62	0.0	2.26	3.88	164.7	269.8	0

The specifications permitted the use of either gravel, crushed gravel or crushed stone for the coarse aggregate and required the use of natural sand for the fine aggregate, but stipulated that each be obtained from a single source and plant. As with the aggregates in the portland cement concrete pavement, the gradation limits were considered only as maximum and minimum values and the gradations of both the coarse and fine aggregates were controlled within plus and minus tolerances of approved gradation formulas.

The coarse aggregate for the concrete mixed at the project site (3,000- and 4,000-psi concrete) was crushed gravel, predominantly limestone, consisting of approximately 30 percent crushed material. The fine aggregate was a natural bank sand, mainly siliceous. Both the gravel and the sand for this concrete were obtained from a single source located at Sheridan, Illinois. Materials from this source also were used in bridges 9A and 9B constructed in 1959.

The gradations of the aggregates for the 3,000- and 4,000-psi concrete are shown in Table 79.

The coarse aggregate for the 5,000-psi concrete in prestressed beams was a crushed limestone obtained from East St. Louis, Ill. The fine aggregate was a natural sand, mainly siliceous, obtained from Springfield, Ill. Table 80 gives the specification requirements and the results of gradation tests on these aggregates.

TABLE 79

AGGREGATES FOR 3,000- AND 4,000-PSI CONCRETE (SUMMARY OF 35 TESTS ON COARSE AGGREGATE AND 14 TESTS ON FINE AGGREGATES)
(Obtained from Data System 2400)

Sieve	Specification Requirements		Test Results	
	Spec. Limits (% Passing)	Formula and Tolerances	Mean (% Passing)	Standard Deviation
	(*a*) COARSE AGGREGATE			
1½ in.	100	100	100	....
1 in.	60–95	80 ± 5	81	3.28
½ in.	10–30	25 ± 5	25	3.21
No. 4	0–5	0 – 5	0.3	0.10
	(*b*) FINE AGGREGATE			
⅜ in.	100	100	100	....
No. 4	95–100	95–100	100	....
No. 8	80–95	85 ± 5	87	2.06
No. 16	45–80	76 ± 4	75	4.18
No. 30	25–60	56 ± 4	56	3.87
No. 50	10–30	13 ± 3	14	1.73
No. 100	0–10	3 ± 2	2.6	0.47

TABLE 80

AGGREGATES FOR 5,000-PSI CONCRETE (SUMMARY OF 9 TESTS ON EACH SIZE AGGREGATE)
(Obtained from Data System 2400)

Sieve	Specification Limits (% Passing)	Test Results	
		Mean (% Passing)	Standard Deviation
(a) COARSE AGGREGATE			
1 in.	100	100	0.43
½ in.	25–50	39	3.38
No. 4	0–10	2.8	1.65
(b) FINE AGGREGATE			
⅜ in.	100	100	....
No. 4	95–100	97	1.54
No. 8	80–95	88	4.39
No. 16	45–80	75	4.44
No. 30	25–60	54	4.92
No. 50	10–30	19	3.04
No. 100	0–10	2.2	0.35

The specifications required the use of Type I portland cement and an air-entraining agent of either Darex or neutralized Vinsol resin for all concrete in the superstructures. The Type I portland cement for the 5,000-psi concrete was obtained at Cape Girardeau, Mo., and tests indicated a mean autoclave expansion of 0.24 percent. That for the 3,000- and 4,000-psi concrete was obtained at Oglesby, Ill., with a mean autoclave expansion of 0.08 percent. Darex was used as the air-entraining agent for all mixes.

7.3 CONSTRUCTION PROCEDURES AND CONTROLS

7.3.1 Construction Sequence

The construction of the test bridges began in October 1956 with the excavation for the substructures and the construction of the footings, piers and abutments. Conventional construction procedures and controls were used for this portion of the work. The substructures at the four locations were completed in the spring of 1957.

The superstructures of the original eight steel beam bridges and the four reinforced concrete beam bridges were completed during the 1957 construction season. The concrete slabs for the steel bridges were cast during the first half of August, and the concrete beam bridges were cast during August and the first week of September.

The fabrication of the prestressed concrete beams was started in July but was discontinued because of difficulties with the prestressing equipment. This work was resumed at the end of September and completed by the end of November. The concrete slabs for bridges 6A and 6B were cast at the end of November. The post-tensioned beams for bridges 5A and 5B were in place and the forms for the slabs were substantially complete by the end of the 1957 construction season. The slabs for these two bridges were cast in April 1958.

The construction of the bridge approach slabs for the reinforced concrete beam bridges was started in the fall of 1957 and was completed the following spring. The bridge approach slabs for all remaining test bridges were constructed in the spring of 1958.

Bridges 9A and 9B were constructed in the spring of 1959. The slabs were cast at the end of May, and on June 19 the existing bridges were removed and the new bridges were set in place.

Significant dates of construction of the test bridges are given in Table 81, including the dates of casting concrete in bridge decks and completion of approach slabs for each bridge, and the dates of casting and prestressing beams for the prestressed concrete beam bridges.

7.3.2 Off-Site Fabrication of Beams

The steel beams and cover plates for the original eight steel beam bridges were rolled 2 ft longer than required by the bridge dimensions. Two-foot coupons were cut off at the fabricating shop and identified to indicate the heat number and the beam or plate number. Coupons also were provided from the beams and cover plates of the additional bridges, 9A and 9B, but the required length of the coupons for the steel beams was increased to 9 ft. The coupons were delivered to the job-site to provide material for tension tests.

The precast prestressed concrete beams were manufactured at Springfield, Ill. Plywood forms were used. The pretensioned beams were made on a 150-ft prestressing bed permitting two beams to be cast at a time. The two outside beams of a bridge were made simultaneously; the center beam was cast alone. The 7-wire prestressing strands were stressed one at a time. The load on each strand was measured by a calibrated load cell, and the elongation by a ruler. An initial load was applied to the strand, reference marks were stablished for measuring the elongation in 140 ft, and the load was increased to obtain the required stress level. Each strand was over-stressed a sufficient amount to compensate for the slippage of the vice grips. The stress was released 3 to 6 days after casting. At the time of release, the strength of concrete had reached at least 4,000 psi.

Table 82 gives the initial tensions and elongations after vice slippage for the pretensioned strands of bridges 6A and 6B.

TABLE 81

RECORD OF TEST BRIDGE CONSTRUCTION

Bridge No.	Date of Casting Concrete Slab	Completion Date[1]	Bridge No.	Beam No.	Prestressed Concrete Beams: Date of Casting	Prestressed Concrete Beams: Date of Stressing
1A	8–15–57	6–17–58	5A	1	11–22–57	11–25–57
1B	8– 1–57			2	10–24–57	11– 4–57
2A	8– 1–57			3	11–21–57	11–25–57
2B	8–15–57					
3A	8– 7–57	5–21–58	5B	1	7–27–57	11– 1–57
3B	8–13–57			2	11– 9–57	11–12–57
4A	8–13–57			3	11– 9–57	11–13–57
4B	8– 7–57					
5A	4–16–58	5–15–58	6A	1	9–28–57	10– 1–57
5B	4–16–58			2	10–16–57	10–22–57
6A	11–27–57			3	9–28–57	10– 1–57
6B	11–27–57					
7A	8– 2–57	4–17–58	6B	1	10– 4–57	10– 7–57
7B	9– 6–57			2	10–10–57	10–14–57
8A	9– 6–57			3	10– 4–57	10– 7–57
8B	8– 2–57					
9A	5–26–59	6–19–59				
9B	5–26–59					

[1] Includes completion of bridge approach slabs for bridges 1A through 8B.

Each post-tensioned beam was manufactured independently. The beams were stressed after the concrete had reached a strength of at least 4,000 psi. The 10-wire tendons were prestressed with two double-acting Freyssinet jacks, one placed at each end of the beam. The load was measured by a calibrated pressure gage, and the elongation with a ruler.

The friction between the conduits and the wires was determined by a trial procedure for each tendon with the aid of the following formula:

$$F_t = 2\left(F_1 - \frac{aeE}{d}\right)$$

in which

F_t = total friction loss;
F_1 = observed tension at the jack;
a = cross-sectional area of the prestressing element;
e = observed elongation of the element at the jack when the force at the jack is F_1;
E = secant modulus of elasticity of the element for the stress $\frac{F_1}{a}$ as determined from the stress-strain diagram of the element (28.6×10^6 psi); and
d = distance from the jack to the midspan of the beam.

TABLE 82

RESULTS OF PRESTRESSING FOR PRETENSIONED BEAM BRIDGES

Bridge No.	Strands per Beam	Beam No.	Initial Tension (ksi): Mean	Initial Tension (ksi): Std. Dev.	Elongation[1] (in.): Mean	Elongation[1] (in.): Std. Dev.
6A	16	1	169.6	3.80	9 15/16	1/16
		2	171.2	3.53	8 3/4	1/8
		3	169.6	3.80	9 15/16	1/16
6B	20	1	185.1	0.87	9 5/8	1/4
		2	183.9	0.96	9 11/16	1/8
		3	185.1	0.87	9 5/8	1/4

[1] Elongation in 140 feet measured from an initial load of 2,000 lb to the final load, except that the initial load for beams 1 and 3 of Bridge 6A was 400 lb.

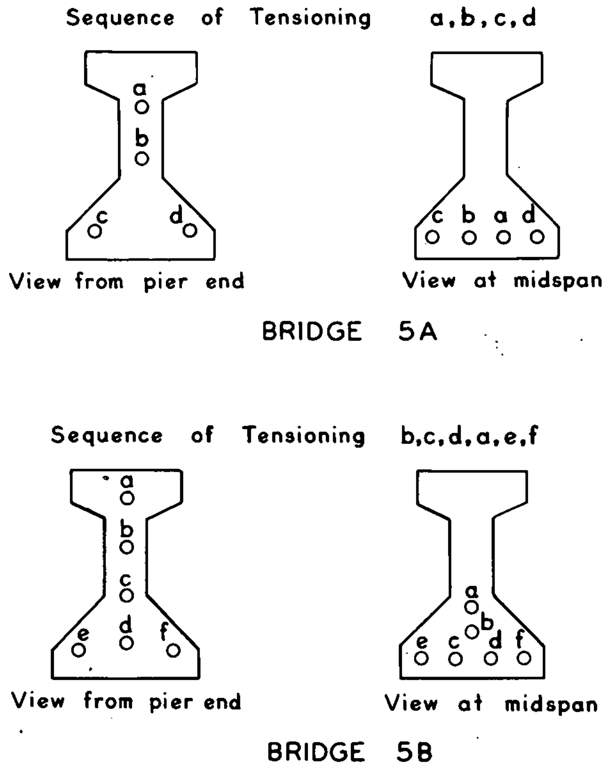


Figure 134. Tendon locations within post-tensioned beams.

An initial force of 500 psi on the gage was applied to each end of a tendon and reference marks were established for measuring the elongation in 52 ft 4 in. The force was then increased to a total force F_1 computed on the basis of an assumed friction loss. The elongation (e) was measured, and the friction loss was computed from the equation. The friction loss so computed was then taken as a basis for recomputation of the force F_1. The recomputed total force was then applied and the process was repeated until the assumed and computed friction losses were identical. Finally, the tendon was over-stressed sufficiently to compensate for the anticipated slippage of the anchorage device. It was then anchored at both ends and the jacks were removed.

The results of tensioning the post-tensioned beams are given in Table 83. Figure 134 shows the locations of the tendons within the beams and the tensioning sequence.

After the post-tensioning of all three beams for a bridge was completed, the space between the steel conduit and wires in each tendon was filled with a neat cement grout made of Type I portland cement and 5 gal of water per bag. The grout was pumped from one end of the beam until a continuous flow was observed at the other end. The free end was then plugged and grouting was continued until refusal.

The 5,000-psi concrete used in the prestressed beams was mixed at the site of the prestressing beds. Approximately ¾ oz of Darex per bag of cement was added to control the air content of the concrete to within 3 to 5 percent. Consolidation of the concrete within the beam forms was obtained by internal vibration. The concrete beams were steam cured for 12 to 84 hr. The maximum temperature was 110 F. Due to low temperatures, the post-tensioned beams were steam cured for an additional 24 hr after grouting.

Control tests for slump and air content were made on each batch of concrete used in the prestressed beams. The average slump and air content were 1⅝ in. and 3.9 percent, and the corresponding standard deviations were ½ in. and 0.5 percent.

Concrete test specimens were cast and cured with the corresponding prestressed beams. Eighteen 6- by 12-in. cylinders, ten 6- by 6- by 30-in. beams and two 6- by 6- by 64-in. beams were made for each prestressed beam. In addition, four 3- by 3- by 11-in. concrete beams were molded during the manufacture of the prestressed beams for the evaluation of the thermal coefficient of expansion of the concrete.

7.3.3 On-Site Construction

The specifications governing the construction of the superstructures for the original 16 test bridges varied from those normally used in bridge construction only in the requirements for forms and falsework, and for mixing and finishing of the concrete.

Plywood forms were used for the construction of the concrete deck slabs. The specifications required that the forms and falsework for any one bridge be independently supported. Concrete footings or mud sills used to support falsework bents in any one of the reinforced concrete bridges had to be separate and independent from those used for the adjacent bridge.

Forms for the concrete decks of the steel and prestressed concrete beam bridges were supported from the beams. The plywood decking and floor joists were supported by stringers placed along each side of the beams and along the edges of the deck. The stringers along each side of a beam were suspended from the top flange by adjustable steel hangers. Those along the edges of the deck were supported by 6- by 8-in. beams. The beams were 16 ft 0 in. long, spaced on 8-ft 0-in. centers (6-ft 0-in. centers for bridges 9A and 9B), and were suspended from the two outside bridge beams by steel hangers. Figure 135 shows details of the forms and falsework for the steel and prestressed concrete beam bridges.

The forms for the deck and beams of each reinforced concrete beam bridge were supported on falsework bents placed near the abutment and pier, and at the quarter points and mid-

TABLE 83

RESULTS OF PRESTRESSING FOR POST-TENSIONED BEAM BRIDGES

Bridge No.	Beam No.	Tendon Designation	Initial Tension at Midspan[1] (ksi)	Friction Loss (ksi)	Total Elongation After Anchorage Slip (in.)
5A	1	a	159.0	16.4	3.67
		b	155.7	17.7	3.61
		c	156.3	16.4	3.61
		d	156.7	15.0	3.61
	2	a	158.1	18.4	3.67
		b	152.9	23.2	3.61
		c	156.3	16.4	3.61
		d	155.7	11.6	3.55
	3	a	155.6	23.2	3.70
		b	157.7	19.1	3.67
		c	157.0	8.5	3.55
		d	151.9	19.8	3.58
5B	1	a	162.4	21.2	3.80
		b	168.9	8.2	3.80
		c	166.2	19.1	3.86
		d	171.3	3.4	3.80
		e	163.5	18.4	3.80
		f	158.4	23.2	3.73
	2	a	163.8	18.4	3.80
		b	167.9	15.7	3.86
		c	167.6	22.5	3.92
		d	164.5	22.5	3.86
		e	163.5	12.3	3.74
		f	160.4	13.7	3.66
	3	a	163.5	19.1	3.80
		b	170.6	12.3	3.89
		c	168.3	26.3	3.99
		d	165.5	20.5	3.86
		e	162.8	13.7	3.74
		f	163.1	13.7	3.67

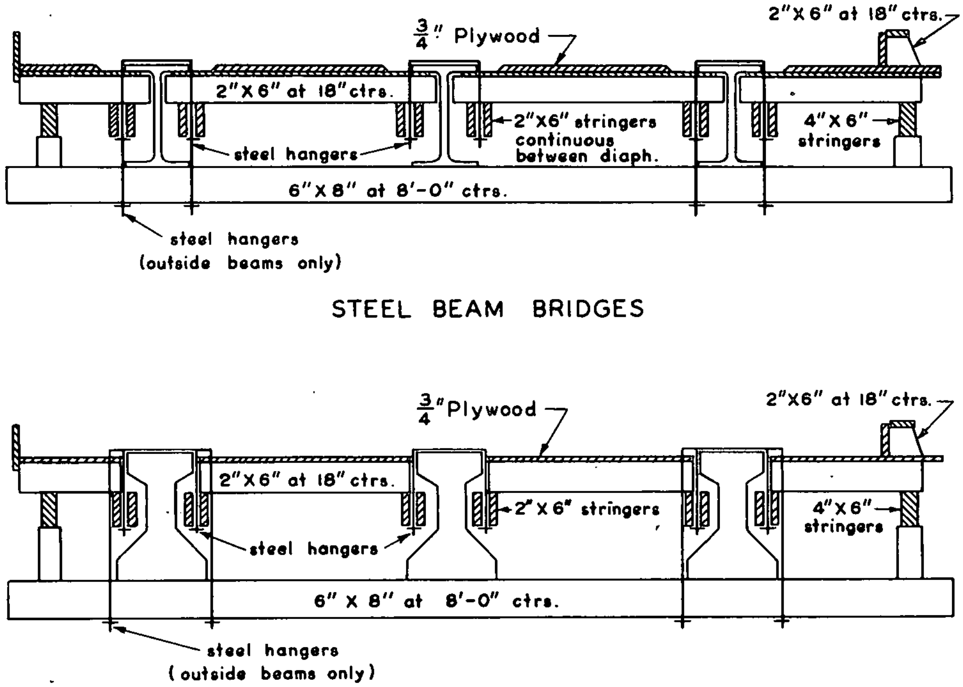
[1] Does not include decrease in stress caused by subsequent stressing of other strands; see Figure 134 for sequence of tensioning.

span. Details of the forms and falsework are shown in Figure 136. As required by the specifications, a screw jack was placed on the top of each bent to permit adjustments for any settlement in the formwork that might occur before or during the placement of the concrete.

The specifications required two mechanical mixers at the site of the work, one of which served as a standby unit in case of a mechanical failure. As specified, concrete placement for the deck slabs began at midspan and progressed simultaneously in both directions. Consolidation of the concrete by either internal or external vibration was not permitted except that internal vibration was required along the vertical edges of all forms and within the beams of the reinforced concrete beam bridges. Consolidation of the concrete around reinforcing bars and shear connectors was accomplished by hand spading. Strike-off of the concrete required the use of a non-vibrating longitudinal screed supported on metal screed strips set transversely across the bridge at the third-points of the span. The concrete in a deck was struck off transversely across the slab in one-third-span lengths starting with the center one-third. Final finish was obtained with two passes of a double-thickness burlap drag.

The concrete in the slabs of all bridges except 6A and 6B was moist cured for 7 days with two layers of wetted burlap blanket covered with a waterproof paper. The slabs of prestressed bridges 6A and 6B were winter-cured for 12 days, using a layer of insulating material over the top surface, a canvas enclosure along the edges of the forms, and heating units beneath the structures. The forms and falsework of each bridge were removed after the concrete had attained the specified strength, but not before 14 days after casting the slab.

The specifications required that the top surfaces of the steel beams for non-composite bridges be treated with a mixture of graphite and linseed oil. A special experiment with bond preventing agents was conducted before this work was started, and the results indicated that a mixture of one part graphite to 4.43 parts linseed oil by weight was satisfactory. Two coats of this mixture were applied to the beams. The minimum drying period between coats was



PRESTRESSED BEAM BRIDGES

Figure 135. Forms for bridge decks.

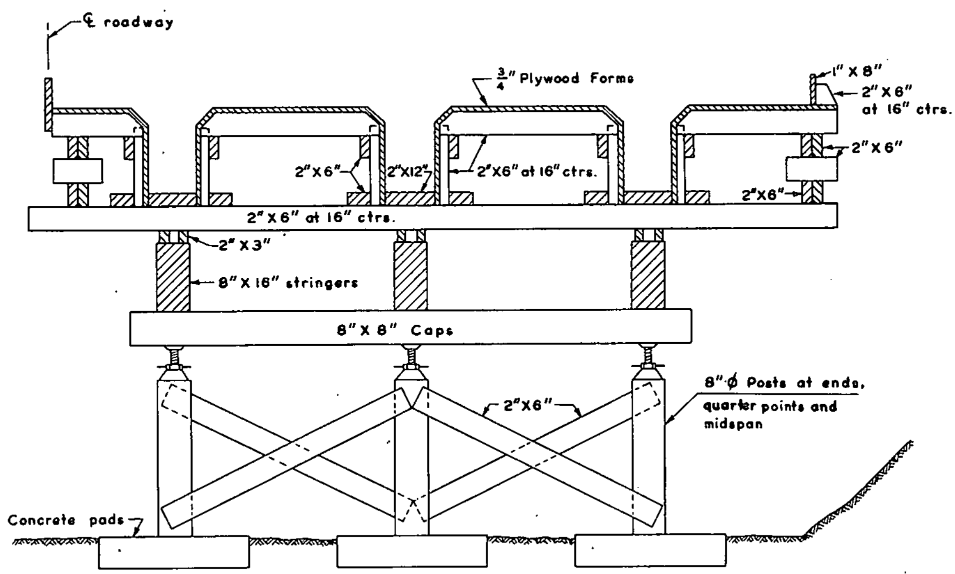


Figure 136. Forms and falsework for reinforced concrete beam bridges.

Figure 137. Casting bridge slab.

24 hr, and the second coat was applied at least 3 days before casting the concrete slab.

