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Bulletin 290

Developments in Salvaging Old Pavements by Resurfacing

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1961

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N.R.C. HIGHWAY RESEARCH BOARD

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Contents

Summarized Committee Report 1948-1960: Salvaging Old Pavements by Resurfacing

VERNON G. GOULD, Assistant Construction Engineer, Iowa State Highway Commission, Ames

Recognizing that the salvage of old pavements was a national problem, the Highway Research Board, through its Department of Maintenance, organized a committee in 1948 to assemble and disseminate information relative to prevailing practices to those agencies concerned with such work.

The first phase of committee study was to assemble information regarding practices in design and construction currently in vogue from 1948 through 1951, and was terminated with the publication of HRB Bulletin 47.

The scope of committee activity was then expanded to keep abreast of new developments in practices, methods and technique in the salvage of both rigid- and flexible-type pavements. To this end, the committee has encouraged the presentation of papers covering experimental work and new methods and means to improve the performance of salvaged pavements, and has published replies to questionnaires dealing with specific aspects of such work.

This paper covers briefly what are considered to be the most significant studies and experimental work, completed or in progress by research agencies of highway departments and educational institutions, as reported to the Highway Research Board.

BY 1947 it was recognized that the salvage of old pavements was a national problem, d in January 1948 the Highway Research Board took action to assemble and dissemin- information relative to prevailing practices to those agencies and individuals con- rned with such work.

A committee was organized under the Highway Research Board, Department of intenance, to "provide information that may be used by cities, counties, states or ividuals in the salvage of old pavements by application of a resurfacing treatment." e committee was designated "Project Committee No. 4, Salvaging Old Pavements by surfacing."

During the period from January 1948 to January 1952, the committee functioned un- the chairmanship of Robert H. Tittle, Engineer of Construction, Illinois Division Highways. Committee membership included individuals from the Portland Cement sociation, the Asphalt Institute, the petroleum industries, educational institutions highway departments.

The author of this paper was not a member of the committee during this period and ailed activities are not available to him. However, it is known that a survey was ducted to assemble information as to current practices in pavement salvage during t period.

Activities of the original committee were concluded with the publication of HRB letin 47 and the committee was inactivated in January 1952. Bulletin 47 contained information assembled by the committee relative to (1) economics of resurfacing, (2) study of existing pavements both portland cement and bituminous types, (3) surveys and plans, (4) preparation of the old pavement for resurfacing, (5) widening pavement and (6) resurfacing with both rigid and flexible-type overlays.

The committee was reactivated in January 1953 to study the performance of pavements after salvaging. Reorganization of the committee was essential due to the resignation of Mr. Tittle as chairman and a few changes in membership. The author of this paper was named to succeed Mr. Tittle, and the purpose of the committee and its scope of activities were revised to read:

Purpose. "... To study the performance of salvaged pavements to provide more comprehensive and instructive information as to methods of salvage, design and construction."

Scope. "1. Study of new developments in pavement salvage.
2. Evaluation studies:
   a. Pavement salvage by resurfacing and for which evaluation data ar available from a survey.
   b. Underseals or other means of restoration of subgrade support for rigid-type pavements in connection with resurfacing.
3. Study of test methods and technique to determine additional strength provided by various thicknesses of resurfacing."

It may be well to state specifically relative to committee activity—its function is n to conduct research or experimental work but to assemble and analyze information a data from any agency performing such work, and disseminate worthwhile findings by committee prepared or sponsored publications.

As a means of gathering information for analysis and/or dissemination with minimum delay, the committee seeks out and encourages prospective authors concerned some phase of research or experimentation within the scope of committee activity to present papers covering such work under committee sponsorship. So far as the committee may have knowledge, authors are selected to assure nationwide coverage and will be from highway departments, educational institutions and engineering, industri and governmental agencies performing or concerned with pavement salvage.

For convenience in this paper, the writer will arbitrarily classify the subject mat of committee sponsored papers, or replies to questionnaires mentioned herein, unde one of the three items set forth above as scope of committee activity.

DISCUSSION

A summary of pertinent information given in committee prepared or sponsored publications, and classified herein as applicable to one of the items previously mentioned as scope of committee activity, follows.

A. New Developments

1. Studies of Reflection Cracking in Flexible-Type Overlays

   a. Personnel of the Joint Highway Research Project, Massachusetts Institute o Technology and Massachusetts Department of Public Works. This agency h made an intensive study of means and/or methods of preventing or controlling reflection cracking in flexible-type overlays on rigid-type pavement. The titles of committee sponsored papers presented by it are:

   (1) "Control of Reflection Cracking in Bituminous Resurfacing Over Old Cement Concrete Pavements." The paper described several method of experimental joint treatment and reinforcing for the years 1952 an 1953, and was reported in 1954. The devices used included plugging joints with cement grout or a stabilized soil mix, breaking bond between the bituminous concrete and cement concrete at joints, placing

metal plates on concrete, and reinforcing bituminous concrete with various types of metal either on the old pavement or between binder and surface courses of the overlay. Partial conclusions from this study are stated to be:

(a) Reflection cracking results from differential vertical or horizontal movement in the underlying concrete. Vertical movement can usually be controlled by adequate subgrade support, but horizontal movements due to temperature changes are inevitable and are a primary cause of reflection cracking, particularly over transverse joints.

(b) Tests described do not establish any sure method of eliminating reflection cracking though there are indications that its extent can be reduced.

(2) "Current Practices and Research on Controlling Reflection Cracking." The paper presented in 1955 reviews methods proposed for controlling reflection cracking and classifies them as (a) prevention and (b) sealing. The methods described and commented on include (a) elimination of joint movement by filling the joints with incompressible material, (b) breaking the pavement into small pieces before resurfacing, (c) increasing the thickness of overlay, (d) use of additive to the bituminous mix to increase ductility, (e) use of mesh reinforcing in overlay over existing joints, (f) use of light metal plates over the joints and under the overlay, (g) use of a granular intermediate course, (h) sawing grooves in the overlay over the joints in the existing pavement, (i) use of burlap layers in the bituminous mix over the existing joints, and (j) sealing methods and materials as maintenance measures for reflection cracks. The paper concludes that no wholly satisfactory method or technique for crack control has been developed. Recommendations relative to further studies are made.

(3) "Progress of Reflection Cracking in Bituminous Concrete Resurfacing." The paper covers a study of reflection cracking in overlays on rigid-type pavements in Massachusetts with standard Class I state specification bituminous mixes and also mixes modified by various types of rubber additives. The study was started in 1952 and reported in 1956. Surveys have been conducted on twenty-five 1,000-ft sections of the pavement to evaluate their relative performance. From the study it is concluded that about 90 percent of the possible potential reflection cracking will take place by the time five or six years have elapsed regardless of the addition of additives.

b. Personnel of the California Division of Highways. Paper by Ernest Zube titled "Wire Mesh Reinforcement in Bituminous Resurfacing," describes an experimental project in which the California Division of Highways in 1954 placed several test sections for the purpose of comparing the relative merits of various types of wire mesh as a preventative for reflection cracking of bituminous overlays. A cost analysis is made and a summary and partial conclusions are given. The cost analysis, in part, states that for continuous reinforcement the cost is equal to 1½ in. of bituminous resurfacing. Limited service prevents definite opinions regarding the effectiveness of the various types of wire mesh used in preventing or retarding reflection cracking. It is stated, however, that there is definite evidence that the wire reinforcement has prevented the formation of longitudinal cracks.

c. Personnel of the Iowa State Highway Commission. Paper by Stephen Roberts titled "Cracks in Asphalt Resurfacing Affected by Cracks in Rigid Bases," describes several test sections established in 1948 and 1949. The methods used include cleaning cracks and filling with binder mix and/or binder mortar containing finer aggregate only, the use of SC-5 cutback asphalt as a first course of the overlay with an additional 3 in. of asphalt concrete placed in two courses using asphalt cement, and the use of 100+ penetration asphalt cement for the surface course versus 70-85 penetration grade for other sections. Based on observations made during the investigation, the summary states in part:

(1) Where base cracks were cleaned and refilled before resurfacing, results indicate that an asphalt mortar is superior as a crack filler versus the regular binder coarse mix; where cracks were not cleaned or refilled an additional 1½ in. of overlay did not significantly reduce reflection cracking.

(2) Some indications were noted that reflection cracking was affected by the penetration grade of asphalt used in the overlay.

d. Personnel Anonymous. HRB Circular 413, "Summary of Replies to Committed Questionnaire." The circular (in part) summarizes replies to a question asked in a questionnaire to all highway departments in 1958 relative to methods used or under test as a preventative for reflection crackin Replies indicate that numerous methods have been or are being used, but perhaps the use of wire mesh and/or granular intermediate courses may b considered as most extensively used.

2. Studies and Experimental Work With Rigid-Type Overlays

a. Personnel of the Portland Cement Association. Paper by Earl J. Felt titled "Resurfacing and Patching Concrete Pavements With Bonded Concrete," covers laboratory bond tests, experimental field projects, a survey of projects in use and recommended practices. A description of five experimental pilot study projects build from 1951 to 1954 is given—overlays varying in thickness from ¼ to 6 in. Field projects in Michigan, Nebraska, New York, Ohio, Pennsylvania, Rhode Island and Wisconsin are described. The paper sets forth the following pertinent findings: Laboratory data and field work indicate that bond strengths as determined by shear test may frequently be 400 lb or more, but that strengths of 200 lb per sq in. or even less may be adequate. The two ma factors governing bonding are (1) the strength and integrity of the old bas concrete and (2) the cleanliness of the old surface. Best bond was obtain when the base concrete was dry and grouted. Good compaction of the fre concrete is also required for a strong bond. The performance of experimental projects and other projects in service shows that properly bonded surfaces will withstand extreme climate conditions and heavy traffic. Th bonded surfaces giving satisfactory service date back to 1913.

b. Jointly by Personnel of the Iowa State Highway Commission and the Portl Cement Association. Paper by J.W. Johnson and W.G. Bester titled "Widening and Resurfacing Highways With Concrete," describes resurfacing rigid-type pavements in Iowa with rigid-type overlays and includes a number of projects over a number of years. The work is arbitr ly divided into five-year periods from 1930 through 1950 for the purpose design comparison and in order not to go into unnecessary detail for each project. Performance data are given for each five-year period.

The paper also covers a research project in 1954 in which the old pavement was widened 2 ft on each side with 10 in. of concrete. Resurfa was 1, 2 and 3 in. in nominal thickness, bonded to a structurally sound c crete pavement. Reinforcement was used in some sections and not in ot sections. In general, the condition of the pavement at time of reporting was good.

The paper concludes in part: It does not appear that bonding or no bo of the overlay to the old pavement was of importance in performance for thicknesses of resurfacing from 4 to 6 in. This paper is considered applicable also to "B—Evaluation Studies."

c. Personnel of the Washington Highway Commission. Paper titled "Preparing Old Pavements for Resurfacing With 50-Ton Compactor," by John L. Stackhouse, describes in detail the use of a 50-ton pneumatic-tired roller in seating an old portland cement pavement preparatory to widening and resurfacing in 1956. After using the roller a crushed stone base course of 3-in. minimum thickness was placed over the widened pavement followed by a top course of like material 2 in. in thickness. A 3-in. flexible-type overlay was then placed over the granular intermediate course.

The paper states that the advantage of the heavy compactor appears to be that by using a weight heavier than legal loads, the weak areas are disclosed and corrections for unstable material in such areas can be made. The paper further states that no reflection cracking has been noted after 18 months, but that main credit for elimination of reflection cracking probably should be given to the granular intermediate course rather than the heavy compactor.