The sand and coarse aggregate for the 3,000- and 4,000-psi concrete mixed at the project site were proportioned by weight in 5- to 7-bag batches. The Type I portland cement was added at the mixer by bags. Approximately ¾ oz of Darex was used per bag of cement. Control tests for slump and air content were made on each batch of concrete placed in the beams of the reinforced concrete bridges, and on the first three batches and every succeeding third batch placed in the slab of each test bridge. The average slumps for the 3,000- and 4,000-psi mixes were 3⅝ and 4⅛ in., and the corresponding standard deviations were 1⅛ and 7⅛ in.

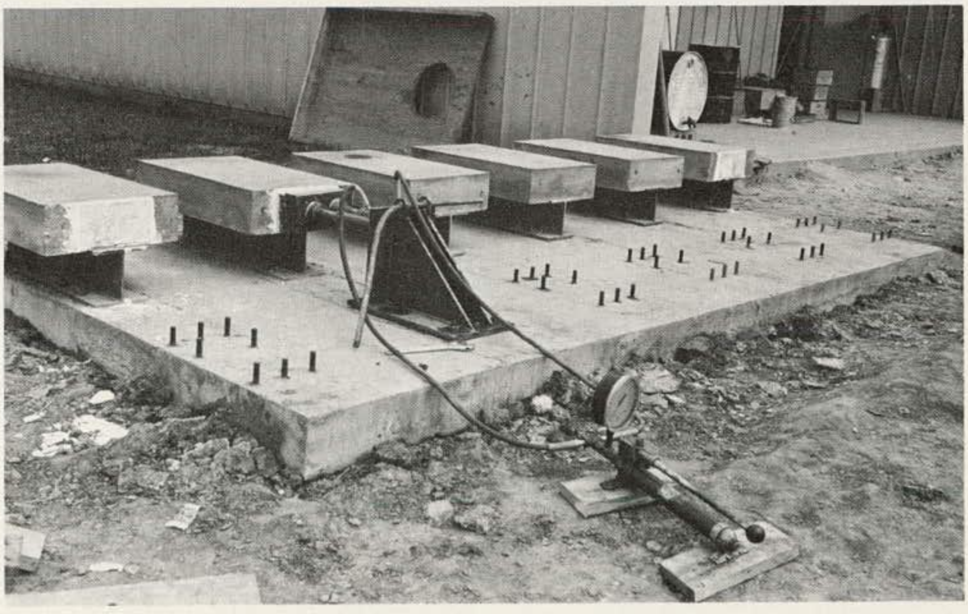


Figure 138. Test of bond-preventing agents.

Figure 139. Preparing concrete test specimens.

The average air contents for the two mixes were 4.3 and 3.5 percent, and the standard deviations were 0.7 and 0.6 percent.

Concrete cylinders (6 by 12 in.) were molded during the placing of concrete, and were cured with the corresponding bridge slabs. Thirty to 36 cylinders were made from the concrete used in the beams of each reinforced concrete bridge, and 18 were made for each slab of all bridges. In addition, two 6- by 6- by 30-in. beams were molded for each bridge to determine the time of removal of forms and falsework.

The procedures and controls for the construction of the two additional steel beam bridges were identical to those used for the original eight structures except that the addi-

Figure 140. Jacking bridges 9A and 9B into place on existing bearings.

tional bridges were constructed on temporary supports adjacent to the bridges they were to replace and a vibrating screed was used for the strike-off of the concrete. Constructing the bridges on temporary supports was required to reduce to a minimum the interruption to the test traffic on the loop. Steel rails and jacks were used to slide the new superstructures onto the existing substructure and to lower them into place on the existing steel bearings.

7.4 PHYSICAL PROPERTIES OF BRIDGE MATERIALS

A comprehensive sampling and testing program was conducted to determine the physical characteristics of the materials used in the superstructures of the test bridges. The program included the determination of static properties, fatigue strength, long-time characteristics and residual stresses. This section reports only the static tests. The fatigue and long-time tests are described in separate reports.* The results of the residual stress study is included in Appendix E.

The results of the static tests were subjected to variance analyses and to significance tests at the 5 percent and 1 percent levels. The test data are presented in accordance with the results of these analyses as arithmetic means for individual bridges and as arithmetic means for groups of bridges. Group means are given only when the analyses indicated no significant differences between the means of the individual bridges in the group.

7.4.1 Tension Tests of Structural Steel

Twelve specimens were cut from each 2-ft coupon obtained from the steel beams of the original steel bridges—four from each flange

* Kingham, R. I., Fisher, J. W., and Viest, I. M., "Creep and Shrinkage of Concrete in Outdoor Exposure and Relaxation of Prestressing Steel." HRB Special Report 66 (in press). Fisher, J. W., and Viest, I. M., "Fatigue Tests of Bridge Materials of the AASHO Road Test." HRB Special Report 66 (in press).

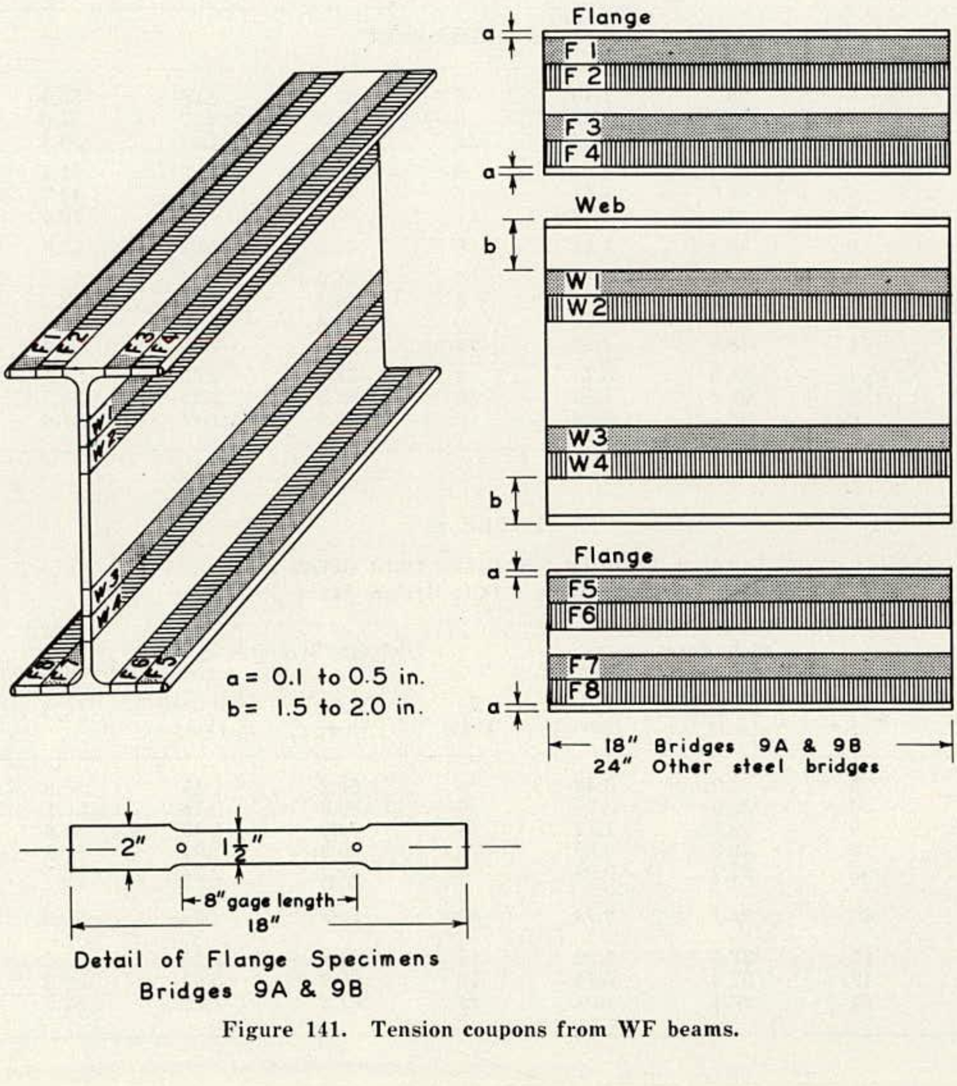


Figure 141. Tension coupons from WF beams.

TABLE 84

TENSION TESTS OF SPECIMENS FROM WIDE FLANGE BEAMS
(Obtained from Data System 2410)

Bridge No.	Yield Point			Ultimate Strength			Reduction in Area (%)	Elongation (%)
	No. Tests	Mean (ksi)	Std. Dev. (ksi)	No. Tests	Mean (ksi)	Std. Dev. (ksi)		
(a) FLANGE SPECIMENS								
1A	12	34.7	1.26	12	59.4	0.93	57.2	31.3
2A	12	34.7	1.45	12	59.6	1.16	56.0	33.3
1A & 2A	*24*	*34.7*	*1.33*	*24*	*59.5*	*1.03*	*56.6*	*32.4*
1B	12	37.8	1.48	12	63.8	0.99	50.6	28.0
2B	12	37.9	1.42	12	63.6	0.75	53.6	31.3
1B & 2B	*24*	*37.9*	*1.43*	*24*	*63.7*	*0.87*	*52.1*	*29.8*
3A	12	35.3	0.47	12	61.3	0.47	56.0	30.9
3B	12	35.2	1.75	12	65.3	2.26	50.0	29.7
4A	12	35.0	1.32	12	64.9	2.43	47.3	29.4
4B	12	35.2	1.62	12	64.4	2.60	51.6	28.9
3B, 4A & 4B	*36*	*35.1*	*1.53*	*36*	*64.9*	*2.40*	*49.6*	*29.3*
9A	12	32.1	1.04	12	61.8	1.34	54.3	31.3
9B	12	32.9	1.30	12	62.1	1.39	53.2	31.1
9A & 9B	*24*	*32.5*	*1.21*	*24*	*62.0*	*1.35*	*53.8*	*31.2*
(b) WEB SPECIMENS								
1A	6	38.1	1.49	6	61.1	1.66	53.9	30.9
2A	6	37.8	2.39	6	62.0	2.42	51.6	32.1
1A & 2A	*12*	*38.0*	*1.91*	*12*	*61.5*	*2.01*	*52.8*	*31.5*
1B	6	40.9	1.41	6	65.5	1.85	51.4	29.8
2B	6	42.0	1.47	6	66.3	1.93	49.7	29.4
1B & 2B	*12*	*41.4*	*1.48*	*12*	*65.9*	*1.86*	*50.5*	*29.6*
3A	6	39.1	1.39	6	63.6	1.02	53.3	31.6
3B	6	40.1	2.10	6	67.0	3.17	51.0	29.7
4A	6	39.8	1.53	6	66.4	2.28	48.9	30.3
4B	6	39.9	3.06	6	66.3	4.13	50.8	29.6
3B, 4A & 4B	*18*	*39.9*	*2.18*	*18*	*66.6*	*3.10*	*50.2*	*29.8*
9A	4	35.8	0.75	4	61.4	0.63	51.7	33.2
9B	6	36.4	1.52	6	62.8	2.25	50.5	33.1
9A & 9B	*10*	*36.1*	*1.22*	*10*	*62.2*	*1.87*	*51.0*	*33.2*

TABLE 85

TENSION TESTS OF SPECIMENS FROM COVER PLATES
(Obtained from Data System 2411)

Bridge No.	Yield Point			Ultimate Strength			Reduction in Area (%)	Elongation (%)
	No. Tests	Mean (ksi)	Std. Dev. (ksi)	No. Tests	Mean (ksi)	Std. Dev. (ksi)		
1A	6	38.0	0.48	6	60.2	1.41	55.5	28.4
2B	6	39.0	0.77	6	60.8	0.78	56.1	29.9
3B	6	39.1	1.17	6	60.8	1.43	54.6	30.6
4A	6	38.3	0.59	6	59.8	1.04	57.2	27.7
4B	6	37.7	0.99	6	59.1	2.74	56.5	30.0
1A, 2B, 3B, 4A & 4B	*30*	*38.4*	*0.95*	*30*	*60.2*	*1.65*	*56.0*	*29.3*
9A	12	37.6	1.22	12	61.6	1.35	52.0	27.1
9B	12	37.3	0.84	12	60.7	0.93	53.0	27.1
9A & 9B	*24*	*37.4*	*1.04*	*24*	*61.2*	*1.23*	*52.5*	*27.1*

and four from the web. Figure 141 shows the location of specimens within the flanges and the web of each coupon. The specimens had a uniform width of approximately 1.5 in., except that specimens from flanges of ten beams were approximately 1 in. wide.

The middle 2-ft section of each 9-ft coupon obtained from the beams of the two additional steel bridges, 9A and 9B, also was cut into twelve specimens as shown in the figure. The web specimens had a uniform width of approximately 2 in. The flange specimens were cut as shown in Figure 141 to permit their testing in the same machine.

Static tension tests were made on six specimens from each coupon; two specimens were selected at random from the four available from each flange and the web.

Each coupon obtained from the cover plates of all steel beam structures was squared off at the edges and cut longitudinally into four specimens of approximately the same width. Static tension tests were made on two specimens selected at random from the four available from each cover plate coupon.

Static tension tests on specimens from the beams and cover plates of all steel beam bridges were made in a 100,000-lb capacity screw-type mechanical testing machine. The specimens were tested in two groups—one for the original eight structures and the other for the two additional bridges. The order of tests was randomized within each group.

The original dimensions, yield point load, ultimate load, final dimensions and location of break were recorded for each specimen. In addition, complete stress-strain curves were obtained for at least two specimens from each beam (one from the flanges and one from the web), and for at least one specimen from each cover plate.

The load was applied at the cross-head speed of 0.104 in. per min up to yielding, and then at the speed of 0.36 in. per min to fracture.

The yield point load was determined by observing the load at the instant of the drop of beam of the testing machine. The ultimate load was the maximum registered by the machine. The ultimate elongation was based on an 8-in. gage length.

The yield points and the ultimate strengths are listed in Table 84 and 85 for the wide flange beams and for the cover plates, respectively. Also included are reductions in area and the elongations after failure. The data are divided into several groups suggested by the analyses. It may be noted by comparison with the data in Table 75 of Section 7.2.1 that each group contains specimens from one heat of steel.

The mean value of the Young's modulus of elasticity, determined from tests of 104 specimens, was 30,000,000 psi and the standard deviation was 980,000 psi. A typical stress-strain curve for the structural steel is shown in Figure 142.

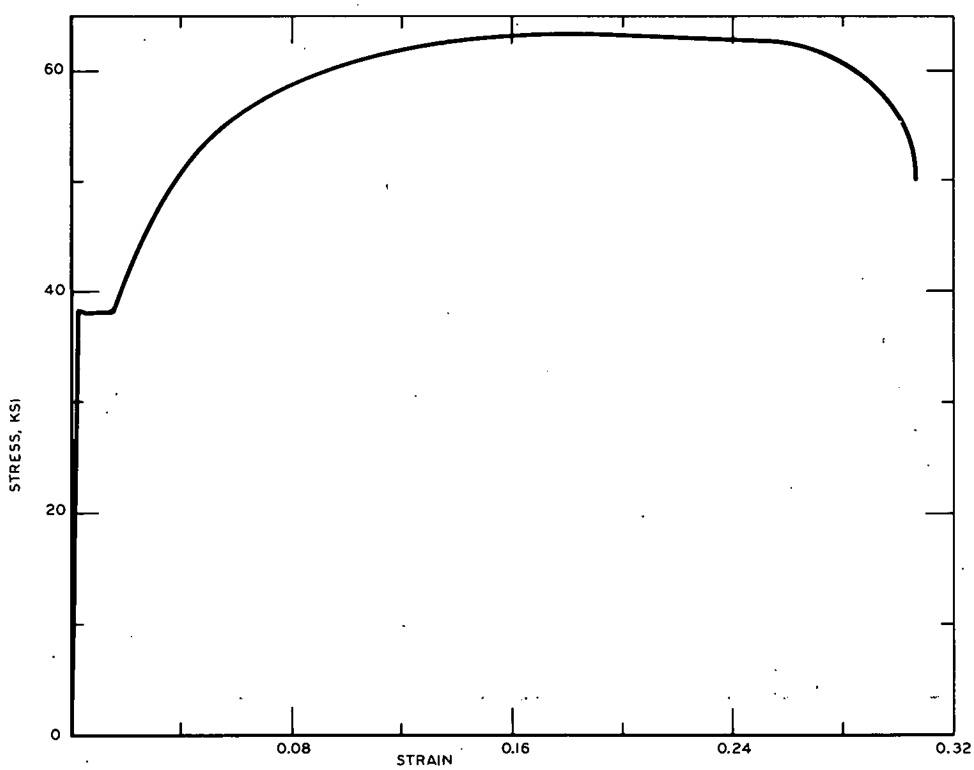


Figure 142. Typical stress-strain curve for structural steel.

TABLE 86

TENSION TESTS OF PRESTRESSING STEEL
(Obtained from Data System 2413)

Bridge No.	Area		Yield Strength[1]			Ultimate Strength		
	No. Samples	Mean (sq in.)	No. Tests	Mean (ksi)	Std. Dev. (ksi)	No. Tests	Mean (ksi)	Std. Dev. (ksi)
5A	36	0.0293	13	227.2	1.93	36	257.5	2.29
5B	54	0.0293	19	227.5	3.47	54	257.2	2.64
5A & 5B	*90*	*0.0293*	*32*	*227.4*	*2.91*	*90*	*257.3*	*2.50*
6A	15	0.0807	15	229.4	7.18	15	265.6	4.11
6B	18	0.0806	18	243.0	7.36	18	275.2	7.01

[1] Stress at 1 percent strain obtained from stress-strain curves.

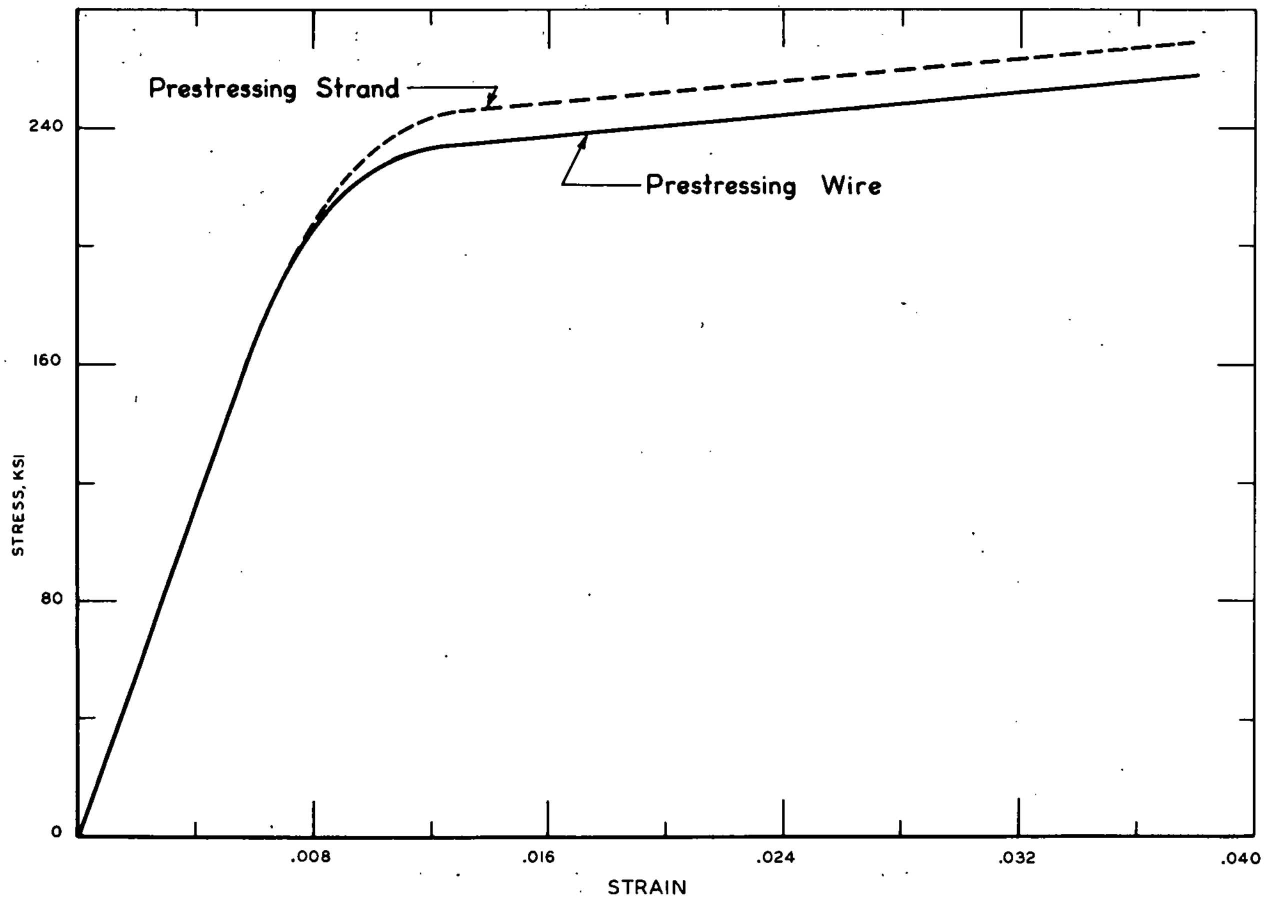


Figure 143. Typical stress-strain curve for prestressing steel.