B. Evaluation Studies

1. Pavement Salvage by Resurfacing

a. Personnel of the Missouri Highway Department. Paper titled "An Investigation of Concrete Resurfacing of a Concrete Pavement in Various Stages of Deterioration," by Messrs, Gotham and Lord, deals with two experimental concrete resurfacing projects on US 40 at specified locations in Missouri—one in 1932 and the other in 1936. The 1932 project included two ¼-mi long sections, one with a 4-in. and the other a 6-in. overlay. The 1936 project was 11¾ mi in length and included overlays 4, 5 and 6 in. in thickness. The report covers a study of the effect of thickness on concrete overlay performance, effects of cracks in the old pavement, effects of expansion joints, etc.

In summary, the paper notes some 27 specific findings but only a few considered of particular significance are mentioned, as follows:

(1) A thickness of 6 in. was more durable and theoretically more economical than 5 or 4 in.

(2) The 6-in. surface built in 1932 gave excellent service for 18½ yr and when again resurfaced in 1951 apparently could still have been used for many years without excessive maintenance.

(3) The 4- and 5-in. resurfaces built in 1936 on old pavement in relatively good and in intermediate condition showed considerable distress after 15 yr of service and these, especially the 4 in., would have required extensive maintenance for further service.

(4) ....Considering only those resurfacing slabs which lay on uncracked old pavement, a tendency was noted for such slabs to crack in lengths averaging 20 to 25 ft regardless of the thickness of the resurfacing.

(5) Surface deterioration developed to some degree in each of the various surfaces, but in general the percentage of area affected varied inversely with the thickness of resurface. This paper is considered applicable also to "A—New Developments."

b. Personnel of the Portland Cement Association. Paper Titled "Bonded Resurfacing and Repairs of Concrete Pavement," by William G. Westall, discusses bonded concrete construction and its various applications to a pavement, and describes projects which have been accomplished at air bases from 1955 to 1959; it also illustrates the essential steps in surface preparation and construction. The paper is in two parts—Part I covers thin bonded concrete and Part II deals with bonded concrete for patching and repairing concrete.

Part I describes rigid-type overlays on rigid-type pavements for five airports. The overlays varied in thickness from 1 in. to 4 in. and all were placed under contract conditions.

Part II describes patching and repair of one air force base project with bonded concrete and includes a variety of types of repairs. Conclusions given in the paper are briefly stated herein as follows:

(1) Resurfacing or patching of concrete pavements with bonded concrete ha been proved feasible in both laboratory investigations and field constru tion projects.

(2) When the overlay is to smooth a rough existing surface, a thin surfaci layer will perform as well as, and in many cases, better than a thicke slab, provided adequate bond is obtained between the old and new concrete.

(3) The satisfactory performance of resurfacing or patching with bonded concrete depends on the securing of adequate and uniform bond betwee the two surfaces.

2. Restoration of Subgrade Support for Rigid-Type Pavements in Connection Wit Resurfacing

a. Personnel Anonymous. HRB Circular 413, "Summary of Replies to Committee Questionnaire." The Circular (in part) summarizes replies to a nationwide questionnaire to highway departments, prepared and mailed jointly by the committee and the Highway Research Board in 1958, reques information as to the method of subgrade restoration for rigid-type pavements used preparatory to resurfacing. Replies were received from all b three highway departments. Although numerous methods were mentioned, replies indicated that perhaps bituminous underseals and mud jacking may be considered as most extensively used. To what extent methods used we considered successful is not stated in replies to the questionnaire.

C. Test Methods and Technique to Determine Required Thickness of Overlays

1. General

Information available to the committee indicated that the thickness of ove lays used, regardless of type, was determined by engineering judgment and observing the condition of the existing pavement. Considering that perhaps scientific methods were being used or developed to determine the thickness of overlays required as the tempo of pavement salvage work increased, the committee jointly with the Highway Research Board prepared and sent out tw questionnaires to determine definitely if scientific methods were being used tested.

a. Personnel Anonymous. HRB Circular 316, "Summary of Replies to Com mittee Questionnaire." The summary mentioned above refers to replies a questionnaire sent to all highway departments, territories and cities wi a population of 500,000 or more, in 1954 requesting information as to whether scientific methods were used in determining the thickness of ove lay required and, if so, what methods were used. From replies receive it was shown that no city contacted uses such methods. A few state high departments indicated the use of Hveem Stabilometer, CBR curves or th triaxial test for flexible-type pavements, and some other states stated that some scientific method was being considered or studied. It did not appear that any state was using a scientific method for determining the thickness of rigid-type overlay required. It was contemplated that the committee should maintain contact with highway departments using scien methods to ascertain whether such methods were considered suitable.

b. Personnel Anonymous. HRB Correlation Circular 413, "Summary of Re to Committee Questionnaire." The summary of replies mentioned in thi case refers to a follow-up questionnaire concerning evaluation methods and sent to all highway departments in 1958. Replies did not indicate any change in status regarding the use of scientific methods for determining the thickness of rigid-type overlays.

c. Personnel of North Carolina State Highway Commission. Paper titled "Flexible Pavement Deflection Study in North Carolina," by L.D. Hicks, covers a three-year deflection study of flexible pavements with Benkelman Deflection Gage from 1957 to 1959. Some of the deflection surveys were made before and some after the flexible-type overlays were placed. Pavement design data, subgrade classification and specifications for base and surface courses, together with volume and character of traffic, are also given. An analysis of deflection data is made and from this analysis and the pavement condition, conclusions are drawn and recommendations for improvement in pavement design and construction can be made. The author states in his opinion:

(1) A deflection survey is an excellent means of pavement evaluation.
(2) The determination of maximum deflection, a bituminous plant mix pavement, can sustain without excessive cracking is a complex problem that will require much investigation—in North Carolina the desirable limit has been tentatively set at 0.3 in.
(3) Excessive deflection of flexible-type pavements must be controlled in the preparation of the subgrade—test rolling is an excellent method for locating faulty subgrade.

This paper is considered applicable also to "B—Evaluation Studies."

CONCLUSIONS

Considering the three principal items under "scope of committee activity" discussed rein it may be concluded that:

1. It is probable that some new developments in pavement salvage which would be of neral interest have not come to the attention of committee members and have not en publicized.

2. Information available to the committee seems to indicate that evaluation studies flexible-type overlay performance are indeed limited.

3. Little, if any, progress is indicated by highway departments in the development scientific means to evaluate the additional strength provided by various thicknesses rigid-type overlays, but a few scientific methods being tested for evaluation of xible-type overlays show promise. A written discussion to follow this paper will line scientific methods developed by the Portland Cement Association and Corps Engineers.

4. Practical and effective methods and/or means to eliminate reflection cracking flexible-type overlays will apparently require further study even though quite exsive investigations have been reported.

5. Reflection cracking for rigid-type overlays, regardless of thickness, may not a problem since no studies of this nature have been reported to the committee. The itten discussion mentioned in Paragraph 3 above will mention effective methods to minate or reduce such cracking.

SUGGESTIONS FOR FUTURE COMMITTEE STUDY

1. Determine if reflection cracking of rigid-type overlays is a problem requiring ther study and, if so, what studies should be or are being made in connection with problem.

2. Keep abreast of new developments in design experimental work or construction nique for both flexible- and rigid-type overlays.

3. It is considered that reports of performance of pavements salvaged by resurng are of prime importance. It is of record that many variables have entered into

design for widening and resurfacing for both flexible- and rigid-type bases and overlays. Many miles of pavement have been salvaged and under traffic for some time. Performance reports are pertinent to committee activity and such reports are lacking for projects employing flexible-type overlays and the rigid, thin, bonded type.

4. Contact with states using or developing scientific methods for determination of overlay thickness for either the rigid- or flexible-type should be maintained.

GENERAL COMMENTS

In closing it may be well to advise that publications mentioned for one item under "scope of committee activity" often includes information pertinent to another item.

Titles and authors of committee prepared or sponsored publications for the period 1952-1960, including those mentioned herein, will be found in the Appendix. Several excellent papers not specifically mentioned in the body of this paper appear in the Appendix.

Appendix

HIGHWAY RESEARCH BOARD PUBLICATIONS (1952-1960) PREPARED OR SPONSORED BY HIGHWAY RESEARCH BOARD, MAINTENANCE DEPARTMENT, COMMITTEE ON SALVAGING OLD PAVEMENTS BY RESURFACING

<u>Highway Research Board Proceedings</u>:

1. "Control of Reflection Cracking in Bituminous Resurfacing Over Old Cement Concrete Pavements," by Alexander J. Bone, Lewis W. Crump and Vincent J. Roggeveen. Vol. 33, pp. 345-354, 1954.

Concerns experimental work conducted by Massachusetts Institute of Technology and Massachusetts Department of Public Works. Causes of reflection cracking and several experimental techniques for control are discussed.

2. "Cracks in Asphalt Resurfacing Affected by Cracks in Rigid Bases," by Stephen E. Roberts, Iowa State Highway Commission. Vol. 33, pp. 341-345, 1954.

Covers investigation of reflection cracks in asphaltic surfaces over old concrete pavements. Test sections were established to provide information concerning the growth of reflection cracks.

3. "Widening and Resurfacing Highways With Concrete," by James W. Johnson, Iowa State Highway Commission, and W.C. Bester, Portland Cement Association. Vol. 34, pp. 434-452, 1955.

Covers widening and resurfacing on 45 projects.

4. "Resurfacing and Patching Concrete Pavements With Bonded Concrete," by Earl J. Felt, Portland Cement Association. Vol. 35, pp. 444-469, 1956.

5. "Preparing Old Pavements for Resurfacing With 50-Ton Compactor," by J.L. Stackhouse, Washington State Highway Commission. Vol. 38, pp. 464-471, 1959.

6. "Investigation of Longitudinal Cracking Reflected Through Asphaltic Concrete Resurfacing," by John E. Boring and Bert Myers, Iowa State Highway Commission. Vol. 38, pp. 472-479, 1959.

<u>Medium of Publication Unknown</u>:

(Papers presented January 1960.)

1. "Flexible Pavement Deflection Study in North Carolina," by L.D. Hicks, State Highway Commission.

Covers three-year study to determine maximum safe deflection a flexible-type pavement can undergo in service as measured by Bendelman Deflection Gage.

2. "Bonded Resurfacing and Repairs of Concrete Pavement," by William G. Westall, 'ortland Cement Association.

A discussion of bonded concrete construction and its various applications to pave-
nents, in two parts. Part I covers thin bonded concrete overlays, and Part II deals
vith bonded concrete for patching and repairing concrete.

Iighway Research Board Bulletins:

1. "Salvaging Old Pavements by Resurfacing," by Robert H. Tittle, Engineer of 'onstruction, Illinois Division of Highways, and Chairman of Project Committee No. , Bulletin 47, 43 pp., 1952.

2. "Concrete Resurfacing of Concrete Pavement in Various Stages of Disintegration," y D.E. Gotham and G.W. Lord, Missouri State Highway Department. Bulletin 87, 9 pp., 1952.

Covers a study of the effect of thickness on concrete overlay performance, effect f cracks in the old pavement, effect of expansion joints, etc., on two experimental esurfacing projects in Missouri.

3. "Current Practices and Research on Controlling Reflection Cracking," by lexander J. Bone and Lewis W. Crump, Joint Highway Research Project, Massachu-etts Institute of Technology. Bulletin 123, pp. 33-39, 1956.

Reviews and discusses prevention and sealing of cracks as methods to control re-ection cracking.

4. "Condition Surveys of Bituminous Resurfacing Over Concrete Pavements," by ewis W. Crump and Alexander J. Bone, Joint Highway Research Project, Massachu-etts Institute of Technology. Bulletin 123, pp. 19-32, 1956.