The second group of tests also included the determination of the static yield point. The results are reported in Appendix E.

7.4.2 Tension Tests of Prestressing Steel

Specimens of the prestressing steel used in the post-tensioned beams of bridges 5A and 5B were obtained after tensioning of the beams. The excess lengths of the ten parallel wires in each tendon were cut off at both ends of the beams. The specimens were 1.5 to 2 ft long.

Three of the available specimens from each tendon were selected at random for tension tests. A 120,000-lb capacity hydraulic testing machine was used. The wires were gripped at both ends with steel jaws.

The order of testing the specimens was randomized. The original dimensions, load corresponding to a noticeable change in the straining rate, ultimate load, final diameter, ultimate elongation in 10 in., and the location of the break were recorded for each wire. Complete stress-strain curves were obtained for one of the three specimens from each tendon.

The load was applied at a constant rate to failure. The ultimate load was the maximum registered by the machine.

Table 86 gives the yield strength and ultimate strength of the 0.192-in. diameter wire used in post-tensioned beam bridges, and includes the means for the individual bridges as well as the over-all mean. Also included are measured cross-sectional areas of the wire. The mean elongation after failure was 4.6 percent in 10 in.

The mean value of the initial modulus of elasticity, determined from tests of 34 specimens, was 28,600,000 psi, and the standard deviation was 710,000 psi.

Specimens of the prestressing steel used in the pretensioned beams of bridges 6A and 6B were obtained from the strand lengths located between the beams and the abutments of the prestressing bed. Seven to nine pieces were selected at random from each bed. A 4-ft specimen was cut from each selected piece for tension tests. The total number of tension specimens was 33.

The static tension tests of the 7-wire strand specimens were made in a 200,000-lb capacity hydraulic machine. To permit satisfactory gripping of the specimens and to avoid failures at the grips, the ends of each specimen were coated with babbitt metal. Flat face grips were used at both ends.

TABLE 87

DIMENSIONS OF REINFORCING BARS [1]
(Obtained from Data System 2412)

Bar Size	No. of Samples	Area (sq in.)	Dimensions of Deformations (in.) Spacing	Height	Gap
3	40	0.111	0.166	0.019	0.080
3 [2]	16	0.106	0.162	0.022	0.044
4	16	0.199	0.222	0.030	0.102
4 [2]	8	0.187	0.217	0.036	0.045
5	32	0.303	0.284	0.040	0.110
8	6	0.782	0.417	0.059	0.162
9	24	0.957	0.476	0.084	0.212
11	36	1.524	0.585	0.073	0.250

[1] See AASHO Specification M137.
[2] Used in prestressed concrete beams; all other bars used at the job site in the original 16 test bridges.

TABLE 88

TENSION TESTS OF REINFORCING BARS
(Obtained from Data System 2412)

Bar Size	Yield Point No. Tests	Yield Point Mean (ksi)	Yield Point Std. Dev. (ksi)	Ultimate Strength No. Tests	Ultimate Strength Mean (ksi)	Ultimate Strength Std. Dev. (ksi)	Reduction in Area (%)	Elongation (%)
3	40	61.2	1.28	40	86.3	1.49	29.5	19.6
3 [1]	15	45.6	1.66	16	70.0	2.05	46.0	23.9
3 [2]	4	47.9 [3]	2.60	4	75.3 [3]	2.60	—	—
4	16	54.9	1.43	16	84.2	2.82	27.0	20.5
4 [1]	8	47.5	1.15	8	79.8	1.84	29.9	20.5
5	32	53.0	1.57	32	85.4	2.19	21.6	19.5
5 [2]	4	50.8 [3]	2.78	4	82.9 [3]	1.11	—	—
8	6	52.6	0.78	6	81.5	1.22	29.4	23.8
9	15	51.8	1.13	15	79.7	1.39	23.5	23.9
11	15	49.5	0.77	15	81.0	1.72	21.3	22.4

[1] Used in prestressed concrete beams.
[2] Used in slabs of additional steel beam bridges 9A and 9B.
[3] Based on nominal cross-sectional area of bar.

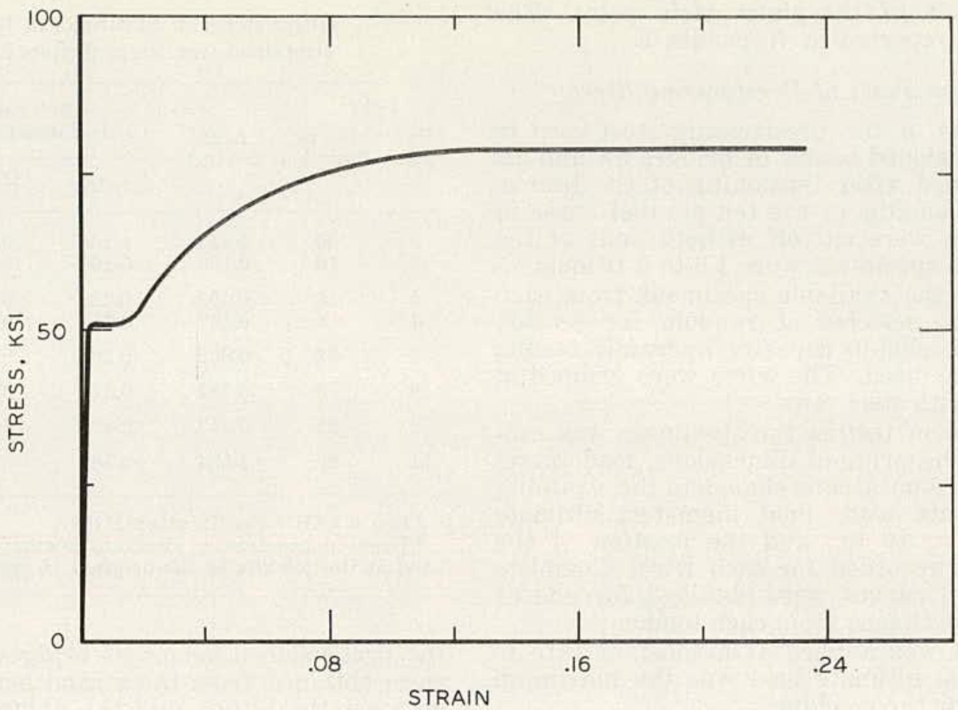


Figure 144. Typical stress-strain curve for reinforcing bars.

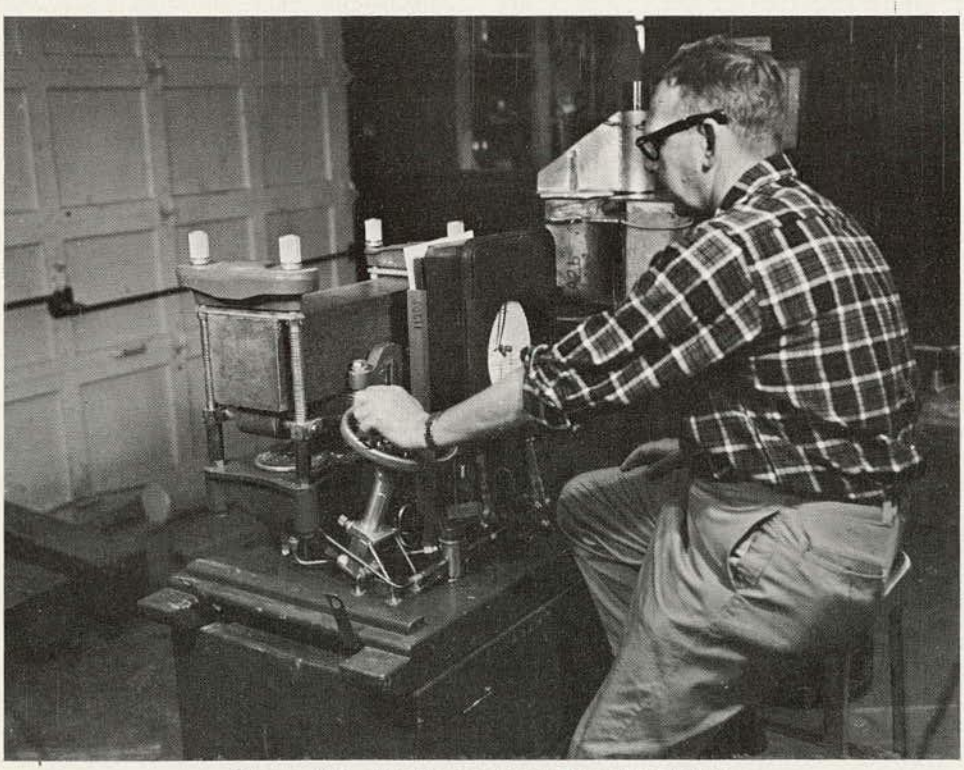


Figure 145. Testing concrete beam in flexure.

TABLE 89

TESTS OF CONCRETE CYLINDERS FOR SLABS OF STEEL BEAM BRIDGES
(Obtained from Data System 2420)

Bridge No.	At 28 Days			Beginning of Test Traffic				End of Test Traffic			
	No. Tests	Mean	Std. Dev.	No. Tests	Age (days)	Mean	Std. Dev.	No. Tests	Age (days)	Mean	Std. Dev.
(a) COMPRESSIVE STRENGTH (psi)											
1A	6	4180	299	6	446	5130	357	6	1218	5350	224
1B	6	4430	63	6	433	5150	264	6	1205	5360	282
2A	6	4200	186	6	433	5010	355	6	1211	5590	348
4B	6	4260	196	6	442	5140	721	6	1214	5400	504
1A, 1B, 2A & 4B	*24*	*4270*	*214*	*24*		*5110*	*432*	*24*		*5430*	*346*
2B	6	4060	157	6	447	4760	203	6	1216	5200	337
3A	6	4570	326	6	442	5530	403	6	1219	5860	171
4A	6	4380	119	6	436	5380	194	6	1207	5680	195
3A & 4A	*12*	*4470*	*255*	*12*		*5460*	*313*	*12*		*5770*	*199*
3B	6	4830	289	6	436	5740	219	6	1213	6020	295
9A	6	5540	488	6	107[1]	6260	463	6	563	6450	516
9B	6	5080	97	6	107[1]	5820	357	6	566	6360	264
9A & 9B	*12*	*5310*	*400*	*12*		*6040*	*458*	*12*		*6410*	*394*
(b) MODULUS OF ELASTICITY (10^6 psi)											
1A	2	4.3	0.14	3	446	4.9	0.28	3	1218	5.5	0.33
1B	3	4.5	0.16	3	433	5.6	0.46	2	1205	5.7	0.00
2A	3	4.5	0.07	3	433	5.1	0.20	3	1211	5.5	0.06
2B	2	4.8	0.10	3	447	4.7	0.15	3	1216	5.2	0.63
3A	3	4.2	0.07	4	442	5.4	0.62	3	1219	6.0	0.22
3B	3	4.4	0.23	2	436	5.2	0.16	3	1213	5.9	0.46
4A	4	4.5	0.28	3	436	5.2	0.59	3	1207	6.0	0.27
4B	4	3.6	0.99	3	442	5.0	0.97	3	1214	5.0	0.52
9A	4	4.6	0.15	4	107[1]	5.2	0.75	3	563	5.7	0.20
9B	4	5.1	0.60	4	107[1]	5.2	0.85	3	566	5.8	0.21
1A through 9B	*32*	*4.4*	*0.51*	*32*		*5.2*	*0.52*	*29*		*5.6*	*0.35*

[1] Traffic started at 28 days; values shown are from 107-day cylinder tests, which are more representative of conditions existing through most of traffic period.

TABLE 90

TESTS OF CONCRETE CYLINDERS FOR REINFORCED CONCRETE BRIDGES
(Obtained from Data System 2420)

Bridge No.	At 28 Days			Beginning of Test Traffic				End of Test Traffic			
	No. Tests	Mean	Std. Dev.	No. Tests	Age (days)	Mean	Std. Dev.	No. Tests	Age (days)	Mean	Std. Dev.
(a) COMPRESSIVE STRENGTH (psi)											
7A	16	3680	308	15	447	4620	376	16	1238	5020	335
8B	16	3820	333	16	446	4500	446	16	1237	4880	409
7A & 8B	*32*	*3750*	*324*	*31*		*4560*	*411*	*32*		*4950*	*374*
7B	18	3470	293	19	412	4390	331	18	1200	4800	415
8A	18	4020	342	18	412	4960	476	18	1206	5290	409
(b) MODULUS OF ELASTICITY (10^6 psi)											
7A	6	3.8	0.46	6	447	4.6	0.70	5	1238	5.4	0.32
7B	6	3.6	0.30	10	412	5.0	0.61	7	1200	4.9	0.75
8A	7	4.3	0.27	9	412	5.2	0.31	9	1206	6.1	0.58
8B	6	4.2	0.28	9	446	5.1	0.41	5	1237	5.7	0.53
7A, 7B, 8A & 8B	*25*	*4.0*	*0.34*	*34*		*5.0*	*0.52*	*26*		*5.5*	*0.59*

TABLE 91

TESTS OF CONCRETE CYLINDERS FOR SLABS OF PRESTRESSED CONCRETE BRIDGES

(Obtained from Data System 2420)

Bridge No.	At 28 Days			Beginning of Test Traffic				End of Test Traffic			
	No. Tests	Mean	Std. Dev.	No. Tests	Age (days)	Mean	Std. Dev.	No. Tests	Age (days)	Mean	Std. Dev.
				(a) COMPRESSIVE STRENGTH (psi)							
5A	6	4950	203	6	193	6440	237				
5B	6	4490	343	6	192	5700	315	22[1]	972 to 1114	5530	572
6A	6	3010	408	6	333	5480	799				
6B	6	2860	364	6	332	4890	643				
				(b) MODULUS OF ELASTICITY (10^6 psi)							
5A	3	4.2	0.03	4	193	5.7	0.56				
5B	3	4.0	0.31	3	192	5.1	0.12				
6A	3	4.0	0.17	3	333	5.7	0.46	12[1]	972 to 1114	5.9	0.32
6B	3	3.5	0.23	2	332	6.2	0.00				
5A, 5B 6A & 6B	*12*	*3.9*	*0.34*	*12*		*5.6*	*0.46*				

[1] Because of illegible markings, cylinders could not be identified with individual bridges.

The order of testing the 33 specimens was randomized. The original strand dimensions, ultimate load, elongations for stress-strain curves, and the ultimate elongation of the center wire in 32 in. were recorded.

The cross-sectional area, yield strength at 1 percent strain and ultimate strength for the prestressing strand are also given in Table 86.

The mean ultimate elongation in 32 in. was 3.9 percent. The mean value of the initial modulus of elasticity was 27,600,000 psi, and the standard deviation was 880,000 psi.

Typical stress-strain curves for the prestressing wire and the prestressing strand are shown in Figure 143.

7.4.3 Tension Tests of Reinforcing Bars

Specimens of concrete reinforcing steel were cut directly from the excess lengths of the deformed bars used in the bridges. All specimens were 2 ft long and were cut at the project site. A total of 156 specimens were tested. Of these, 148 were from the original 16 bridges and 8 were from bridges 9A and 9B.

Test specimens were obtained for each size of deformed bar used in the slabs of all bridges and in the beams of the reinforced and prestressed concrete bridges. One specimen was cut from each tension bar in the beams of the reinforced concrete bridges. Two specimens were taken from the web reinforcement of each reinforced concrete bridge. For the prestressed concrete beams, two specimens were taken from the web reinforcement and one from the reinforcement of the end blocks for each beam of bridges 5B and 6B, and for the center beam of bridges 5A and 6A. Four test specimens were taken from the slab reinforcement of each test bridge—two from the transverse bars and two from the longitudinal bars.

The static tension tests of all bars, except No. 11 and those from bridges 9A and 9B, were carried out in a 100,000-lb capacity screw-type mechanical machine. The No. 11 bar specimens and those from bridges 9A and 9B were tested in a 200,000-lb capacity hydraulic machine.

The 148 specimens from the original 16 bridges were divided into 20 groups, each group consisting of 5 to 8 bars of one size. The distribution of the bars to the groups and the order of testing of the groups were selected at random. The weight, original dimensions, yield point load, ultimate load, final dimensions and location of break were recorded for each specimen. Complete stress-strain curves were determined for five specimens of each size and origin. Only the yield point load and ultimate load were recorded for the eight specimens of bridges 9A and 9B.

Except in tests of the No. 11 bars, the load was applied at the speed of 0.104 in. per min up to yielding and then at the speed of 0.36 in. per min to failure. The No. 11 bars were loaded at a constant slow speed to failure.

The yield point was determined by the drop of beam of the mechanical machine and by the halt of the needle of the hydraulic machine. The ultimate load was the maximum registered. The elongation was based on an 8-in. gage length.

The results of the tests are summarized in Tables 87 and 88. The mean dimensions of the various sizes of bars are listed in Table 87. Table 88 includes the mean and standard devia-

TABLE 92

TEST OF CONCRETE CYLINDERS FOR BEAMS OF PRESTRESSED CONCRETE BRIDGES

(Obtained from Data System 2420)

Bridge No.	At Transfer				At 28 Days			Beginning of Test Traffic				End of Test Traffic			
	No. Tests	Age (days)	Mean	Dev. Std.	No. Tests	Mean	Std. Dev.	No.	Age (days)	Mean	Std. Dev.	No. Tests	Age (days)	Mean	Std. Dev.
							(a) COMPRESSIVE STRENGTH (psi)								
5A	6	6	5430	666	6	6250	500	6	357	9250	649	5	1127	9070	203
5B	6	35[1]	5790	1245	6	7030	653	6	395	9200	944	6	1132	8360	745
6A	6	4	5080	478	6	6880	797	6	396	9510	396	6	1166	9110	1626
6B	6	3	5020	322	6	6430	296	6	394	9090	597	6	1164	9050	432
5A, 5B 6A & 6B	*24*		*5330*	*823*	*24*	*6650*	*645*	*12*		*5.7*	*0.30*	*12*		*5.8*	*0.32*
							(b) MODULUS OF ELASTICITY (10^6 psi)								
5A	3	6	4.2	0.29	3	4.9	0.08	3	357	5.7	0.50	3	1127	6.1	0.47
5B	3	35[1]	3.8	0.90	1	5.2	....	3	395	5.5	0.22	3	1132	5.4	0.23
6A	3	4	4.3	0.32	4	5.1	0.25	3	396	5.8	0.17	3	1166	5.9	0.22
6B	3	3	4.6	0.44	0	...	....	3	394	5.9	0.22	3	1164	5.7	0.30
5A, 5B 6A & 6B	*12*		*4.2*	*0.57*	*8*	*5.1*	*0.21*	*12*		*5.7*	*0.30*	*12*		*5.8*	*0.32*

[1] Ages of specimens were 97 days, 3 days and 4 days for Beams 1, 2 and 3, respectively.

TABLE 93

FLEXURAL STRENGTH OF CONCRETE FROM BEAMS OF PRESTRESSED CONCRETE BRIDGES

(Obtained from Data System 2421)

Bridge No.	At Transfer				At 28 Days			Beginning of Test Traffic				End of Test Traffic			
	No. Tests	Age (days)	Mean (psi)	Std. Dev.	No. Tests	Mean (psi)	Std. Dev.	No. Tests	Age (days)	Mean (psi)	Std. Dev.	No. Tests	Age (days)	Mean (psi)	Std. Dev.
5A	6	6	672	72	6	606	12	6	344	732	90	6	1121	1075	348
5B	4	35[1]	698	99	6	649	19	6	382	771	100	6	1159	1250	130
6A	6	4	662	69	6	825	57	6	383	725	114	6	1160	1125	75
6B	6	3	652	42	6	721	6	6	381	730	88	6	1158	1160	26
5A, 5B 6A & 6B	*22*		*670*	*66*	*24*	*700*	*27*	*24*		*739*	*94*	*24*		*1155*	*190*

[1] Ages of specimens were 97 days, 3 days and 4 days for Beams 1, 2 and 3, respectively.

TABLE 94

AVERAGE UNIT WEIGHT OF CONCRETE IN TEST BRIDGE SUPERSTRUCTURES

Mix Design (psi)	Bridge No.	Fresh Conc. (pcf)	At 28 Days (pcf)	Beginning of Test Traffic (pcf)	End of Test Traffic (pcf)
3,000	5A, 5B, 6A, 6B (slabs) 7A, 7B, 8A, 8B (slabs and beams)	149	147	147	147
4,000	1A, 2A, 3A, 3B, 4A 4B (slabs)	149	148	148	149
4,000	1B (slabs)[1]	162	162	160	162
4,000	9A, 9B (slabs)	150	—	149	149
5,000	5A, 5B, 6A, 6B (beams)	147	—	148	148

[1] Concrete with steel punchings.

TABLE 95

COEFFICIENT OF THERMAL EXPANSION OF CONCRETE IN TEST BRIDGES
(Obtained from Data System 2422)

Bridge Type	Element	Coefficient of Thermal Expansion for Temperature Change (μ in./in./°F)			
		73 F to 100 F	100 F to 73 F	73 F to 37 F	37 F to 73 F
Reinforced concrete	Slabs and beams	6.72	6.40	4.77	4.84
Prestressed concrete	Slabs	7.46	6.53	4.95	5.08
Steel	Slabs	7.14	6.69	5.22	5.21
Prestressed concrete	Beams	7.04	7.50	5.86	5.50

TABLE 96

CHEMICAL ANALYSIS OF PRESTRESSING STEEL
(Obtained from Data System 2416)

Type	No. Tests	Chemical Analysis (%)				
		C	Mn	P	S	Si
0.192-in. dia. wire:						
Shipment 1	3	0.80	0.67	0.016	0.040	0.25
Shipment 2	3	0.80	0.63	0.016	0.037	0.25
Shipments 1& 2	6	0.80	0.65	0.016	0.039	0.25
7-wire strand:						
Spool 1	3	0.72	0.71	0.015	0.034	0.23
Spool 2	3	0.73	0.78	0.017	0.039	0.24
Spools 1 & 2	6	0.73	0.75	0.016	0.037	0.24

tion for the yield point and ultimate strength of each bar size, as well as the mean value for reduction in area and the elongation in 8 in. after failure. It may be noted that the mean yield point for bars made from the same heat of steel (Table 77, Section 7.2.1) increases with decreasing bar size.

The mean initial modulus of elasticity of 53 specimens representing all bar sizes was 28,800,000 psi and the standard deviation was 1,060,000 psi. A typical stress-strain curve for the reinforcing bars is shown in Figure 144.

7.4.4 Compression Tests of Concrete

The concrete cylinders for compression tests were 6 in. in diameter and 12 in. long. They were stored in racks beneath the corresponding bridges until tested.

The test cylinders molded from the concrete used in the slab of any one bridge were made in six sets of three each, one set from every third batch of concrete. One of each set was tested at the age of 28 days, the second at the beginning of test traffic, and the third at the conclusion of test traffic.

Concrete cylinders for the beams of the reinforced concrete bridges were made and tested by the procedure described for the slabs except that a set of three cylinders was taken from each batch of concrete placed in the beams.

A set of nine cylinders was cast from each of the two batches of concrete placed in each prestressed concrete beam. Four cylinders of each set were tested in compression. One was tested at the time of transfer of the prestressing force to the beams, and the other three were tested at 28 days, at the beginning of test traffic and at the conclusion of test traffic. The remaining cylinders were used to determine the time of transfer of the prestressing force to the concrete and in a study of the creep and shrinkage characteristics of concrete.

The concrete cylinders from the prestressed concrete beams were capped with a sulfur compound and tested in a 300,000-lb capacity hydraulic testing machine. All other cylinders were capped with high strength gypsum plaster and tested in a 200,000-lb capacity portable hydraulic testing machine.

The test cylinders were loaded at a constant rate of approximately 1,200 lb per sec to failure. The original average diameter and the maximum load indicated by the testing machine were recorded for each cylinder. In addition, elongations for the stress-strain relationship were obtained on a few of the cylinders from each slab and beam, using an 8-in. extensometer.

Tables 89, 90, 91 and 92 summarize the results of the compression tests on the concrete cylinders.

7.4.5 Miscellaneous Tests

A set of five 6- by 6- by 30-in. beams for flexural tests was made from each batch of concrete used in the prestressed concrete beams. One of each set was tested at the time of transfer of the prestressing force to the beams, at the age of 28 days, and at the beginning and end of the regular test traffic.

The flexural tests of concrete beams were performed in accordance with AASHO Designation: T97-57. The tests were made with third-point loading. The dimensions of the cross-section and the breaking load were recorded for each test beam. The results are given in Table 93.

The unit weight of concrete was determined for each of the three mix designs from yield tests on the fresh concrete and from tests on cylinders at the age of 28 days and at the beginning and end of the regular test traffic. Table 94 gives the average unit weights of the 3,000-, 4,000- and 5,000-psi concrete. Each mean value in the table represents the average of the results of at least three tests per test bridge.

Four 3- by 3- by 11-in. prisms were made during the casting of the prestressed concrete beams for the evaluation of the coefficient of thermal expansion of the concrete. Twelve

TABLE 97

DIMENSIONS OF STEEL BEAMS

Bridge	Nominal Beam Size	Wt. per Foot (lb)	Area (sq in.)	Depth (in.)	Flange Width (in.)	Flange Thickness (in.)	Web Thickness (in.)
1A	18WF55	56.89	16.72	18.22	7.64	0.640	0.396
1B	18WF50	51.69	15.19	18.09	7.57	0.576	0.369
2A	18WF55	56.79	16.69	18.22	7.63	0.641	0.395
2B	18WF50	52.06	15.30	18.10	7.59	0.579	0.372
3A	21WF62	61.18	17.98	20.91	8.15	0.600	0.402
3B	18WF60	59.00	17.34	18.29	7.51	0.668	0.418
4A	18WF60	59.04	17.35	18.28	7.54	0.672	0.413
4B	18WF60	59.79	17.57	18.30	7.57	0.679	0.418
9A	18WF96	93.41	27.45	18.22	11.77	0.814	0.486
9B	18WF96	93.92	27.60	18.21	11.78	0.819	0.488

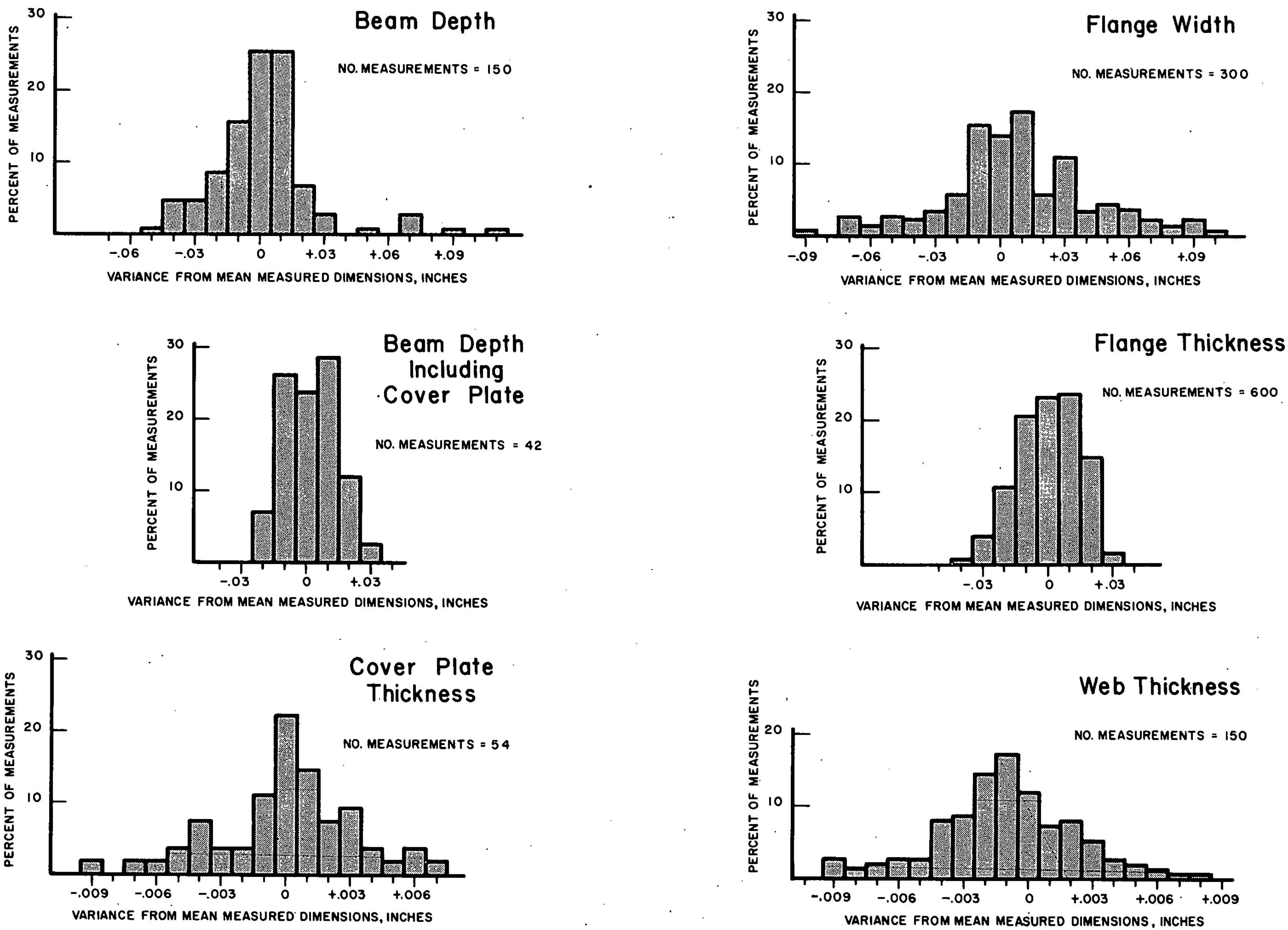


Figure 146. Measurements of WF steel I-beams.

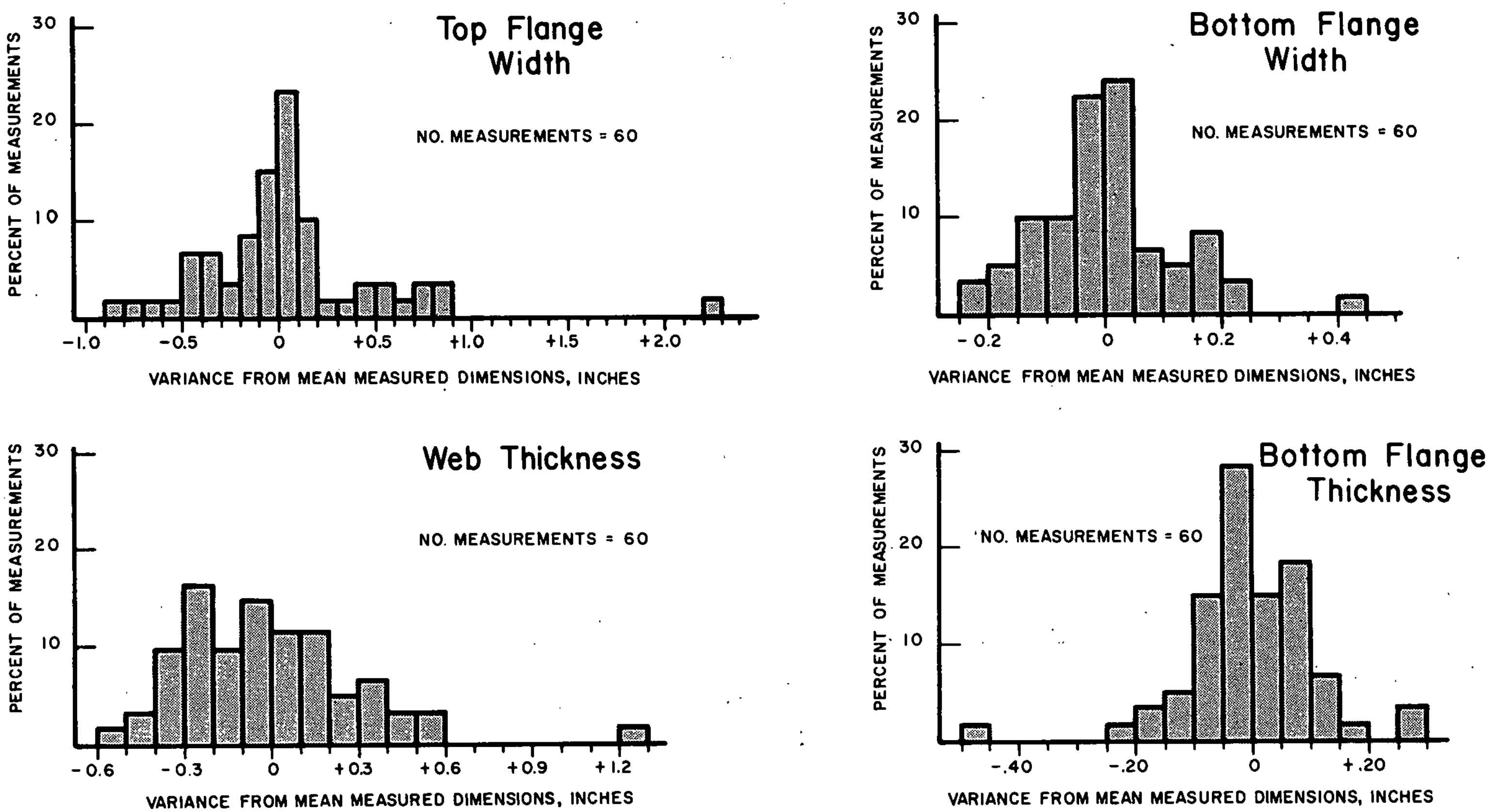


Figure 147. Measurements of prestressed concrete beams.

TABLE 98

DIMENSIONS OF STEEL BEAMS WITH COVER PLATES

Bridge	Nom. Plate Size (in. × in. × ft)	Beam Wt. per Foot (lb)	Area (sq in.)	Depth (in.)	Plate Thickness (in.)
1A	6 × 7/16 × 20.5	65.91	19.37	18.68	0.441
2B	6 × 7/16 × 14.0	61.32	18.02	18.57	0.452
3B	6 × 7/16 × 18.5	68.16	20.03	18.76	0.448
4A	6 × 7/16 × 19.0	68.09	20.01	18.75	0.443
4B	6 × 7/16 × 19.0	68.80	20.22	18.77	0.442
9A	6 × 7/16 × 17.0	111.44	32.75	19.10	0.442
9B	6 × 7/16 × 17.0	112.02	32.92	19.09	0.443

TABLE 99

DIMENSIONS OF PRESTRESSED CONCRETE BEAMS

Bridge	Wt. per Foot (lb)	Depth Below Slab (in.)	Top Flange		Web Thick. (in.)	Bottom Flange		Area of Reinforcement (sq in.)			Area of Top Reinf. (sq in.)
			Width (in.)	Thick. (in.)		Width (in.)	Thick. (in.)	Bot. Row	2nd Row	Top Row	
5A	275.0	24.76	13.08	3.70	5.68	17.50	4.41	1.173	0	0	0
5B	270.3	24.53	12.98	3.89	5.40	17.29	4.27	1.172	0.293	0.293	0
6A	275.4	24.28	13.31	3.92	5.72	17.31	4.26	0.565	0.404	0.161	0.161
6B	283.9	24.05	13.85	3.80	6.09	17.38	4.32	0.645	0.645	0.161	0.161

TABLE 100

DIMENSIONS OF REINFORCED CONCRETE BEAMS

Bridge	Wt. per Foot (lb)	Depth (in.)	Width (in.)	Area of Reinforcement (sq in.)	
				Bottom Row	Top Row
7A	264.0	19.98	11.60	4.568	1.941
7B	262.8	20.03	11.51	4.579	1.923
8A	264.1	19.95	11.53	4.576	2.694
8B	264.9	19.97	11.56	4.567	2.662

TABLE 101

DIMENSIONS OF BRIDGE SLABS

Bridge	Nominal Slab Size (in. × ft)	Wt. per Sq Foot (lb)	Thickness (in.)	Area[1] (sq in./ft)	
				Trans. Bars	Long. Bars
1A	6.5 × 15	83.18	6.44	1.454	0.133
1B	6.5 × 15	91.08	6.56	1.454	0.133
2A	6.5 × 15	83.05	6.43	1.454	0.133
2B	6.5 × 15	81.45	6.30	1.454	0.133
3A	6.5 × 15	83.05	6.43	1.454	0.133
3B	6.5 × 15	83.55	6.47	1.454	0.133
4A	6.5 × 15	83.05	6.43	1.454	0.133
4B	6.5 × 15	83.30	6.45	1.454	0.133
5A	6.5 × 15	83.83	6.62	0.909	0.239
5B	6.5 × 15	84.75	6.69	0.909	0.239
6A	6.5 × 15	83.21	6.57	0.909	0.239
6B	6.5 × 15	83.70	6.61	0.909	0.239
7A	6.5 × 15	82.37	6.48	0.909	0.345
7B	6.5 × 15	83.23	6.55	0.909	0.345
8A	6.5 × 15	83.96	6.61	0.909	0.345
8B	6.5 × 15	83.72	6.59	0.909	0.345
9A	6.5 × 15	87.35	6.73	1.454	0.133
9B	6.5 × 15	84.99	6.54	1.454	0.133

[1] Area given represents total of bottom and top layer; one-half of total was placed in each layer.

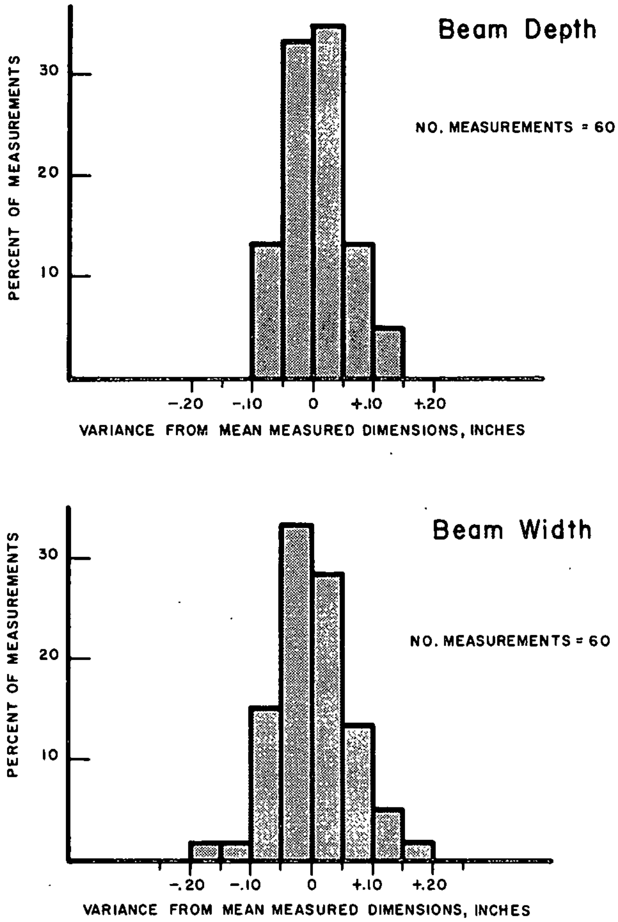


Figure 148. Measurements of reinforced concrete beams.

cylinders made of the concrete used in the slabs of the test bridges also were tested for thermal coefficient of expansion—one cylinder from each slab of the reinforced and prestressed concrete beam bridges and one cylinder from each slab of four of the steel beam bridges.

The four prisms and the twelve cylinders were equipped with gage plugs and wrapped in a plastic membrane to inhibit moisture loss during testing. The results of the tests on the 16 specimens are summarized by bridge type and element in Table 95. The tests were made in the fall of 1960.

Samples of the prestressing steel were obtained during the manufacture of the prestressed concrete beams for chemical analysis of the steel. Three samples were analyzed for each of the two shipments of wire for the post-tensioned beams and for each of the two spools of wire strand for the pretensioned beams. The results are given in Table 96.

A sample of the 12- by 12-in. timber wheel guard was obtained from each of the four bridge sites to determine its unit weight. The average of these four samples, corrected for the weight of the hardware, was 40 lb per linear foot.

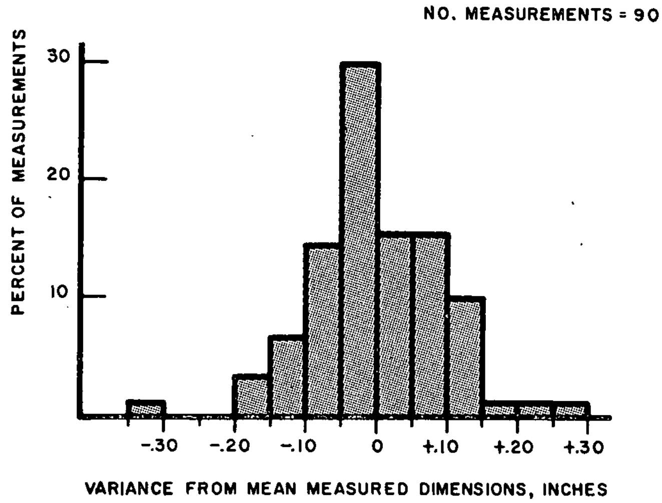


Figure 149. Measurements of slab thickness, all bridges.

7.5 DIMENSIONS AND PROFILES OF COMPLETED TEST BRIDGES

7.5.1 Bridge Dimensions

The dimensions of beams and slabs were measured after the completion of construction of the test bridge superstructures. All measurements were made with conventional instruments adapted for each specific purpose.

Cross-section properties, including areas and weights of beams and slabs, were computed for each bridge using the mean dimensions. The unit weight of steel was taken as 490 lb per cu ft. Unit weights measured at the beginning of test traffic were used for concrete.

Span lengths, beam spacings, slab widths and cover plate lengths were spot checked for each bridge. They were within ± 0.5 in. of those shown in the plans (Section 7.1).

Slab thickness and cross-sectional dimensions of beams larger than 1 in. were measured to the nearest 0.01 in., and cross-sectional dimensions smaller than 1 in. were usually measured to the nearest 0.001 in. They were determined systematically at intervals of 10 ft or less along each bridge.

At each interval, the slab thickness was measured at four transverse locations. The thickness of the top and bottom flanges, the depth of the web, the depth of the stem of reinforced concrete beams, and the depth of the beam below the slab were measured on both sides of the web. The web thickness and the width of the stem of reinforced concrete beams were measured at two or three depths. The flange width was measured for both the top and bottom flange. The thickness of cover plates for the steel beams was determined by measurements of the test coupons.