Describes a technique for making and analyzing condition surveys of flexible over-ys on concrete pavements.

5. "Pavement Widening and Resurfacing in Idaho," by L.F. Erickson and Philip Marsh, Idaho Department of Highways. Bulletin 131, pp. 19-25, 1956.

Describes the manner and method in which structurally inadequate and narrow ghways were reconstructed to desired standards in Idaho.

6. "Highway Rehabilitation by Resurfacing," by K.B. Hirashima, Territorial ghway Department, Honolulu, Hawaii. Bulletin 131, pp. 26-30, 1956.

Covers rehabilitation of old pavements by resurfacing in the City and County of nolulu.

7. "Conditioning an Existing Concrete Pavement for Bituminous Resurfacing," by ed W. Kimble, Ohio Department of Highways. Bulletin 123, pp. 11-18, 1956.

Describes method of conditioning an unreinforced concrete pavement constructed in rtime for a flexible-type overlay.

8. "Progress of Reflection Cracking in Bituminous Concrete Resurfacings," by ncent J. Roggeveen and Egons Tons, Joint Highway Research Project, Massachusetts titute of Technology. Bulletin 131, pp. 31-46, 1956.

Covers condition surveys of 25 sections of pavement resurfaced during 1949-1952 Massachusetts.

9. "Rejuvenating Highway Pavement," by John L. Stackhouse, Washington State hway Commission. Bulletin 123, pp. 1-10, 1956.

Describes the resurfacing with a flexible-type overlay of a section of State Highway ear Tacoma, Washington.

10. "Wire Mesh Reinforcement in Bituminous Resurfacing," by Ernest Zube, Cali-nia Division of Highways. Bulletin 131, pp. 1-18, 1956.

Describes an experimental project in California covering eight test sections with ious types of wire mesh in a flexible-type overlay.

Highway Research Board Correlation Service Circulars:

1. "Salvaging Old Pavements by Resurfacing—Summary of Replies to Committee Questionnaire." Anonymous. Circular 316, 1956.
Summarizes replies to a committee questionnaire to highway departments and large cities relative to pavement salvage evaluation.

2. "Salvaging Old Pavements by Resurfacing—Summary of Replies to Committee Questionnaire." Anonymous. Circular 413, 1960.
Summarizes replies to questionnaire to highway departments relative to methods of subgrade restoration for rigid-type pavements, methods used as a preventative for reflection cracking, and methods used or under test for evaluation of pavements befor and after resurfacing.

Highway Research Board Bibliographies:

1. "Salvaging Old Pavements by Resurfacing, Annotated." Anonymous. Bibliography 21.
A single volume in two sections, each separately indexed, one section covering rigid-type overlays and the other overlays of the flexible type. References total 256 with annotations for each entry.

Discussion

WILLIAM G. WESTALL, Supervising Engineer, Airfield Program, Portland Cement Association.—The paper prepared by Mr. Gould provides a comprehensive summary of committee activities and the significant developments since 1947 in the field of pav ment resurfacing. This report will be of great value in establishing the current statu of pavement resurfacing and in pointing the direction the committee should take in future investigations.

The purpose of this discussion is to supplement statements made by the author in paragraphs three and 5 of his conclusions.

The reference to concrete overlays contained in paragraph 3 is quoted as follows:

"Little, if any, progress is indicated by highway departments in the development of scientific means to evaluate the additional strength provided by various thicknesse of rigid type overlays...."

The accuracy of this statement is in no way questioned. The available reports an records indicate that the design of concrete resurfacing for highway pavement has generally been on a trial-and-error basis with thickness determinations based on the experience and judgment of local paving engineers. The evaluation of such resurfaci has been dependent on its performance under traffic. This type of evaluation has se ous shortcomings. If the resurfaced pavement shows early distress, there is no way of determining the degree of design deficiency. On the other hand, if the resurfacing performs adequately over a long period of time, there is no existing method for chec ing the possibility of over-design. In either case, the engineer is placed in a quanda

The trial-and-error approach is understandable in view of the fact that well-estab lished design criteria have been lacking. Tests conducted in recent years, however, have provided data for the development of design equations that are more realistic a less conservative in comparison with the formulas previously used.

For many years concrete resurfacing courses were designed on the theory that th strength of the two slabs was equal to that of a single slab having a thickness equal to the square root of the sum of the squares of the pavement and resurfacing.

$$h = \sqrt{h_r^2 + h_e^2} \quad ($$

/here

h = thickness of required single slab, in inches;
h_r = thickness of resurfacing, in inches;
h_e = thickness of existing slab, in inches;

r

$$h_r = \sqrt{h^2 - h_e^2} \tag{2}$$

This method of analysis is theoretically correct only when the old pavement and the esurfacing slab are of the same stiffness and act as two independent beams deflecting qually under load, with no bond between them. Traffic tests on overlay pavements ave provided evidence that this formula will yield thicknesses which are very conserva-ve. Also, there is definite evidence that the bond and friction between the resurfacing nd existing slab are sufficient to cause them to act to some extent as a single unit ather than independently as assumed in the preceding formula. The U.S. Army Corps f Engineers, which has conducted more tests on overlay pavements than any other gency, makes this comment regarding the formula in Appendix II to their Rigid Pave-ent Design Manual (1):

"The results of the traffic testing at Lockbourne No. 1 and No. 2 and Sharonville o. 2 indicated that the above relationship was approximately correct when a leveling ourse, cushion course or bond-breaking course was placed between the two slabs, and at the relationship was too conservative when the overlay was placed directly on the se slab without purposely destroying the bond between the slabs."

Using a coefficient to express the condition of the existing pavement (as will be il-strated later), both the Corps of Engineers and Portland Cement Association use the eviously stated formula for calculating the required overlay thickness when bond tween the two layers is prevented by the use of some form of separating course.