Tables 97 and 98 give the mean dimensions and computed cross-section properties for the

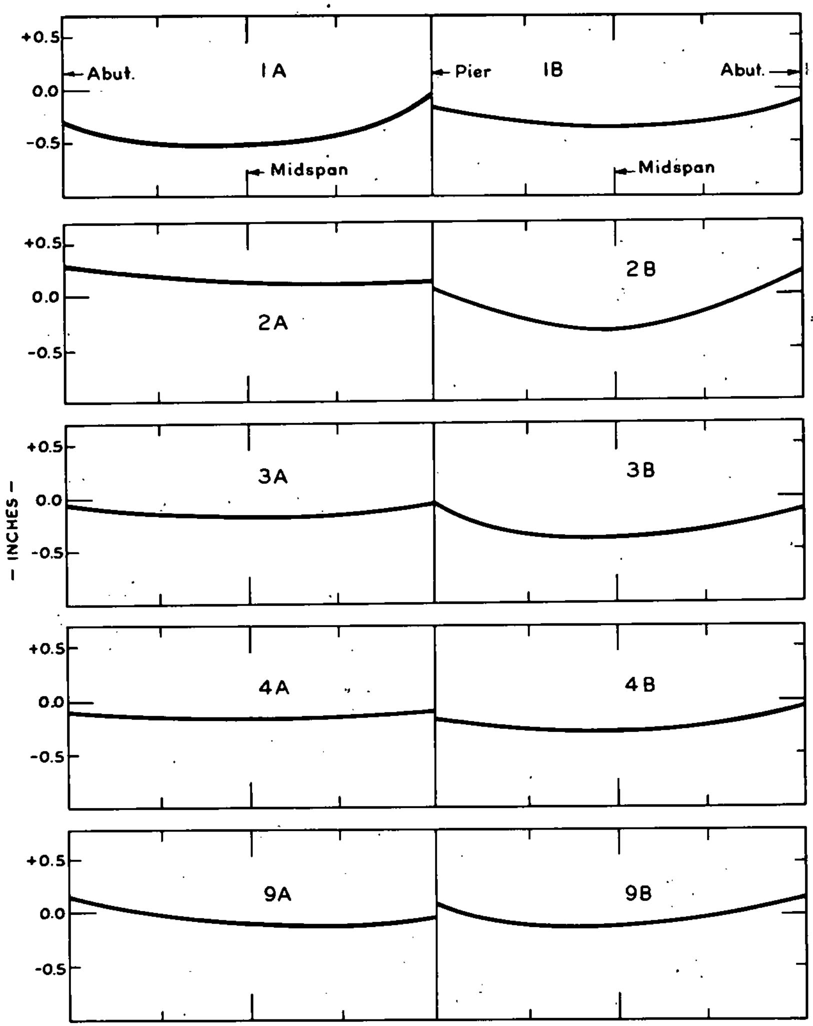


Figure 150. Longitudinal profiles of steel beam bridges.

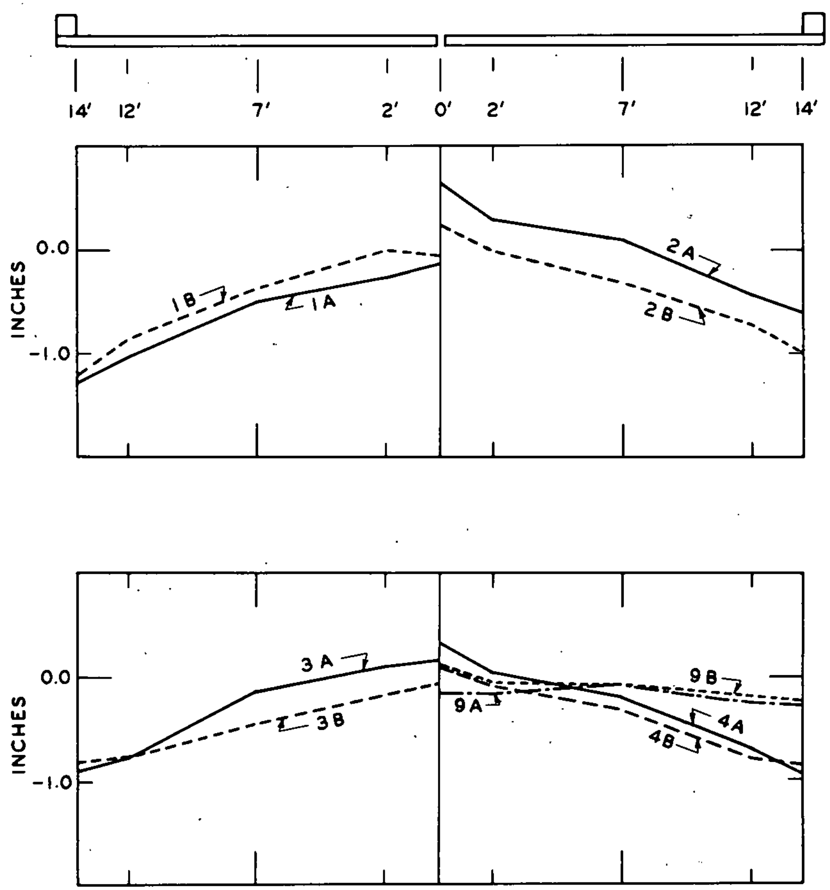


Figure 151. Transverse profiles of steel beam bridges at midspan.

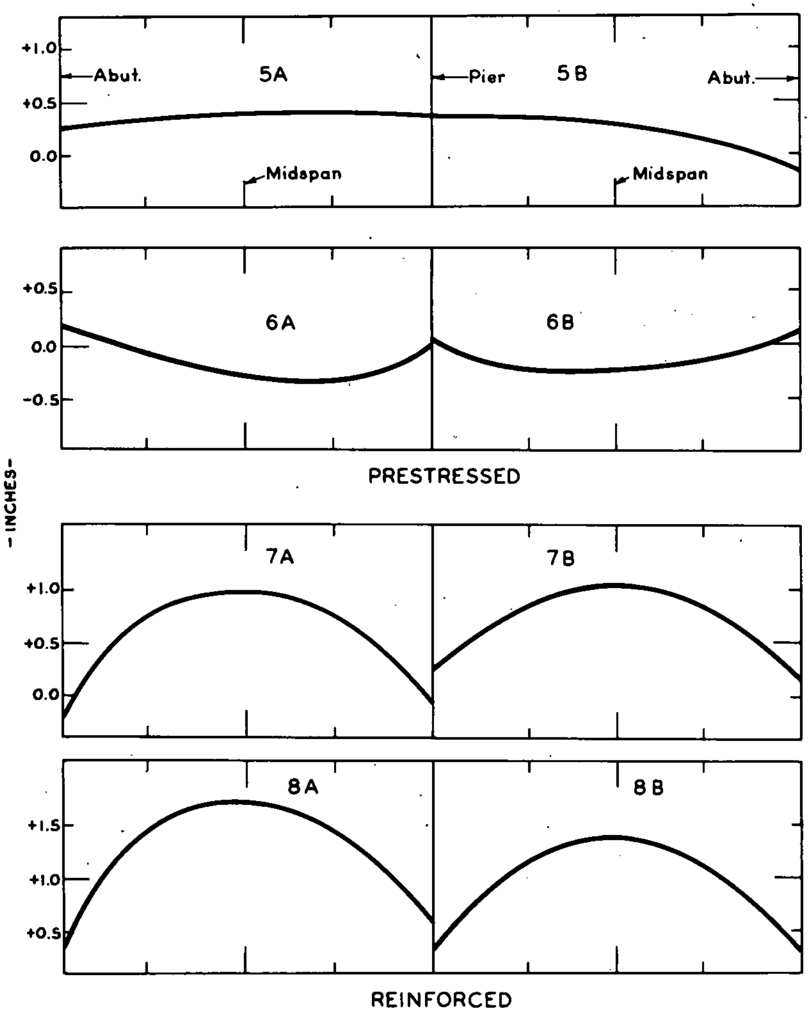


Figure 152. Longitudinal profiles of prestressed and reinforced concrete beam bridges.

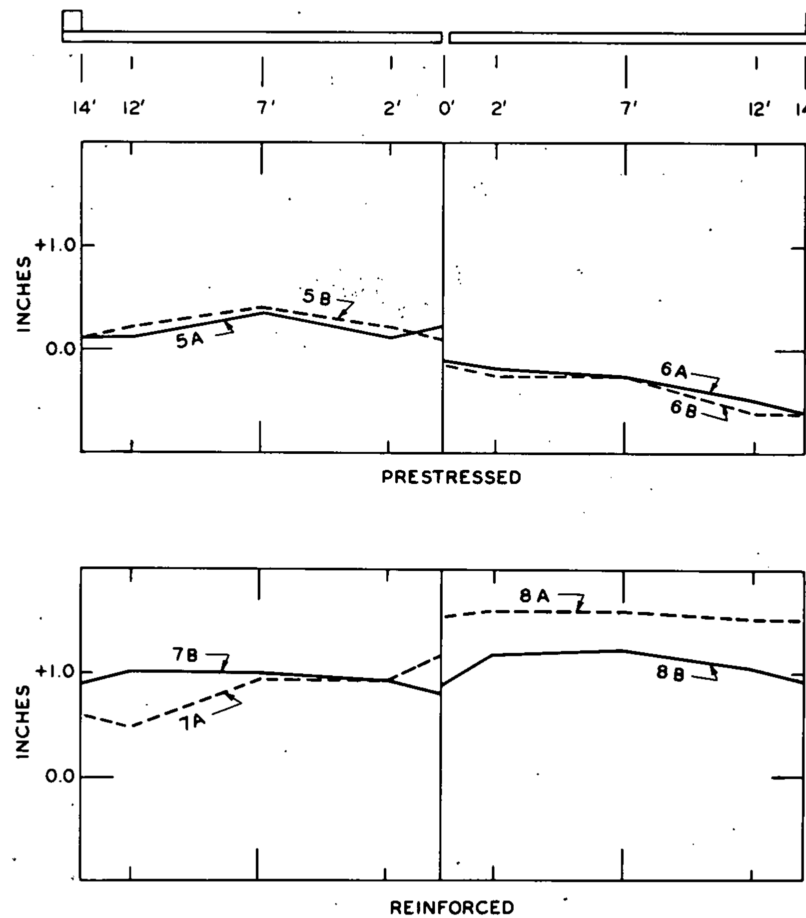


Figure 153. Transverse profiles of concrete beam bridges at midspan.

beams in each of the steel beam bridges. Figure 146 shows the distributions of the measurements on all steel beams.

It may be noted that the sum of the depth of the wide flange beam in Table 97 and the thickness of the cover plate in Table 98 is in every case smaller than the over-all depth given in Table 98. This difference represents a gap between the cover plate and the beam. The measured width of cover plates is not included in the tables. This width could not be measured accurately in the completed structure because of the weld; however, spot checks indicated that it may be taken as 6 in.

Table 99 gives the mean measured dimensions and computed cross-section properties for the prestressed concrete beams of bridges 5A, 5B, 6A and 6B. Other exterior dimensions and the spacings of the prestressing steel within the beams were considered constant for the three beams in each bridge and are shown in Figures 127 and 128 of Section 7.1.3.

Figure 147 depicts the distributions of the measurements on the prestressed concrete beams. The form ties at the center of one beam were removed too early, and forms spread at the top. This accounts for the wider variation of a small percentage of top flange-width and web-thickness measurements.

The measurements of the dimensions of the reinforced concrete beams are summarized in Table 100 and shown graphically in Figure 148. The spacing of the tensioning steel and other exterior dimensions not included in the table were considered constant for all four bridges and are shown in Figure 132 of Section 7.1.4.

The measurements of slab thickness and area of reinforcing bars are summarized by bridges in Table 101. Figure 149 shows the distribution of slab thickness measurements for all bridges.

7.5.2 Slab Profiles

The profiles of the slabs of the test bridges were determined immediately after the superstructures were completed. Longitudinally, elevations were taken at the bearing points, the quarter points and at midspan. Transversely, they were obtained at the inside edge of the slab and at 2-, 7-, 12- and 14-ft intervals. All elevations were recorded to the nearest 0.001 ft.

The longitudinal and transverse profiles of the ten steel beam bridges, showing the variations from the theoretical elevations, are shown in Figures 150 and 151, respectively. The longitudinal profiles shown in Figure 150 are along the center beam of the bridges, and the transverse profiles are shown at midspan.

The longitudinal profiles along the center beam of the ten steel beam bridges varied from a relatively straight profile to a sag of approximately ½ in. at midspan. However, as shown in Figure 151, the transverse profiles at midspan of the original eight bridges were sloped. The exterior edges of these bridges were lower than interior edges by approximately ¾ to 1¼ in. For construction purposes, the deflection diagrams were assumed to be the same for all three beams in a bridge, but, due to the difference in the width of the slab overhangs, the deflections were greatest in the exterior beams and least in the interior beams. The deflection diagrams for bridges 9A and 9B were computed separately for each beam, and the transverse profiles at midspan of the completed structures were relatively flat.

Figures 152 and 153 show the longitudinal profiles along the center beams and the transverse profiles at midspan for the completed prestressed concrete and reinforced concrete beam bridges.

As shown in the figures, the center beams of prestressed concrete beam bridges 6A and 6B had a sag at midspan of slightly less than ½ in. while the sag was a little greater in the exterior beams and a little less in the interior beams. Based on these results, adjustments were made in the deflection diagrams for bridges 5A and 5B. The effects of the adjustments are evident from comparing the profiles of the four bridges.

The reinforced concrete beam bridges 7A, 7B, 8A and 8B were constructed with camber. In computing the deflection diagrams, consideration was given to the effects of creep of concrete as well as the instantaneous dead load effects. Therefore, the slabs of the completed structures retained part of the camber at the time the forms and falsework were removed. The camber at midspan varied from approximately ¾ to 1¼ in.

Appendix A

STATISTICAL ANALYSIS OF COMPACTION DATA FOR CONSTRUCTION CONTROL

Compaction control for the embankment, subbase and base course construction at the AASHO Road Test was based upon acceptance of individual layers of material within construction blocks. Since it was impossible to sample the entire area of a layer of a construction block, a statistical sampling plan was developed which reduced the frequency of sampling and permitted the compacted condition of the layer to be estimated. The plan, in simple terms, made it possible to compute from the results of tests on a limited number of samples a valid estimate of the mean percent compaction and the percent of the compacted area above and below the specification limits for each layer of material.

The development of the procedure and the derivation of the tables used for the analysis are beyond the scope of this report. The following discussion describes briefly how the procedure works and how it was used on the Road Test.

For convenience of presentation, assume that the rectangle shown in Figure 1-A represents a construction block-lift of embankment, that the eight divisions represent the limits of the eight test sections included in the block, and that each square represents one possible location to be

1			2			3			4		
1 95	2 96	3 91	1 94	2 96	3 97	1 94	2 96	3 98	1 96	2 94	3 98
4 97	5 95	6 98	4 97	5 97	6 96	4 97	5 97	6 95	4 93	5 98	6 97
7 98	8 99	9 99	7 97	8 95	9 96	7 97	8 98	9 93	7 96	8 98	9 96
10 95	11 96	12 99	10 100	11 94	12 100	10 99	11 95	12 101	10 101	11 95	12 99
1 99	2 94	3 98	1 97	2 98	3 96	1 100	2 96	3 99	1 93	2 97	3 102
4 94	5 97	6 97	4 96	5 94	6 97	4 96	5 98	6 100	4 92	5 94	6 96
7 96	8 100	9 97	7 102	8 101	9 95	7 95	8 99	9 95	7 98	8 94	9 98
10 98	11 100	12 101	10 101	11 103	12 102	10 97	11 97	12 99	10 97	11 99	12 97
5			6			7			8		

Figure 1–A. Example of universe, percent compaction.

TABLE 1–A

RANDOM SAMPLE OF $n = 8$ FROM THE UNIVERSE OF FIGURE 1–A

Test Section No.	Sampling Point	Value
1	1	95
2	12	100
3	12	101
4	1	96
5	5	97
6	11	103
7	7	95
8	9	98

$\Sigma x = (95 + 100 + \cdots + 98)$	785
$\bar{x} = \Sigma x/n$	98.1
$\Sigma x^2 = (95^2 + 100^2 + \cdots + 98^2)$	77,089
$(\Sigma x)^2/N = 785^2/8$	77,028
$SS = \Sigma x^2 - (\Sigma x)^2/N$	61
$S^2 = SS/N{-}1$	8.7143
S = Standard deviation $= \sqrt{S^2}$	2.95

tested. What is shown in the figure then may be said to represent the universe for the block-lift. The numbers shown in the centers of the 96 squares are the results that would be obtained if the complete universe was tested.

The problem is to sample and test the fictitious block-lift in Figure 1-A in order to determine an estimate of its compacted condition for acceptance or rejection.

First, it is necessary to determine the number of samples that can be taken. Since the larger the number of samples taken the more precise the estimate of the universe, the largest number of samples should be taken that time and money permit. For the block-lift shown in Figure 1-A, it is assumed that one sample will be taken for each test section, making a total of eight samples. This method restricts the sampling so that if there is a uniform variation from low to high densities over the area, for example, the chances of completely missing the low or high area are greatly reduced. To estimate the condition of the block-lift without bias, the location of each sample should be randomly selected from the 12 possible locations in the respective test sections. The selected locations may be determined from tables of random numbers or by simply drawing the numbers from a hat.

A randomly selected sample of eight for the universe shown in Figure 1-A is given in Table 1-A, which includes the location and value for each sample, and the estimated mean percent compaction ($\bar{x}$) and the standard deviation (S). If the specifications required the percent compaction to be 95 or greater, the analysis should give the estimated percent of the universe that is below 95 percent compaction. For this analysis it is necessary to compute a value of $C = (\bar{x} - 95) \div S$; and then obtain the estimated percent of the universe that is below 95 from published tables.* The estimated "percent out" (P) corresponding to the value of C and the sample size is read directly from the tables. For the sample shown in Table 1-A, the value of C is $\frac{98.1 - 95}{2.95}$, or 1.06, and the estimated percent out (P) from the table is 14.5.

* Lieberman, G. J. and Resnikoff, G. J. "Sampling Plans for Inspection by Variables," *Jour. of Amer. Statist. Assoc.*, June 1955.

TABLE 2–A

TABLE FOR SAMPLING PLANS BASED ON UNKNOWN STANDARD DEVIATIONS[1]

Sample Size	Allowable Estimated Percent Defective for Acceptable Quality Level of:						
	1.0	2.5	4.0	6.5	10.0	15.0	20.0[2]
3		7.6	18.9	26.9	33.7	40.5	
4	1.5	10.9	16.4	22.9	29.4	36.9	
5	3.3	9.8	14.4	20.2	26.6	34.0	40
7	3.6	8.4	12.2	17.4	23.3	30.5	38
10	3.3	7.3	10.5	15.2	20.7	27.6	34
15	3.0	6.6	9.5	13.7	18.9	25.6	32
20	3.0	6.2	8.9	13.0	18.0	24.5	30
30	2.8	5.9	8.5	12.4	17.2	23.6	28
40	2.7	5.6	8.1	11.8	16.6	22.9	28
50	2.5	5.2	7.6	11.2	15.9	22.0	28

[1] Taken from G. J. Lieberman and G. J. Resnikoff, "Sampling Plans for Inspection of Variables" *Jour. Amer. Statistical Assn.* June 1955).

[2] Extrapolated values.

TABLE 3–A

TEN RANDOM SAMPLES OF n = 8 FROM THE UNIVERSE OF FIGURE 1-A

Test Section No.	Sample Number[1]									
	1	2	3	4	5	6	7	8	9	10
1	$^{1}/95$	$^{1}/95$	$^{11}/96$	$^{9}/99$	$^{7}/98$	$^{9}/99$	$^{12}/99$	$^{9}/99$	$^{5}/95$	$^{8}/99$
2	$^{12}/100$	$^{11}/94$	$^{6}/96$	$^{11}/94$	$^{5}/97$	$^{2}/96$	$^{3}/97$	$^{2}/96$	$^{2}/96$	$^{7}/97$
3	$^{12}/101$	$^{11}/95$	$^{11}/95$	$^{1}/94$	$^{1}/94$	$^{11}/95$	$^{8}/98$	$^{9}/93$	$^{11}/95$	$^{2}/96$
4	$^{1}/96$	$^{3}/98$	$^{1}/96$	$^{3}/98$	$^{5}/98$	$^{4}/93$	$^{9}/96$	$^{11}/95$	$^{5}/98$	$^{10}/101$
5	$^{6}/97$	$^{4}/94$	$^{10}/98$	$^{9}/97$	$^{5}/97$	$^{10}/98$	$^{9}/97$	$^{2}/94$	$^{11}/100$	$^{9}/97$
6	$^{11}/103$	$^{10}/101$	$^{11}/103$	$^{5}/94$	$^{3}/96$	$^{9}/95$	$^{9}/95$	$^{9}/95$	$^{6}/97$	$^{12}/102$
7	$^{7}/95$	$^{11}/97$	$^{5}/98$	$^{5}/98$	$^{10}/97$	$^{10}/97$	$^{12}/99$	$^{4}/96$	$^{2}/96$	$^{8}/99$
8	$^{9}/98$	$^{12}/97$	$^{4}/92$	$^{7}/98$	$^{3}/102$	$^{2}/97$	$^{10}/97$	$^{11}/99$	$^{4}/92$	$^{4}/92$
$\bar{x}$	98.1	96.4	96.8	96.5	97.4	96.3	97.3	95.9	96.1	97.9
S	2.95	2.39	3.16	2.14	2.27	1.93	1.41	2.17	2.36	3.14
P[2]	14.5	28.9	29.5	24.8	15.7	26.5	4.0	35.2	32.3	18.3

[1] Numerator is sample location (see Fig. 1–A).
[2] Estimated percentage out.