After their test data demonstrated that the foregoing formula was too conservative r the design of overlays that would be partially bonded, the Corps of Engineers de-loped a revised empirical formula which expresses this relationship of the required erlay thickness to the single slab equivalent and the existing pavement:

$$h_r = \sqrt{h^{1.87} - h_e^2} \tag{3}$$

This equation gives lesser pavement thicknesses for the partially bonded condition ich results when no measures are employed to prevent bond. After the coefficient presenting pavement condition has been explained, an example will be given com-ring the overlay thickness given by this equation with the thickness given by Equation

Since concrete pavements considered for resurfacing will have variable structural lue depending on their condition, it has been necessary to introduce a coefficient, to express the pavement condition. The Corps of Engineers has used the following lues for C:

C = 1.0 - when the existing pavement is in good condition.
C = 0.75 - when the existing pavement has initial joint and corner cracks due to loading, but no progressive cracks.
C = 0.35 - when the existing pavement is badly cracked or crushed.

Using the coefficient C, the two basic equations for expressing overlay thickness uld become:

$$h_r = \sqrt{h^2 - Ch_e^2} \tag{4}$$

ich is the equation for use when bond is prevented;

and

$$h_r = \sqrt{h^{1.87} - Ch_e^2} \tag{5}$$

which is the equation for use when partial bond will be obtained.

As an illustration of the difference between the two equations, it is assumed that the existing pavements in both cases are 6 in. thick, the required thickness of an equivale single slab is 9 in. and the condition of the existing pavement in each case is such that C = 0.75. The overlay thickness required where bond is prevented would be

$$h_r = \sqrt{9^2 - (0.75 \text{x} 6^2)} = 7.35 \text{ in.}$$

The overlay thickness required where partial bond exists would be

$$h_r = \sqrt{9^{1.87} - (0.75 \text{x} 6^2)} = 5.82 \text{ in.}$$

The difference in thickness of the required overlays is 1½ in. or 20 percent less thickness where partial bond is obtained.

While these empirical equations can hardly be classed as "scientific means" of evaluating the additional strength provided by concrete overlays, their value as design criteria has been established in tests conducted by the Corps of Engineers in the development of design criteria for airfield pavements. The test program included 21 different test items of rigid overlay on rigid pavement where a "partial bond" conditio existed. Loading on the overlay sections ranged from a 20,000-lb single-wheel load to a 325,000-lb multiple-wheel gear load. The test sections were trafficked to either a predetermined number of coverages or to failure, depending on the adequacy of desig The Rigid Pavement Design Manual (1) is again quoted with reference to the tests:

"The Pavement Overlay Investigation was initiated for the purpose of obtaining dat to aid in the formulation of design criteria for overlay pavements. This objective wa accomplished in stages by construction of a series of full-scale test sections. After completion of testing of each successive test section, the results were analyzed and formulated into tentative design criteria. Each test section incorporated the results of previous projects and included new variables of loading, type and thickness of over lay, thickness and strength of the concrete base pavement, strength of subgrade, gea configuration, and total test load.... The equations used for determination of concrete overlay thickness have been developed from a correlation of theoretical develop ments and actual test results."

The design criteria based on the extensive investigations of concrete overlays by the Corps of Engineers should provide dependable guidance for highway engineers. I should be pointed out that the Corps has made further revision to the design equation used for partially bonded concrete resurfacing with the required overlay thickness expressed as

$$h_0 = \sqrt[1.4]{h_d^{\,1.4} - Ch^{1.4}} \tag{6}$$

in which

h_0 = thickness of concrete overlay, in inches;
h_d = thickness of required single slab, in inches;
C = coefficient for pavement condition; and
h = thickness of existing slab, in inches.

This equation has been compared with the equation

$$h_r = \sqrt{h^{1.87} - Ch_e^2}$$

used previously in this discussion (and in Portland Cement Association literature) as he basis for design of partially bonded overlays. In the range of normal resurfacings hat would be applicable to highways, the two equations result in overlays with less than /10-in. difference in thickness. The Corps has found this revised formula more feasible or use in designing either exceptionally thick overlays for thin pavements or thin overays for thick pavements. These design requirements are seldom encountered exept in dealing with pavements for military airports. Design agencies may prefer to se the revised Corps of Engineers formula, however, since it will provide a dependable nethod for the design of concrete overlays for any combination of pavement and overlay hickness.

The tests conducted by the Corps of Engineers have conclusively proved that the omposite pavement has greater load-carrying capacity when the concrete overlay is artially bonded to the base slab. This evidence, together with other comparatively ecent developments, should lead to serious consideration of the advantages of fully onded resurfacing as a means of strengthening existing pavements.

Development studies by the Portland Cement Association during the past several ears have demonstrated the feasibility of bonding two layers of pavement together by conomical methods. A report of the laboratory results was published in the Proeedings of the Highway Research Board, Vol. 35, 1956 (2). The practicability of field onstruction methods for placing bonded resurfacing has been demonstrated on comaratively large-scale projects built on airfield pavements during the past four years 1). All the PCA test slabs and large-scale projects referred to utilized portland ement grout as a bonding agent. Recent repetitive load tests on full-scale slabs, built itdoors at the Portland Cement Association Laboratories, indicate that a composite vement of two bonded layers has the same strength as a monolithic slab of the same ickness. Using the cement grout bonding method, the Corps of Engineers constructed 1 11-in. overlay on a 17-in. base slab at Sharonville, Ohio. This overlaid pavement s tested in comparison with a single-layer 28-in. slab by trafficking with a test rig ich applied a dual-tandem gear load of 325,000 lb on the pavement. After more than),000 coverages with this loading, it was concluded that the bonded section performed a monolitic slab. This conclusion was based, not only on the equivalent durability the layered section, but also on deflection measurements and strain gage readings.

The performance of bonded overlays in test installations is such as to indicate that is type of construction is feasible and economical when structural improvement is cessary. Pavements on two airports, one civil and one military, were strengthened this method within the past two years. A 5-in. bonded overlay on an aircraft park-g apron is scheduled for construction early this year by the U.S. Navy.