To make use of this analysis, it is conventional to specify an acceptable quality level (AQL) which is simply the universe percent defective, above which the universe is to be rejected. Since the sample can only produce an estimate of the universe, the allowable estimated percent out must be somewhat greater than the AQL in order to provide for sample fluctuations and reduce the risk that the whole universe will be rejected when it should be accepted. The estimated percent out that should be allowed is obtained from a table such as that shown in Table 2-A. If an AQL of 20.0 is considered acceptable for the universe in Figure 1-A, then the block-lift would be acceptable if the estimated percent out does not exceed 37, when using a sample size of eight.

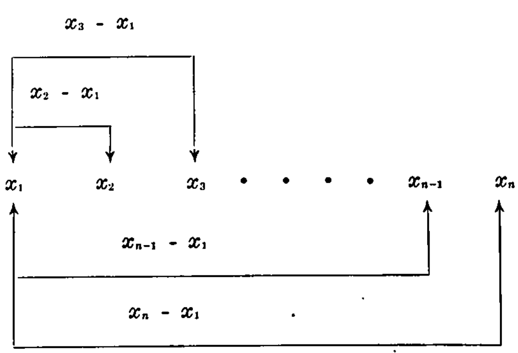
The information contained in Table 3-A includes ten sets of eight samples from the universe in Figure 1-A. The estimated percents out for the ten sets ranged from 4.0 to 35.2 but none were over the maximum allowable of 37. An analysis of the complete universe indicates a mean percent compaction of 97.1, a standard deviation of 2.39, and 19.9 percent of the samples below 95 percent compaction.

It is possible through this type of sampling plan to obtain extreme values that are not valid members of the universe sampled. Extreme values may arise, for instance, from gross errors in the testing technique. A suspected extreme value may be tested by the significance tests shown in Table 4-A to infer (at some significance level) whether or not it is a bona fide member of the universe. The significance level chosen represents the risk that the value will be rejected even though it is a bona fide member of the universe. As an illustration, the value shown in Table 1-A for section 6 happened to be the highest value in the universe. If this value were assumed to be an extreme value, the significance test utilizing Table 4-A, would be as follows: $\dfrac{101 - 103}{96 - 103} = 0.286$. Since this ratio does not exceed that shown in the table for the 5 percent level or the 1 percent level, the value is assumed to be a member of the

TABLE 4–A

CRITERIA FOR REJECTION OF EXTREME VALUES

Number of Samples	Criterion for Extreme, x_1	Critical Value for:	
		5% Risk	1% Risk
3		0.941	0.988
4	$\dfrac{x_2 - x_1}{x_n - x_1}$	0.765	0.889
5		0.642	0.780
6		0.560	0.698
7		0.507	0.637
8		0.544	0.683
9	$\dfrac{x_2 - x_1}{x_{n-1} - x_1}$	0.512	0.635
10		0.477	0.597
11	$\dfrac{x_3 - x_1}{x_{n-1} - x_1}$	0.576	0.679
12		0.546	0.642



universe and should not be rejected. If the ratio had exceeded that shown in the table for either level, the value could have been rejected from the sample as an extreme value and the percent out computed from the remaining seven tests.

The specifications for compaction control on the Road Test set both upper and lower limits on percent compaction for the embankment, subbase and base. Thus the analysis involved computing both the estimated percent above and the estimated percent below the specification requirements (P_u and P_l) and combining them to obtain the total estimated percent out for each block-lift. Controlling embankment construction to the degree specified for the Road Test had not previously been attempted. Also the procedures were different from those normally used. Thus, it was not known what AQL would be satisfactory, nor was it known how well the construction could be controlled. Therefore, rather than attempt to establish an AQL, arbitrary values were selected for the maximum allowable estimated percent out after the job had been running for a short time. These values were selected as practical values that should be obtained without rejecting an excessive amount of the work.

The maximum allowable $P_u + P_l$ for embankment construction was set at 45 percent and was later lowered to 40 percent for the top lifts. The results of this construction indicated that even a lower allowable value was feasible, and, consequently, the $P_u + P_l$ for the subbase and base was reduced to 35 percent. Figure 47, Section 2.3, shows that the mean estimated percent out for the 875 block-lifts of embankment was 20.8, and only 12 percent of the block-lifts were above 35 percent out. For the most part this 12 percent occurred at the start of the embankment construction.

Appendix B

ONE-POINT METHOD FOR DETERMINING MAXIMUM DRY DENSITY

Embankment on test tangents was constructed in block-lifts, approximately 4 in. thick when compacted. It was required for each construction block that each lift be tested and accepted before commencing construction of the succeeding lift. To prevent undue delay between the completion of a block-lift and the starting of the next, the testing was accelerated by the development of a one-point procedure for determining maximum dry density of the embankment soil. The one-point procedure was based on previous work by the Wyoming State Highway Department and the Ohio Department of Highways. It made use of the Proctor penetration needle and a family of moisture-density-Proctor needle reading curves.

The procedure required a single molded specimen. First it was necessary to determine the wet density, moisture content and Proctor needle reading for the specimen. With this information it was possible to obtain the wet density corresponding to maximum dry density and the adjustment of the moisture content of the specimen to optimum from the family of curves, and to compute the corresponding maximum dry density.

The family of curves used at the Road Test was developed from standard tests on the A-6 embankment soil. In the portion of the family shown (Fig. 2-B), the curves are plots of wet density moisture content and needle reading. The optimum conditions line is based on maximum dry density and coincides with the 600-psi needle reading line for that portion. However, it extends below the 600-psi line on the upper part of the complete family of curves and above the 600-psi line on the lower part.

The preparation of a specimen involved pulverizing a soil sample, removing the plus ¼-in. material, and adjusting the moisture content of the sample to within a few percentage points of optimum. The soil was then compacted in three layers in a 4-inch diameter mold, using a 5.5-lb hammer and a 12-in. drop.

To illustrate the procedure, a sample problem is included (Fig. 2-B). The determinations on the compacted specimen indicated a wet density of 132.2 pcf, a needle reading of 700 psi, and a moisture content of 14.8 percent. The wet density and needle reading are used with the family of curves to determine the wet density corresponding to maximum dry density and the correction needed to adjust the moisture content of the specimen to optimum. The intersection of the wet density of the specimen with the corresponding needle reading determined the appropriate moisture-density curve for the soil sample. The intersection of this moisture-density curve with the optimum conditions line determined the wet density corresponding to the maximum dry density of the sample. The difference in moisture contents read on the moisture scale at the points of intersection of the appropriate moisture-density curve with the optimum conditions line and the wet density of the specimen with the corresponding needle reading determined the correction needed to adjust the moisture content of the specimen to optimum. Thus, the wet density corresponding to the maximum dry density for the sample problem is 132.8 pcf and the moisture correction is + 0.4 percentage points. The optimum moisture content is then equal to 14.8 + 0.4, or 15.2 percent, and the maximum dry density is 132.8 ÷ 1.152, or 115.3 pcf.

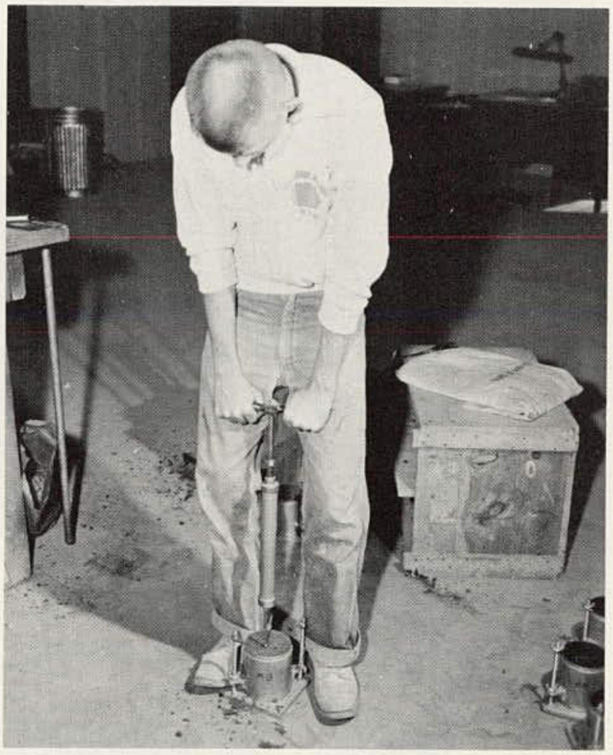


Figure 1–B. Obtaining Proctor needle reading on molded soil samples.

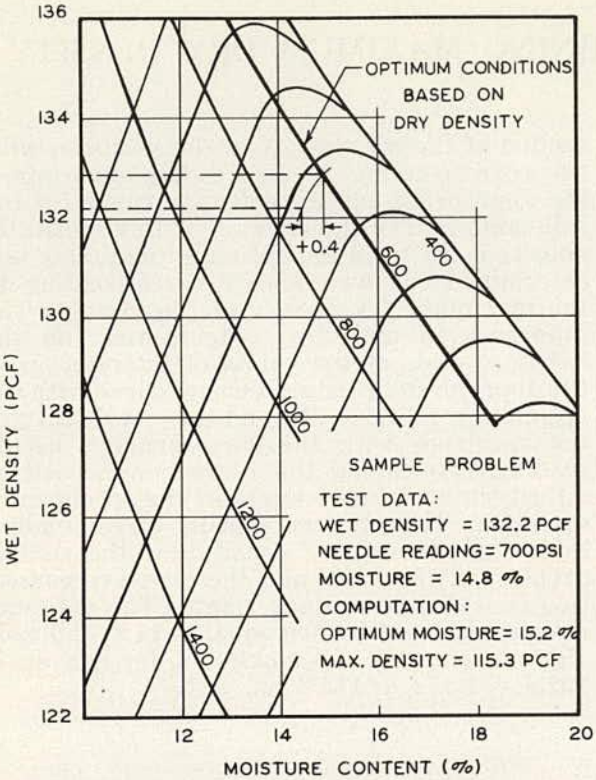


Figure 2–B. Moisture-density-Proctor needle reading curves.

In addition to the initial development work, the one-point method was further tested in the laboratory to check upon its reliability. The maximum dry density for the 62 samples used in this study was determined by both the standard AASHO procedure (T99-49) and the one-point method. The mean maximum dry density and optimum moisture content of the 62 samples were 116.7 pcf and 15.2 percent by the standard procedure, and 116.5 pcf and 15.4 percent by the one-point method. The mean algebraic differences between the two methods were 0.2 pcf in dry density and 0.2 percentage points in moisture content, and the corresponding standard deviations of these differences were 0.84 pcf and 0.62 percentage points.

Figure 3–B. Preparing molded soil specimen.

Appendix C

COOPERATIVE MATERIALS TESTING PROGRAM

During the early phases of construction, the project staff recognized that it could not provide all the information that might be requested on the properties of the materials used in the Road Test. Therefore, a program was established for cooperative testing of the Road Test materials by interested agencies. The primary purpose was to provide the agencies with an opportunity to acquire an intimate knowledge of the materials that would be of value to them in applying the Road Test findings to their areas. The program also presented an opportunity for the staff to secure information on the materials that could not be obtained with the equipment available at the project site. Over 60 agencies participated in the program, representing state highway departments, universities, Canadian provincial highway departments, the Federal Government, and other organizations.

The materials included in the program were the embankment soil, the subbase and the crushed stone base. Representative samples of large stocks of these materials obtained during construction were shipped to the agencies for testing.

A standard report form was provided. However, each agency was free to select its own tests and test methods, and to report as much or as little as it desired.

A report on the complete program entitled "Cooperative Materials Testing Program at the AASHO Road Test", by James F. Shook and H. Y. Fang, was presented at the 40th Annual Meeting of the Highway Research Board (HRB Special Report 66).

Summaries of data from the program are included in this appendix to indicate the main characteristics of the materials and selected properties for design obtained by the cooperating agencies. In many cases, the summaries represent selections of data from the available information. Exclusions were made for a variety of reasons, including great variability in either method or results, incompleteness, or a lack of applicability.

It will be noted that in some instances the data presented in this appendix varies somewhat from that obtained at the Road Test for materials and construction control. However, the tests by the agencies were run on representative samples of a large stock of each of the materials, while the data at the project were obtained by daily sampling and testing. All the variability that existed within each of the materials could not be contained in one sample. Also these differences indicate the inability of the test procedures to reproduce the same results exactly, since there were differences among individual agencies when using the same test procedure on samples obtained from one stock of material.

Selected classification and quality tests for the three materials are summarized in Table 1-C. Included are the Atterberg limits, moisture-density relationships at optimum conditions, apparent specific gravities, selected sizes for mechanical analyses, and results of sand equivalent, Los Angeles abrasion and soundness tests.

Most laboratories classified the embankment soil in the A-6 category, with variations in group index values. These variations largely reflected differences in liquid and plastic limits. Of the 54 agencies reporting, one reported a plasticity index below 10 and one above 16.

Most of the reporting agencies classified the minus No. 40 fraction of the subbase and base as non-plastic. However, 17 percent gave PI values from 0.2 to 3.4 for the subbase and 29 percent from 0 to 4.3 for the base. In both cases one agency reported a minus PI.

Considerable variations in procedure were noted for the methods used to determine maximum density and optimum moisture content. Table 1-C includes the mean value of those tests reportedly run according to AASHO Method T99-49 (5.5-lb hammer, 12-in. drop, 3 layers, 25 blows per layer in a 4-in. mold) or the equivalent ASTM Method D 698 and the AASHO Method T180-57 (10-lb hammer, 18-in. drop, 5 layers, 56 blows per layer in a 4-in. mold).

Variant procedures not always noted but which may have had an influence on the reported values included re-use of the sample, maximum size of the subbase or base aggregate used, and size of mold. In arriving at the means in Table 1-C, no distinction was made between those who reported the above and those who did not. However, values corrected mathematically for plus No. 4 material were not included, nor were values determined with different sizes and weights of hammer or number of blows.

Table 2-C gives what may be considered design properties of the soil, subbase and base

TABLE 1–C

Summary of Material Characteristics[1, 2]

Item	Soil No. Tests	Soil Mean	Soil Std. Dev.	Subbase No. Tests	Subbase Mean	Subbase Std. Dev.	Base No. Tests	Base Mean	Base Std. Dev.
Liquid limit	59	27.7	1.9						
Plastic limit	59	15.1	1.4						
Plasticity limit	59	12.6	1.8		N.P.			N.P.	
AASHO Classification (Group Index)	54	A–6(9)			A–1(0)				
Optimum moisture (%):									
AASHO T 99	44	13.5	1.0	35	7.7	1.0	27	7.6	1.2
AASHO T 180	8	10.4	1.0	10	7.1	0.6	8	6.6	0.4
Maximum density (pcf):									
AASHO T 99	44	119.2	2.2	35	133.1	3.3	27	137.9	4.2
AASHO T 180	8	128.8	2.4	10	137.4	2.2	8	142.1	2.5
Spec. gravity, apparent	53	2.72	0.04	34	2.70	0.05	37	2.74	0.05
Mech. anal.[3]:									
No. 4 sieve	58	96.6	2.2	48	73.5	4.5	43	48.9	7.0
No. 40 sieve	57	88.6	3.9	47	26.9	4.7	41	20.3	2.5
No. 200 sieve	58	75.5	3.2	29	8.1	2.0	24	9.9	1.7
0.02 mm	42	61.9	5.1	4	4	a	4	5	c
0.005 mm	47	40.3	5.4	4	2	b	4	3	d
Sand equivalent	1	6		5	51	e	5	46	f
Sodium sulfate soundness loss, 5 cycles							5	3.0	g
Los Angeles abrasion loss[4] (%)				6	31	h	18	27.3	2.2

[1] Latest standard AASHO test methods used (as of 1958).

[2] Ranges: a = 3-6, b = 2-4, c = 4-6, d = 2-4, e = 40-73, f = 41-54, g = 0.7-6.0, h = 25-35

[3] Percent finer than.

[4] 500 revolutions.

TABLE 2–C

Material Design Properties

Item	Soil No.	Soil Mean	Soil Range	Subbase No.	Subbase Mean	Subbase Range	Base No.	Base Mean	Base Range
Property at approximately T 99 optimum conditions:									
CBR, soaked, Corps of Engineers' procedure	8	5.0	4.0-6.7	5	58	32-86	5	100+	83–140
CBR, soaked, static method	1	1.7		2		5-33	3	200+	145-200+
Hveem stability, R-value	3	21[1]		9	78	69-84	7	83	81-86
Kansas triaxial, modulus of deformation (psi)		1,300			8,000			10,000	
Texas classification		5.6 (96% max. dens.)			3.7 (102% max. dens.)				
Property at approximately T 180 optimum conditions:									
CBR, soaked, Corps of Engineers' procedure	5	18	10-24	5	100+	86-163	3	186	135–233
CBR, soaked, static method				1	20				
Texas classification		4.7 (97% max. dens.)						1.0 (102% max. dens.)	

[1] Determined at 300-psi exudation pressure.

materials. Factors included are from CBR, Hveem stability (R-values), Kansas triaxial (modulus of deformation), and Texas triaxial (classification number) tests. These tests were selected from the group available because the molding moisture content and density were close to the reported average for the standard, T99, or modified, T180, conditions. No extreme values were included.

Of the 28 reports showing CBR data for the

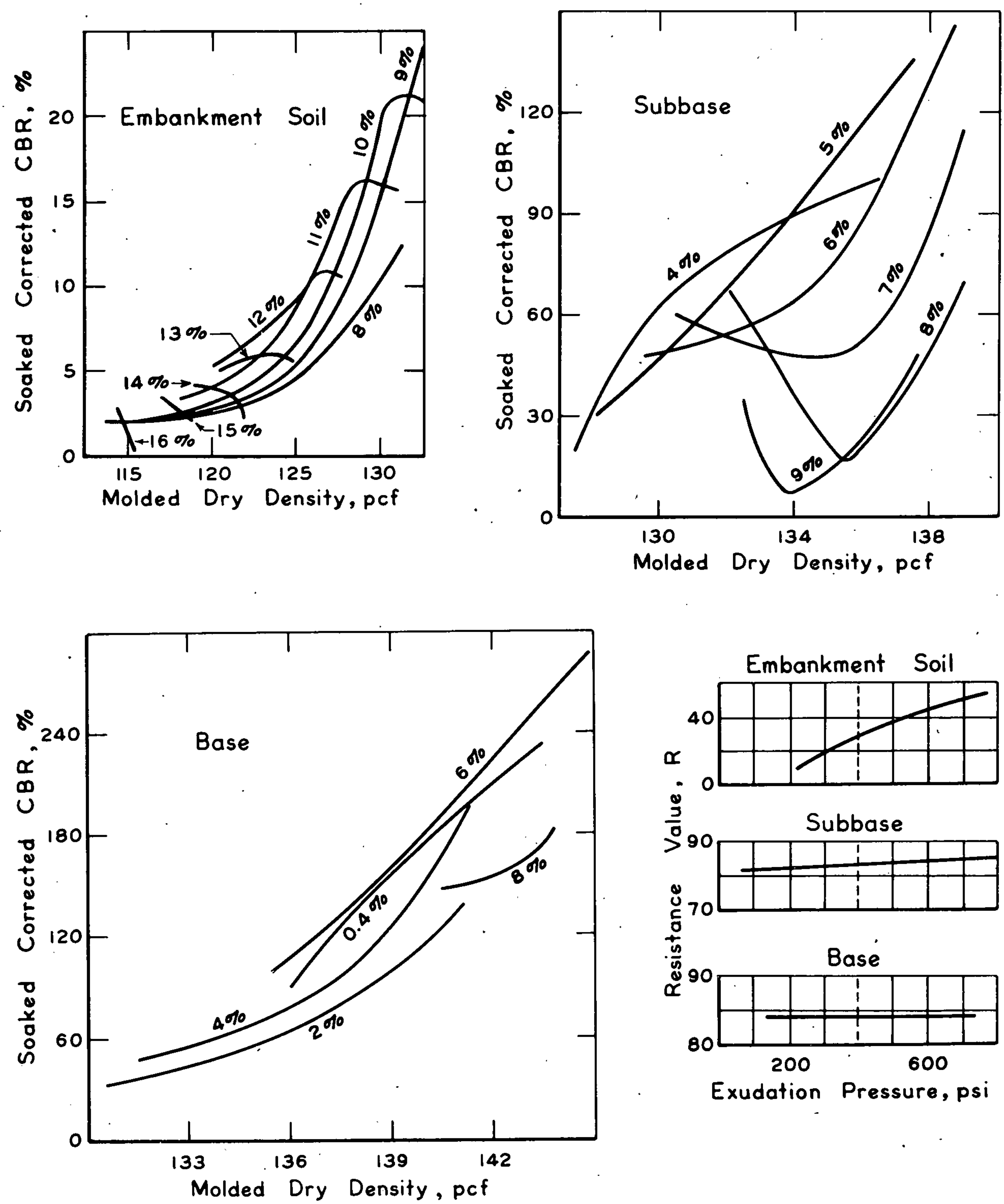


Figure 1–C. CBR and Hveem stabilometer curves.

embankment soil, 14 were included in three categories in Table 2-C. Twelve out of 26 subbase reports and 11 out of 24 base reports were included.