The performance of these resurfaced pavements should provide information for rther evaluation of this method of construction.

In paragraph five of his conclusions, Mr. Gould states: "Reflection cracking for gid-type overlays, regardless of thickness, may not be a problem since no studies this nature have been reported to the committee."

In this connection it is necessary to report that joints and random cracks in the base vement will generally be reflected in the concrete resurfacing unless some preventae measures are taken. The practice generally used to prevent joint reflection can easily described by quoting again from the Corps of Engineers design manual:

"Joints in the overlay pavement will coincide with all joints of the base pavement. is not necessary for joints to be over like joints." (1)

The second statement in the quotation requires some amplification. Many old conete pavements were built with expansion joints at relatively short intervals. Expanon joints can usually be omitted in the resurfacing or placed at much longer intervals. ntraction joints should be placed over the expansion joint location. This procedure plies to overlays that are partially bonded. In placing thin layers of bonded resuring, it is considered necessary to match existing joints in both location and kind.

Some success has been experienced in preventing reflective cracking by the use of parating courses between the base slab and overlay. There is not sufficient data ilable, however, to indicate the minimum thickness of separating course that will completely effective. There are indications that any type of bond breaker will reduce amount of reflective cracking, but as has been previously discussed, the use of a nd breaker creates the requirement for a thicker overlay.

If the old pavement is badly cracked, the use of distributed steel is probably the most dependable method of minimizing cracking in the resurfacing. This is true for both bonded and partially bonded overlays. If distributed steel was used in the origina pavement, the amount of steel in the resurfacing should be based on the thickness of the overlay slab. If the old slab does not contain distributed steel, the amount of steel should be based on the combined thickness of the two slabs.

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Five-Year Performance of Welded Wire Fabric in Bituminous Resurfacing

EGONS TONS, Assistant Professor of Transportation Engineering, and
ALEXANDER J. BONE, Associate Professor of Transportation Engineering, M.I.T.; and
VINCENT J. ROGGEVEEN, Associate Professor of Civil Engineering and Transportation, Stanford University

The primary objective of this paper is to report on placement and performance of reinforced bituminous concrete resurfacing over old portland cement concrete roadway.

In 1955 two sections of road (in Massachusetts) totaling three miles in length were resurfaced with a 3-in. bituminous concrete reinforced with various styles of welded wire fabric. The main purpose of this reinforcement was to prevent reflection cracks caused by underlying slab movements.

In one of the test roads the resurfacing was reinforced with strips placed above the slab joints only, whereas the other had various styles of continuous reinforcement extending into the shoulders. Altogether 25 test sections were installed with comparable unreinforced control sections.

The method of placing the reinforcement was simple and did not cause much difficulties.

After five years under traffic and weathering all reinforced test sections had lower cracking indexes than comparable controls. The best performance was by 3 x 6, $^{10}/_{10}$ welded wire fabric placed in continuous rolls where transverse cracking was about one-eighth of that found in the control, and longitudinal cracking was negligible.

THE CAUSES of reflection cracking have been described and discussed in numerous [pu]blications and it is evident that the amount of cracking will depend on many different [fac]tors. In this paper the emphasis is on reflection cracking as it has been observed [in] Massachusetts.

STRAINS IN RESURFACING CAUSED BY UNDERLYING SLAB MOVEMENTS

Many miles of old portland cement concrete slabs which have been covered with a [...]- to 3-in. bituminous resurfacing, are about 8 in. thick, 10 ft wide and 57 ft long. [Th]ey have been placed on a 12-in. gravel base with about 0.13 percent of longitudinal [re]inforcement. These roads were constructed in the thirties using only expansion [joi]nts with four 1-in. load transfer dowels for each 10-ft joint.

[Ho]rizontal Joint Opening

The amount of horizontal joint opening was studied in several Massachusetts loca[tio]ns over a period of years. The measurements show that about 80 percent of the [tra]nsverse expansion joints open 0.05 in. or more during a one-year cycle. The [larg]est joint opening measured was 0.18 in. with a maximum daily variation of about [0.0]4 in. Laboratory strain tests have shown that a 4-in. long bituminous concrete specimen can be strained about 0.04 to 0.05 in. before it breaks (12). A comparison between this obtained strain for the Massachusetts mix and the measured joint opening on a road indicates that due to horizontal strains alone at least 80 percent of the resurfaced transverse joints will have reflection cracks above them within a relatively short period.

Vertical Differential Movements

To estimate the vertical differential movements of the slabs numerous field measurements were made using a truck and a deflection gage. The maximum relative deflection at a joint was 0.09 in. with about 50 percent of the measurements being 0.005 in. or less. Thus, it appears that the vertical joint deflections are relatively small compared to the horizontal movements. Laboratory measurements indicate that a crack might not appear in the surfacings if such low deflections are applied without any strains due to horizontal slab contraction. These studies further show that the differential vertical movement due to traffic load causes a flexural distortion in the resurfacing rather than shear deformation (11).

Combined Effects

Although field observations and laboratory measurements show that the main reaso for reflection cracking in Massachusetts is the horizontal joint opening or tensional stress, it often can be the combined effects of both horizontal joint opening and vertica deflection movements which cause the cracking. Damaging axial and flexural tension in the resurfacing at a joint affect not only the immediate area above the joint, but also extend some distance on each side. Once a crack has appeared it has a tendency to deteriorate and widen.

PURPOSE OF STUDY

The main objective of this study was to test on the road the effectiveness of various welded wire fabric types as a reinforcement in bituminous concrete resurfacing against reflection cracking (in Massachusetts). The second purpose was to compare the field behavior of welded wire fabric reinforcement with data obtained from tests i the laboratory.