Hveem stabilometer test data for the embankment soil also varied appreciably from agency to agency. There was more uniformity for the granular materials. In Table 2-C, averages of reported values at 400-psi exudation pressure were used to obtain the subbase and base R-values. For the soil, these values were taken at 300-psi exudation pressure after considering the plots of moisture content versus exudation pressure for the three agencies.

Several agencies reported the results of triaxial tests, a few reporting data for all three materials. Since only two, Kansas and Texas, reported explicitly on design tests, they alone have been included in Table 2-C.

Figure 1-C shows curves constructed from data on CBR tests supplied by the Corps of Engineers and from data on Hveem stabilometer tests supplied by the State of California for the embankment soil, subbase and base. The CBR curves are plots of CBR values versus dry density at various molding moisture contents. The stability curves are plots of R-values exudation pressure. The CBR specimens were compacted in 5 layers using a 10-lb hammer and an 18-in. drop. They were soaked for four days prior to testing. A 20-lb surcharge weight was used both during soaking and testing.

Appendix D

TESTS ON ROAD TEST MATERIALS BY THE BUREAU OF PUBLIC ROADS

At the request of the project staff, the Bureau of Public Roads, Division of Physical Research, conducted tests on the A-6 embankment soil and on the materials used in the asphaltic concrete and portland cement concrete. The tests were selected to provide information not being obtained on the project.

(a) **Embankment Soil.**

Samples of the yellow-brown A-6 soil were obtained from borrow pits 1 and 2 in the fall of 1956. The samples were tested to determine the soil characteristics and design properties. The results of X-ray diffraction tests indicated that the clay fractions of the two samples were predominantly illite (about 60 percent) with about 30 percent chlorite and 5 to 10 percent montmorillonite. The results further indicated that the two samples were almost identical from the clay mineral standpoint, but the sample from borrow pit 1 appeared to contain a little more montmorillonite than did that from borrow pit 2.

Characteristics of the soil samples, given in Table 1-D, include moisture-density relationships and Atterberg limits. The results of direct shear, triaxial compression, CBR, and Hveem stabilometer tests are summarized in Table 2-D.

(b) **Asphaltic Concrete Materials.**

The samples of the asphalt cement and aggregates were representative of the materials used throughout the production of the asphaltic concrete, July to September 1958. Samples of the asphalt cement (85-100 penetration grade) were obtained from the feeder lines, and samples of the mineral aggregates from the hot bins of the asphalt plants.

TABLE 1–D

CHARACTERISTICS OF EMBANKMENT SOIL[1]

Borrow Pit	AASHO Class.	Max. Dry Density (pcf)	Opt. Moist. Content (%)	Liquid Limit	Plastic Index	Shrinkage	
						Limit	Ratio
1	A–6(9)	118	14	28	12	16	1.94
2	A–6(10)	117	15	32	16	16	1.92

[1] Latest standard AASHO test methods used (as of 1958).

TABLE 2–D

PROPERTIES OF EMBANKMENT SOIL

Borrow Pit	Density[1] (%)	Direct Shear Tests		Triaxial Compr. Tests			CBR Value, Soaked	Hveem Stabil. Tests			
								At 400-Psi Exud. Press.		At 300-Psi Exud. Press.	
		Angle of Internal Friction (°)	Cohesion (psi)	Angle of Internal Friction (°)	Cohesion (psi)	Unconf. Compr. Str. (psi)		Density[1] (%)	R-Value	Density[1] (%)	R-Value
1	95	27.5	0.0	31.0	11.1	38.8	2.7	104	23	103	9.5
	100	24.7	5.2	27.0	18.9	59.9	5.7				
2	95	29.2	0.0	20.5	8.9	30.3	2.5	103	21	100	12
	100	24.7	5.2	20.8	18.6	62.4	7.0				

[1] Percent of Standard AASHO (T 99-49).

TABLE 3-D

RESULTS OF TESTS ON ASPHALTIC CEMENT[1]

Characteristic	Mean Value
Specific gravity, 77 F/77 F	1.021
Penetration, 77 F, 100 g, 5 sec	88
Flash point (°F):	
Cleveland open cup	559
Pensky-Martens	510
Softening point (°F)	118
Ductility, 77 F, 5 cm/min (cm)	149
Loss on heating, 325 F, 5 hr (%)	0
Penetration of residue, 77 F, 100 g, 5 sec	77
Soluble in CCl_4 (%)	99.88
Spot test, standard naphtha	Neg.
Furol viscosity, 275 F	158
Thin film oven test, ⅛-in., 5 hr, 325 F:	
Change in weight (%)	+0.04
Tests on residue:	
Penetration, 77 F, 100 g, 5 sec	55
Percent of original penetration	63
Ductility, 77 F, 5 cm/min (cm)	152
Softening point (°F)	129

[1] Latest standard AASHO test methods used (as of 1958).

The results of the tests on the asphalt cement are given in Table 3-D. Tests for specific gravity and absorption of the composite aggregate portion of the binder and surface course mixtures are summarized in Table 4-D.

Tests on the coarse aggregates for abrasion (Los Angeles machine) and soundness (5 cycles

TABLE 4-D

SPECIFIC GRAVITY AND ABSORPTION VALUES FOR COMPOSITE AGGREGATE MIXTURE[1]

Aggregate	Specific Gravity			Absorp. (%)	
	Apparent	Bulk	Effective[2]	Water	Asphalt
Binder course	2.754	2.642	2.724	1.54	1.14
Surface course	2.755	2.648	2.704	1.47	0.79

[1] Latest standard AASHO test methods used (as of 1958).
[2] By Rice's saturation method.

TABLE 5-D

TESTS OF LABORATORY-PREPARED BINDER COURSE MIXTURE

Determination	Values					
(a) MARSHALL STABILITY						
Asphalt content (% of tot. mix.)	3.6	3.9	4.3	4.3	4.6	4.9
Type of compaction[1]	Mech.	Mech.	Mech.	Manual	Mech.	Mech.
Bulk specific gravity, compacted mix	2.449	2.460	2.465	2.433	2.464	2.457
Air voids, app. sp. gr. basis (%)	5.7	4.7	3.9	5.2	3.6	3.4
Air voids, effective sp. gr.[2] basis (%)	4.8	3.8	3.0	4.3	2.6	2.5
Voids filled with asphalt (%)						
Apparent sp. gr. aggr. basis	60	67	73	66	75	78
Bulk sp. gr. aggr. basis	98	99	100	86	100	100
Effective sp. gr. aggr., corrected[3]	55	64	72	64	76	79
Marshall stability (lb)	1580	1609	1446	1363	1328	1207
Flow value (0.01 in.)	9	10	11	12	12	13
(b) UNCONFINED COMPRESSION AND IMMERSION COMPRESSION TESTS						
Asphalt content (% of tot. mix)	3.6	3.9	4.3	4.6	4.9	
Bulk specific gravity, compacted mix	2.376	2.391	2.421	2.433	2.441	
Air voids, compacted mix (%)	8.0	6.8	5.0	4.0	3.2	
Compressive strength (psi)	268	271	273	275	270	
Retained strength[4] (%)	—	—	90	—	—	
(c) HVEEM STABILITY TEST						
Asphalt content (% of tot. mix)	3.6	3.9	4.3	4.6	4.9	
Stability value	50	51	46	42	41	
Cohesiometer value	243	269	280	285	277	

[1] 70 blows per face for mechanical compaction; 50 blows per face for manual compaction.
[2] Determined by Rice's saturation method.
[3] Computed by using bulk specific gravity of aggregate for total voids and using effective specific gravity of aggregates for air voids.
[4] After soaking in water at 120F for four days.

sodium sulphate) gave a weighted average loss of 23 percent and 4.9 percent, respectively.

Sand equivalent tests on the fine aggregate indicated the quality of the blend of the two sands to be satisfactory. The values were 87 for the coarse sand and 39 for the fine.

Binder and surface course mixtures were prepared in the laboratory at varying asphalt contents to meet the job-mix gradation formulas. Specimens were molded from these mixtures and tested for stability by the Marshall, Hveem, and unconfined compression test methods. The results are given in Tables 5-D and 6-D.

In preparing the Marshall test specimens, some were compacted with a mechanical Marshall compactor and others by a hand-operated hammer. The comparative effects of the two types of compaction are illustrated by the test results.

The immersion compression tests demonstrated that the asphaltic concrete mixtures were highly resistant to the action of water.

(c) **Portland Cement Concrete Materials.**

Two samples of the portland cement and of the fine and coarse aggregates were obtained to represent materials used at the beginning and near the end of paving on test tangents.

Chemical and physical properties of the Type I portland cement are summarized in Table 7-D. The values represent averages of the results for the two samples.

Tests for potential alkali-reactivity were conducted on the aggregates and cement. These tests indicated that the aggregates were nonreactive with the cement (Table 8-D). Each value of expansion is the average of nine individual tests.

The results of the tests for mechanical analysis and physical properties of the sand are given in Table 9-D. The Bureau of Public Roads also reported that no important amounts of deleterious substances were present in the fine aggregate.

A lithological analysis was made on the two sizes of gravel aggregate. The aggregate was

TABLE 6-D

TESTS OF LABORATORY-PREPARED SURFACE COURSE MIXTURE

Determination	Values					
(a) MARSHALL STABILITY						
Asphalt content (% of tot. mix)	4.2	4.7	5.1	5.1	5.6	6.0
Type of compaction [1]	Mech.	Mech.	Mech.	Manual	Mech.	Mech.
Bulk specific gravity, compacted mix	2.402	2.430	2.431	2.437	2.431	2.417
Air voids, app. sp. gr. basis (%)	6.6	4.8	4.1	3.8	3.4	3.3
Air voids, effective sp. gr.[2] basis (%)	5.0	3.2	2.5	2.3	1.9	1.8
Voids filled with asphalt (%):						
Apparent sp. gr. aggr. basis	60	70	75	76	80	81
Bulk sp. gr. aggr., corrected [3]	76	89	95	97	100	100
	62	74	81	82	86	88
Marshall stability (lb)	1810	1923	1678	1734	1493	1167
Flow value (0.01 in.)	9	9	10	13	12	16
(b) UNCONFINED COMPRESSION AND IMMERSION COMPRESSION TESTS						
Asphalt content (% of tot. mix)	4.2	4.7	5.1	5.6	6.0	
Bulk specific gravity, compacted mix	2.309	2.330	2.351	2.364	2.382	
Air voids, compacted mix (%)	8.7	7.0	5.6	4.5	2.8	
Compressive strength (psi)	302	310	308	290	270	
Retained strength[4] (%)	—	—	94	—	—	
(c) HVEEM STABILITY TEST						
Asphalt content (% of tot. mix)	4.2	4.7	5.7	5.6	6.0	
Stability value	49	46	35	8	2	
Cohesiometer value	233	279	322	196	155	

[1] 70 blows per face for mechanical compaction; 50 blows per face for manual compaction.

[2] Determined by Rice's saturation method.

[3] Computed by using bulk specific gravity of aggregate for total voids and using effective specific gravity of aggregates for air voids.

[4] After soaking in water at 120 F for four days.

TABLE 7–D

TESTS ON TYPE I PORTLAND CEMENT

Chemical Analysis		Physical Properties		Other Tests	
Determination	Percent	Determination	Value	Determination	Value
Silicon dioxide	20.3	Apparent specific gravity	3.14	Merriman sugar test:	
Aluminum oxide	6.3	Fineness, Blaine		Neutral point (ml)	34.1
Ferric oxide	3.1	(sq cm per g)	3497	Clear point (ml)	54.1
Calcium oxide	64.4	Time of set:		Test for false set	Neg.
Magnesium oxide	1.7	Initial	2 hr, 50 min		
Sulfur trioxide	2.4	Final	5 hr, 13 min		
Sodium oxide	0.32	Soundness, autoclave expansion	0.23		
Potassium oxide	0.58	Normal consistency	25.2		
Sodium and potassium oxide[1]	0.70	Mortar air content, W/C–67 (%)	8.0		
Chloroform soluble	0.009	Tensile strength (psi):			
Free lime, calcium oxide	0.00	3 days	395		
Water-soluble alkalies:		7 days	440		
Sodium oxide	0.10	28 days	490		
Potassium oxide	0.33	Compressive strength (psi):			
Loss on ignition	0.9	3 days	3170		
Insoluble residue	0.14	7 days	4610		
Tricalcium silicate	54	28 days	5940		
Dicalcium silicate	17				
Tricalcium aluminate	12				

[1] $Na_2O + 0.658\ K_2O$.

separated into six sizes and an analysis was made independently for each size (Table 10-D). The gravel aggregate was predominantly dolomite and argillaceous limestone, and contained smaller amounts of sandstone, chert, diabase, granite and quartz. Table 10-D also includes values obtained on each size for specific gravity and absorption.

Sodium sulfate soundness tests (AASHO T 104-57) on the gravel aggregate gave a weighted average loss of 14.4 percent after 5 cycles. The abrasion test by the Los Angeles machine indicated a weighted loss of 29.4 percent.

TABLE 8–D

RESULTS OF TESTS FOR POTENTIAL ALKALI-REACTIVITY[1]

Test Specimen	Expansion of Specimen (%)		
	At 3 Months	At 6 Months	At 12 Months
Cement with sand	0.02	0.03	0.03
Cement with coarse aggregate[2]	0.02	0.02	0.02
Cement with sand and coarse aggregate[3]	0.01	0.02	0.00

[1] ASTM C227–58T.

[2] Crushed to sand size.

[3] Particles larger than ¾ in. were crushed to pass ¾-in. size.

TABLE 9–D

TESTS ON P.C.C. FINE AGGREGATE[1]

(a) MECHANICAL ANALYSIS

	Percent Passing
⅜ in.	100
No. 4	99
No. 8	86
No. 16	67
No. 30	46
No. 50	13
No. 100	3
No. 200	1.2
Fineness modulus	2.86

(b) PHYSICAL PROPERTIES

Specific gravity:	
Bulk, dry basis	2.59
Bulk, saturated basis	2.64
Apparent, dry basis	2.71
Absorption (%)	1.9
Soundness, 5 cycles sodium sulfate (% loss)	8.1
Mortar strength ratio:	
Standard Ottawa sand	3.44
Sample sand	4.09

	3 Days	7 Days	28 Days
Tensile strength (psi)	470	615	700
Strength ratio	1.22	1.40	1.41
Compressive strength (psi)	2035	3510	5150
Strength ratio	0.98	1.03	1.02

[1] Latest standard AASHO test methods (as of 1958).

TABLE 10–D

TESTS ON P.C.C. COARSE AGGREGATES

Item	Retained on Sieve (% by wt.)					
	2-1½	1½-1	1-¾	¾-½	½-⅜	⅜-No. 4
(a) LITHOLOGICAL ANALYSIS						
Dolomite	38	47	38	64	59	59
Argillaceous limestone	28	27	23	9	12	14
Soft sandstone	15	12	11	8	11	11
Hard sandstone	0	5	6	4	5	3
Chert	13	4	18	8	7	7
Diabase	4	0	0	2	2	2
Granite	2	2	2	3	3	4
Quartz	0	3	2	2	1	0
Specific gravity, (% below 2.50)[1]	8.8	5.2	8.7	9.1	12.3	10.9
(b) PHYSICAL PROPERTIES						
Bulk dry specific gravity	2.62	2.63	2.62	2.61	2.60	2.59
Absorption (%)	1.8	1.8	1.9	2.1	2.2	2.4

[1] All material below 2.50 specific gravity was soft granular sandstone or chert, flint and chalcedony.

Appendix E

RESIDUAL STRESSES AND STATIC YIELD POINT OF STRUCTURAL STEEL FOR TEST BRIDGES*

Residual Stresses

Residual stresses were determined for 16 wide-flange beams by the method of sectioning. Measurements were made on a 10-in. gage length at locations shown in Figure 1-E on six 9-ft beams and ten 2-ft beams. The 9-ft beams had a 2-ft section cut out of the center which was then cut into ½- by 11-in. strips. Readings were taken before cutting, after the 2-ft section was cut out, and again after the section was cut into strips. The 2-ft beams were sectioned into ½- by 11-in. strips after the initial readings were taken; a second set of readings was taken on the strips.

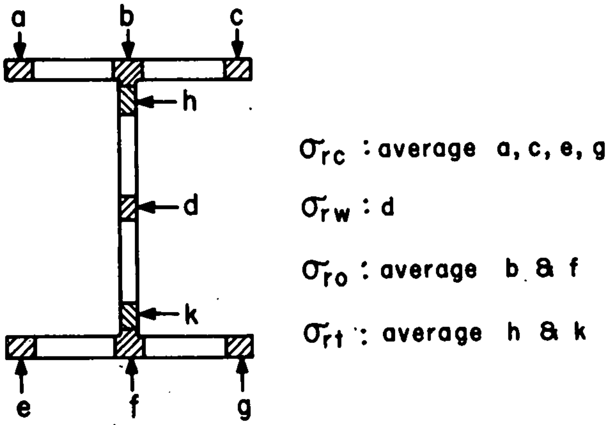


Figure 1–E. Location of residual stress measurements.

Table 1-E contains a listing of all specimens by heat and shape and the measured residual stresses. The residual stresses for both the 9-ft and the 2-ft beams are the differences between measurements made before and after each set of beams was cut into strips. The meaning of symbols σ_{rc}, σ_{ro}, σ_{rw} and σ_{rt} is explained in Figure 1-E.

The residual stresses obtained from the 9-ft 18WF96 beams may be compared to those reported by Beedle and Tall.** The ratio of the depth of section to its flange width, d/b, for the 18WF96 beam is 1.55, making it a borderline case in terms of the classification suggested by Beedle and Tall. Table 2-E gives the suggested

* The material included in this Appendix may be found also in a discussion by Fisher and Viest published in *Proc. ASCE*, V. 87: ST2, pp. 53-57 (Feb. 1961).

** Beedle, Lynn S., and Tall, Lambert, "Basic Column Strength." *Proc. ASCE*, V. 86: ST7, pp. 139–173 (July 1960).

TABLE 1–E

RESIDUAL STRESSES[1, 2]

Test No.	Heat	Shape	Obtained from 9-Ft Beams				Obtained from 2-Ft Beams			
			σ_{rc}	σ_{ro}	σ_{rw}	σ_{rt}	σ_{rc}	σ_{ro}	σ_{rw}	σ_{rt}
9A1	36	18WF96	− 6.3	−9.5	15.8	8.5	− 1.7	− 7.6	0.9	5.8
9A2	36	18WF96	− 9.5	−6.4	18.4	12.4	− 4.4	− 4.1	1.5	9.5
9A3	36	18WF96	− 9.1	0.2	2.2	6.8	− 7.9	0.7	− 1.5	5.5
9B1	36	18WF96	−11.1	−2.9	6.8	10.7	− 7.3	− 1.4	0.8	9.2
9B2	36	18WF96	−13.5	−7.9	19.0	9.6	− 8.7	− 5.4	4.1	6.9
9B3	36	18WF96	−13.3	−7.0	19.5	13.5	− 7.4	− 5.2	2.4	8.5
1A1	74	18WF55					− 5.4	16.9	−16.1	− 7.3
1A2	74	18WF55					− 6.8	16.5	−17.2	− 5.5
1B1	90	18WF50					− 9.8	11.9	−13.6	− 6.8
2B1	90	18WF50					− 7.2	17.8	−10.8	−14.1
3A1	93	21WF62					− 7.6	13.7	− 7.3	− 9.5
3A2	83	21WF62					−11.4	17.5	− 8.9	− 3.1
3A3	83	21WF62					− 8.1	15.3	− 8.3	− 5.5
3B3	42	18WF60					− 9.4	14.7	−13.0	− 9.4
4A2	42	18WF60					− 2.4	7.2	− 9.2	− 6.4
4B1	86	18WF60					− 1.7	0.9	−15.4	− 0.7

[1] Summary of Data System 2417.
[2] All stresses in ksi.

maximum, minimum and average values of the residual stresses as well as those found in the 18WF96 shapes. The average residual stress at the center of the flange (σ_{ro}) and at the center of the web (σ_{rw}) fall outside, but the residual stress at the flange edge (σ_{rc}) falls inside the limits suggested by Beedle and Tall.

An anaylsis of variance was made for the residual stress in the flange edge (σ_{rc}) of the 9-ft beams to determine the variation between specimens within a beam and between beams within a heat. This analysis showed that the variation in residual stress from beams of the same heat was significant, at the 1 percent level, when compared to the variation within a beam.

An analysis of variance was made also to determine the variation within a beam, between beams within a heat, and between heats (the comparison between heats is confounded by beam size) for the residual stress at the flange tips (σ_{rc}) of the 2-ft beams. The variation between beams and between heats was found significant, at the 1 percent level, compared to the variation within a beam.

The results obtained from the 2-ft specimens are smaller than those in longer beams. This is evident from examining the data for heat 36 in Table 1-E. The ratios of mean residual stresses observed on 9-ft beams to those on 2-ft beams are 1.9, 1.5, 5.8 and 1.4 for σ_{rc}, σ_{ro}, σ_{rw} and σ_{rt}, respectively.

Beedle and Tall reported that ". . . cooling residual stresses are constant along the member except for a distance approximately equal to the

TABLE 2–E

RESIDUAL STRESSES IN WIDE-FLANGE SHAPES DUE TO COOLING

Item	Flange Edge, σ_{rc}			Flange Center, σ_{ro}			Web Center, σ_{rw}		
	Max	Avg	Min	Max	Avg	Min	Max	Avg	Min
Authors' columns $d/b \leqq 1.5$	−7.7	−12.8	−18.7	+16.5	+ 4.7	−4.1	+18.2	+ 8.0	−15.5
18WF96 $d/b = 1.55$	−6.3	−10.5	−13.5	+ 0.2	− 5.6	−9.5	+19.5	+13.6	+ 2.2
Authors' beams $d/b > 1.5$	−4.1	− 7.5	−10.8	+24.2	+15.1	+8.3	− 8.8	−21.8	−41.0

[1] All stresses in ksi.

TABLE 3–E

YIELD POINT OF COUPONS FROM WIDE-FLANGE BEAMS AND COVER PLATES [1]

Heat	Beam Size	Number of Beams or Plates	Static Yield Point			Drop-of-Beam Yield Point			Mill Test
			No. Spec.	σ_{ys} (ksi)	s [2] (ksi)	No. Spec.	σ_y (ksi)	s [2] (ksi)	σ_y (ksi)
(a) FLANGE SPECIMENS									
90	18WF50	3	6	35.9	2.00	6	38.8	2.24	—
74	18WF55	3	6	31.5	1.22	6	34.4	1.15	—
83	21WF62	2	4	31.9	0.92	4	34.6	1.43	—
42	18WF60	5	10	32.8	1.70	10	35.5	1.79	—
36	18WF96	6	24	29.9	1.04	24	32.5	1.21	—
(b) WEB SPECIMENS									
90	18WF50	3	3	36.8	4.17	3	39.8	4.22	38.2
74	18WF55	3	3	36.6	1.30	3	39.5	1.22	40.3
83	21WF62	2	2	37.5	0.42	2	40.6	0.35	41.3
42	18WF60	5	5	36.9	1.98	5	39.6	1.77	39.9
36	18WF96	6	10	33.2	1.27	10	36.1	1.25	37.9
(c) PLATE									
85	7/16 x 6	12	24	34.8	1.02	24	37.5	1.04	36.1

[1] Summary of Data Systems 2410 and 2411.
[2] Standard deviation.

larger cross-sectional dimension at the ends." They also reported data on the effect of beam length on the magnitude of ratios σ_{rc} and σ_{ro}. Their data are in general agreement with the values reported herein when compared on the basis of equal values of $1/d$ (1 = beam length, d = the larger cross-sectional dimension of the shape). This suggests that the residual stresses σ_{rc} and σ_{ro} in long beams 18 in. deep may be estimated from the values obtained from short beams by multiplication with the ratios obtained for heat 36.

Static Yield Point

The results of 98 static tension tests concerning the yield stress level are given in Table 3-E. The tests were carried out in a 100,000-lb capacity screw-type mechanical testing machine. The data covered six different heats, five sizes of rolled wide-flange beams and one thickness of plate material.

The order of testing the coupons was randomized with respect to heat and thickness. The load was applied at a rate of 140 μ in./in. sec up to the yielding of the steel. The yield point was determined by the drop-of-beam of the testing machine. This same rate of straining was continued in the plastic region up to the point of strain hardening; however, in the plastic region the testing machine was stopped three times and a few minutes were allowed to elapse to enable the load to decrease to the minimum value and thus to determine the value of the static yield point. (The mean of the three minimum values was taken as the static yield point.) Deformations were observed on each specimen with an 8-in. extensometer up to strain hardening to insure that the static level was approached through increasing rather than decreasing strain.

The large number of corresponding data on the static yield point and the drop-of-beam yield point made possible a statistical analysis of the effect of heat, thickness and yield stress level on the ratio of the dynamic yield stress to the static yield stress for the strain rate of 140 μ in./in. sec. Tests at the 5 percent level of significance indicated no effect of heat, thickness, or yield stress level on the ratio of dynamic yield stress to the static yield stress. Hence, the mean value of the ratio, 1.08, represents all tests reported herein regardless of the source.

The mean ratio of dynamic to static yield stress is compared in Figure 2-E with the limits suggested by Beedle and Tall. The spread of the data reported herein is indicated by the 95 percent confidence limits (95 percent confidence limits correspond to $\pm$ 2s around the mean). The mean falls within the suggested limits but is located closer to the lower than to the upper limit.

The tension tests of structural steel from test bridges reported in Section 7.4 were made at the speed of 0.104 in. per min up to yielding corresponding to the strain rate of about 140 μin./in. sec. Thus, the static yield point for any wide-flange beam or coverplate is equal to 92.5 percent of the yield point given in Section 7.4. The dynamic yield point corresponding to any given strain rate may be estimated from the static yield point with the aid of Figure 2-E.

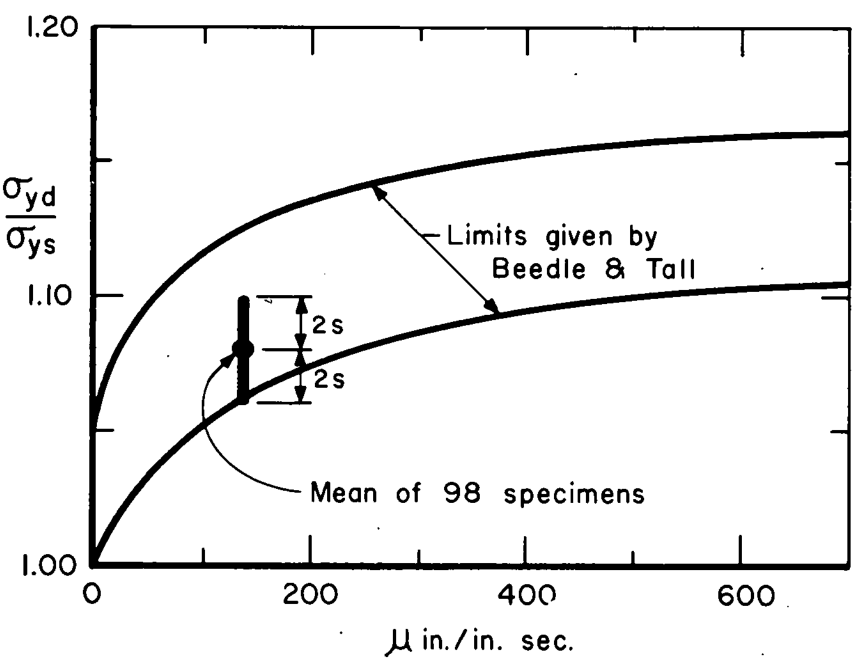


Figure 2–E. Influence of strain rate on yield stress level.

Appendix F

REGIONAL ADVISORY COMMITTEES

These committees were appointed by the Highway Research Board to maintain liaison between the state highway departments and the research project, through the National Advisory Committee. Three members of each Regional Committee were appointed to the National Advisory Committee.

Region 1

F. M. Auer, Planning and Economics Engineer, New Hampshire Department of Public Works and Highways

E. B. Bly, Engineering Assistant to Commissioner, Vermont Department of Highways

T. V. Bohner, Special Assistant, Engineering Department, D. C. Department of Highways and Traffic

W. M. Creamer, Chief, Highway Staff Services, Connecticut State Highway Department

F. W. Hauck, Supervising Civil Engineer (Road Designing), Rhode Island Department of Public Works

C. D. Jensen, Director of Research and Testing, Pennsylvania Department of Highways

G. W. McAlpin, Assistant Deputy Chief Engineer (Research), New York State Department of Public Works

J. F. McGovern, Structures Maintenance Engineer, Massachusetts Department of Public Works

L. W. Novinger, Contract and Design Engineer, Delaware State Highway Department

V. A. Savage, Engineer of Primary Highways, Maine State Highway Commission

W. Van Breemen, Research Engineer, New Jersey State Highway Department

The following were members of the Region 1 Advisory Committee during the years indicated:

H. F. Clemmer, *formerly Chairman;* Consultant, D. C. Department of Highways and Traffic (1956-1960)

R. A. Farley, formerly Deputy Secretary, Engineering, Pennsylvania Department of Highways (1956-1958)

W. C. Hopkins, Deputy Chief Engineer, Maryland State Roads Commission (1956-1961)

F. S. Poorman, Deputy Secretary, Engineering, Pennsylvania Department of Highways (1959)

L. K. Murphy, formerly Construction Engineer, Primary Highways, Maine State Highway Commission (1955-1959)

Region 2

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

W. F. Abercrombie, Engineer of Materials and Tests, Georgia State Highway Department

T. L. Bransford, Engineer of Research and In-Service Training, Florida State Road Department

L. D. Hicks, Chief Soils Engineer, North Carolina State Highway and Public Works Commission

G. W. McAlpin, Director, Program Office, and Assistant Chief Engineer, West Virginia State Road Commission

J. D. McMahan, Construction Engineer, South Carolina State Highway Department

A. O. Neiser, Assistant State Highway Engineer, Kentucky Department of Highways

T. W. Parish, Assistant Chief Engineer (Construction), Louisiana Department of Highways

R. S. Patton, Engineer of Surveys and Designs, Tennessee Department of Highways and Public Works

Angel (2) Silva, Director, Puerto Rico Department of Public Works

H. O. Thompson, Testing Engineer, Mississippi State Highway Department

J. F. Tribble, Materials and Research Engineer, Alabama State Highway Department

E. L. Wales, Engineer of Materials and Tests, Arkansas State Highway Commission

The following was a member of the Region 2 Advisory Committee during the years indicated:

J. L. Land, formerly Chief Engineer, Bureau of Materials and Tests, Alabama State Highway Department (1956)

Region 3

W. E. Chastain, Sr., *Chairman*, Engineer of Physical Research, Illinois Division of Highways

J. G. Butter, Consultant, Iowa State Highway Commission

E. A. Finney, Director, Research Laboratory, Michigan State Highway Department

R. A. Helmer, Research Engineer, Oklahoma State Highway Department

J. W. Hossack, State Engineer, Nebraska Department of Roads

C. P. Jorgensen, Manager, Research and Planning, South Dakota State Highway Commission

H. E. Marshall, Research Engineer, Ohio Department of Highways

R. L. Peyton, Assistant State Highway Engineer, State Highway Commission of Kansas

J. S. Piltz, Engineer of Design, Wisconsin State Highway Commission

C. K. Preus, Materials and Research Engineer, Minnesota Department of Highways

F. V. Reagel, Engineer of Special Assignments, Missouri State Highway Commission

W. T. Spencer, Soils Engineer, Indiana State Highway Department

W. A. Wise, Director, Field Division, North Dakota State Highway Department

The following were members of the Region 3 Advisory Committee during the years indicated:

L. N. Ress, formerly State Engineer, Nebraska Department of Roads (1956-1958)

H. G. Schlitt, formerly Deputy State Engineer, Nebraska Department of Roads (1959)

C. W. Allen, formerly Research Engineer, Ohio Department of Highways (1956-1958)

J. H. Swanberg, Chief Engineer, Minnesota Department of Highways (1956-1958)

Region 4

R. E. Livingston, *Chairman*, Planning and Research Engineer, Colorado Department of Highways

J. R. Bromley, Superintendent and Chief Engineer, Wyoming State Highway Department

L. F. Erickson, Assistant Construction Engineer, Idaho Department of Highways

L. B. Fox, Construction Engineer, Montana State Highway Commission

T. S. Huff, Chief Engineer of Highway Design, Texas State Highway Department

F. N. Hveem, Materials and Research Engineer, California Division of Highways

C. W. Johnson, Materials and Testing Engineer, New Mexico State Highway Commission

D. F. Larsen, Chief Materials Engineer, Utah State Road Commission

C. E. Minor, Materials and Research Engineer, Washington Department of Highways

W. G. O'Harra, Materials Engineer, Arizona Highway Department

W. M. Wachter, Highway Engineer, Hawaii Division of Highways

W. O. Wright, State Highway Engineer, Nevada Department of Highways

The following were members of the Region 4 Advisory Committee during the years indicated:

W. T. Holcomb, formerly Assistant State Highway Engineer, Nevada Department of Highways (1956-1959)

I. B. Miller, Operations Engineer, New Mexico State Highway Commission (1956-1958)

B. E. Nutter, formerly Territorial Highway Engineer, Hawaii Territorial Highway Department (1956-1958)

S. B. Sanders, formerly District Engineer, Montana State Highway Commission (1956-1958)

W. C. Williams, State Highway Engineer, Oregon State Highway Commission (1956-1961) (deceased)

ADVISORY PANELS

Advisory Panel on Materials and Construction

This panel was appointed by the Highway Research Board to advise on matters related to paving materials and construction techniques. It examined the plans and specifications with a view to reducing the cost of construction of the test facility.

R. L. Peyton, *Chairman,* Assistant State Highway Engineer, Kansas State Highway Commission

H. W. Allen, Chief, Physical Research Branch, Bureau of Public Roads

C. L. Gleason, Construction Engineer, Iowa State Highway Commission

F. N. Hveem, Materials and Research Engineer, California Division of Highways

J. D. Lindsay, Engineer of Materials, Illinois Division of Highways

G. W. McAlpin, Assistant Deputy Chief Engineer (Research), New York State Department of Public Works

F. V. Reagel, Engineer of Special Assignments, Missouri State Highway Department

W. T. Spencer, Soils Engineer, Indiana State Highway Department

R. H. Tittle, Assistant Chief Highway Engineer, Illinois Division of Highways

The following were members of this panel during the years indicated:

A. A. Anderson, Chief Highway Consultant, Portland Cement Association (1956-1960)

Hugh Barnes, Assistant Vice-President, Portland Cement Association (1960—Resigned March 1961)

J. O. Goshorn, District Engineer, The Asphalt Institute (1956—Resigned March 1961)

Advisory Panel on Soils

This panel was appointed by the Highway Research Board to advise on matters dealing with construction of the soil embankment.

Harold Allen, *Chairman,* Chief, Physical Research Division, Bureau of Public Roads

A. W. Johnson, Engineer of Soils and Foundations, Highway Research Board

N. E. Mason, Testing and Research Laboratories, Ohio Department of Highways

G. W. McAlpin, Assistant Deputy Chief Engineer (Research), New York State Department of Public Works

Chester McDowell, Supervising Soils Engineer, Texas Highway Department

The following was chairman of this panel during the years indicated:

Frank Olmstead, formerly Chief, Soils Branch, Bureau of Public Roads (1956-1958)

Special Publication Subcommittee for AASHO Road Test Report 2 on Materials and Construction

This subcommittee was appointed by the Highway Research Board to advise the staff in the preparation of AASHO Road Test Report 2, "Materials and Construction," and recommend approval of the report for publication.

E. H. Holmes, *Chairman,* Assistant Commissioner for Research, Bureau of Public Roads

W. F. Abercrombie, Engineer of Materials and Tests, Georgia State Highway Department

W. E. Chastain, Sr., Engineer of Physical Research, Illinois Division of Highways

F. N. Hveem, Materials and Research Engineer, California Division of Highways

G. W. McAlpin, Assistant Deputy Chief Engineer (Research), New York State Department of Public Works

The following were members of this subcommittee during the years indicated:

R. E. Jorgensen, formerly Engineering Counsel, National Highway Users Conference (Resigned May, 1961)

W. C. Williams, formerly State Highway Engineer, Oregon State Highway Commission (1956-1961)

PROJECT PERSONNEL

Project Staff and Engineers During Construction Phase

W. B. McKendrick, Jr., Project Director
W. N. Carey, Jr., Chief Engineer for Research
Peter Talovich, Business Administrator
A. C. Benkelman, Flexible Pavement Research Engineer
Howard H. Boswell, Maintenance Engineer
Henry C. Huckins, Supervisor, Instrument Laboratory
Paul E. Irick, Chief, Data Processing and Analysis
R. C. Hain, Assistant Chief Data Processing and Analysis
Rex C. Leathers, Engineer of Special Assignments
W. J. Schmidt, Chief, Public Information
Frank H. Scrivner, Rigid Pavement Research Engineer
W. R. Hudson, Assistant Rigid Pavement Research Engineer
James F. Shook, Materials Engineer
H. Y. Fang, Field Engineer
J. F. Reynolds, Field Engineer
H. M. Schmitt, Field Engineer
R. K. Williamson, Field Engineer
Arthur C. Tosetti, Road Engineer
Donald R. Schwartz, Assistant Road Engineer
Ivan M. Viest, Bridge Research Engineer
J. W. Fisher, Assistant Bridge Research Engineer

The following were members of the project staff during the years indicated:

L. Q. Mettes, Materials Engineer (1956)
Moreland Herrin, Assistant Materials Engineer (1958)
O. B. Andersland, Assistant Materials Engineer (1956-1957)

Illinois Division of Highways

Permanent Task Force During Construction Phase

The Illinois Task Force was responsible for the preliminary engineering and for the plans, specifications, and construction of the test facility.

W. E. Chastain, Sr., Engineer of Physical Research, In Charge
Arthur C. Tosetti, Road Engineer
Donald R. Schwartz, Assistant Road Engineer
T. E. Hagerman, Resident Engineer, Bridges and Rigid Pavement
Jack Licata, Resident Engineer, Flexible Pavement
L. E. Dixon, Office Engineer
D. W. Ballinger, Field Engineer
H. R. Hubbell, Field Engineer
R. J. Little, Field Engineer
R. D. Stewart, Field Engineer
A. J. Wright, Field Engineering Technician

The following were task force members during the years indicated:

A. H. Herberts (1955-1956)
R. H. Long (1955-1956)

Staff Consultants and Engineer Observers

The following resident staff consultants and engineer observers were on the project during the construction phase.

B. E. Colley, Senior Development Engineer, Portland Cement Association

F. N. Finn, Special Projects Engineer, The Asphalt Institute

S. M. King, Special Projects Engineer, American Trucking Associations, Incorporated

R. I. Kingham, Resident Engineer to AASHO Road Test, Canadian Good Roads Association

W. E. Teske, Highway Engineer, Paving Bureau, Portland Cement Association

G. A. Wrong, Principal Soils Engineer, Province of Ontario, Canada

The following were staff consultants or engineer observers during the construction phase:

R. A. Lill, Highway Engineer, American Trucking Associations, Inc. (1955-1957)

G. D. Campbell, Director of Technical Services, Canadian Good Roads Association (1956-1957)

The following non-resident engineer observers periodically visited the project:

E. R. Feldman, Highway Engineer, Division of Competitive Transportation, Association of American Railroads

E. J. Ruble, Research Structural Engineer, Association of American Railroads

Rockwell Smith, Soils Engineer, Association of American Railroads

Loaned Personnel

The following personnel of various state highway departments were loaned to the Project during the construction seasons to assist the Illinois Task Force in the supervision of the construction of the test facilities. These men served in the capacity of loop resident engineers for embankment construction, paving engineers, inspectors, and survey party chiefs.

Richard Bacon, Michigan (1958)
J. A. Beanblossom, Illinois (1956 and 1958)
E. R. Becker, Illinois (1956)
R. R. Biege, Jr., Kansas (1958)
J. C. Delbridge, Iowa (1956)
P. H. Ferleman, Iowa (1957)
W. D. Ferree, Illinois (1956, 1957 and 1958)
N. E. Grigsby, Ohio (1958)
R. D. Hamilton, Iowa (1958)
R. E. Hay, Kansas (1956)
E. J. Heinen, Minnesota (1958)
J. R. Jensen, Nebraska (1957 and 1958)
D. A. Juza, Wisconsin (1956)
T. J. Kinsman, Missouri (1958)
T. F. Krarup, Illinois (1958)
R. H. Mitchell, Illinois (1956 and 1958)
W. D. Payne, Missouri (1956)
W. W. Rickerson, Missouri (1957)
C. L. Shively, Illinois (1956)
R. M. Short, Ohio (1956)
T. W. Sonnenberg, Wisconsin (1958)
T. R. Terry, Oklahoma (1956)

Junior engineers employed by the Bureau of Public Roads were assigned to the project for approximately six months each as a part of their training program. The following include those assigned during the construction phase (August 1956 through November 1958) who assisted the Illinois Task Force in the preparation of the plans and the supervision and inspection of the construction and assisted the Project Materials Laboratory Branch in the on-site testing for materials and construction control.

J. C. Becker
G. R. Brooks
T. J. Chipera
J. P. Clark
A. R. Cowan
Daniel Dake
Rothe Davis
T. E. Difloe
W. S. Dunbar
W. A. Eager
G. D. Gibson
R. D. Gingrich
G. L. Green
R. H. Hogrefe
G. K. Hossner
Donald Jacobsen
R. H. Jones
R. C. Kay
D. B. Lewis
C. W. Manaton
Herbert Marshall
V. W. Segelke
H. M. Schmitt
C. H. Snow

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.