PREVIOUS INSTALLATIONS OF WELDED WIRE FABRIC REINFORCEMENT

The Walpole and Raynham Test Roads described in this paper were built in 1955. Before that, numerous experimental wire fabric installations had been placed in serv and various results were obtained with the oldest dating back as far as 1945. These field installations were placed without any engineering calculations or experience, regardless of whether the wire fabric reinforcement was used for airports, highways, city streets, etc. The main emphasis in these experiments was on transverse joint reinforcement using both strip and continuous welded wire fabric.

TEST ROADS IN MASSACHUSETTS

The planning and execution of a test road involves cooperation and interest by num ous agencies. The interest of the Commonwealth of Massachusetts in reflection crac prevention was the principal reason for making road tests in Massachusetts. The pe sonnel of the state-supported Joint Highway Research Project of the Massachusetts Institute of Technology and the Department of Public Works had been working on the reflection cracking problem for several years and had accumulated a considerable amount of data which were useful in planning actual welded wire fabric reinforcemen test installations. Another reason for having the test roads in Massachusetts was th climatic conditions which are quite severe and to a degree typical of the northern states.

PLANNING SCOPE OF PROJECT

Planning and building a field test calls for consideration of many variables. Only those who have participated in an actual execution of a test road can fully appreciate the complexity of such an undertaking. The condition of the old portland cement concrete roadway and the types and sizes of wire fabric reinforcement to be used in the resurfacing were the two major variables. Although the type and size of reinforcement could be selected, the condition of the old roadway was much more difficult to control.

Desired Variables and Length of Test Road

During the preliminary planning it was assumed that the test road itself would be niform for all test sections. Therefore, the major variables were reduced to the type of vire fabric and its placement. The following major factors were discussed and considered: (a) wire size, (b) transverse wire spacing, (c) longitudinal wire spacing, (d) strip reinforcement versus continuous reinforcement, (e) position of the wire fabric in he resurfacing, and (f) ways of holding the wire fabric down.

If all these variables were to be included in this research the total number of test ections would have exceeded 10,000. As this was a practical impossibility a small umber of tests were agreed on which are described later in this paper.

umber of Transverse Joints Needed to Obtain Meaningful Data

To compare two types of wire fabric the test conditions have to be similar. This ften is not possible where a large number of test sections is involved. The slight variions in a test road of appreciable length can affect the results and prohibit a direct omparison between a section at one end of the test with that at the other. Therefore, was decided that control sections would be placed between the test sections and the erformance of the various wire fabrics would be compared directly with the adjacent ontrols. To decide on the minimum number of test joints, condition survey data from umerous field installations over the past five years were studied. It was found that at east 12 transverse joints would be necessary to keep a 5-yr cracking index within 10 ercent, between two adjacent 12-joint test sections, 95 percent of the time. It was so anticipated that some joints would have to be eliminated due to construction faults d inherent irregularities in the portland cement concrete base. Therefore, the final mber specified was 16 to 20 joints for each test and control section.

ractical Limitations

The amount of road surface available as well as budgetary limitations finally led to choice of 25 sections, excluding controls.

After the number of test sections was decided on a special committee (consisting of re fabric manufacturers and the staff of the Joint Highway Research Project at Masshusetts Institute of Technology) listed the types of wire fabric that were to be used. e practical limitations favored the use of standard size wire fabrics rather than ecial fabrications. An attempt was made to incorporate various wire sizes in tests th the 10-gage wire dominating because it is easy to install in a 3-in. resurfacing.

les of Wire Fabric Chosen

Besides the common spacings (4 by 4 in., 6 by 3 in., and 3 by 6 in.) for the 10-gage re, other dimensions and wire sizes were also used. The largest wire was 6 gage cause the committee felt that anything stiffer than that would not be compatible with relatively thin flexible overlay. The smallest wire was 14 gage. One section of test road in which continuous reinforcement was planned contained wire fabric in lls. In those installations the longitudinal wire was 10 gage or thinner because it s too difficult to flatten out a roll when stiffer wire was present.

e of Mats and Types of Reinforcement Chosen

As mentioned before, the most critical points where cracking is usually more severe are the transverse joints. Therefore, attempts have been made in the past to place a strip of reinforcement right above the joints to prevent a reflection crack. To protect the longitudinal joints from crack appearance, similar strip reinforcement would be necessary also in these locations. This led to the conclusion that three major types of reinforcement should be tried: (a) strip reinforcement over transverse joints only, (b) strip reinforcement over longitudinal joints, and (c) a continuous reinforcement which covers the whole portland cement concrete roadway and extends a short distance beyond the longitudinal edges of the slabs. Small test installations in the past using strip reinforcement have indicated that the strip width of 3 to 4 ft is not always enough to curb reflection cracking (3, 4). If the strip is narrow there is a tendency to get a crack at the edge of the reinforcement. Therefore, about 7-ft wide reinforcement was adapted for the transverse joints. A few installations using 5- and 10-ft wide strip reinforcement were also tried.

The continuous reinforcement had most of the fabric in rolls. The length of the rol was 57 ft or the same length as the underlying slabs. This was chosen to have the lapping of the rolls far away from the joints at the center of the slabs.

In three test sections a continuous sheet of reinforcement was planned using 18-ft long wire fabric.

TEST ROAD SPECIFICATIONS

After deciding on the number of test sections and the types of wire fabric to be used a search for a test road was undertaken which would fit the following requirements:

1. The test section should contain at least two adjacent lanes of old portland cemen concrete.
2. The horizontal curves in the test road should not be less than 600-ft radius and the grades should be less than 2 percent.
3. The test road should have a uniform subgrade.
4. The underlying concrete slabs should not have longitudinal reinforcement failur
5. The slabs should exhibit similar crack frequency characteristics.
6. Control sections of identical length should be spaced in between the test section and at least 12 transverse joints or 120 ft of joint should be usable in each test.
7. The test sections should carry similar traffic.

FINAL SELECTION OF TEST SITES

To place the 25 test sections with appropriate controls, about three miles of two-la road were required. As it was not possible to find such a length of road under unifor conditions in one location, two separate test roads were selected. In this paper they are referred to as Raynham and Walpole Test Roads. At the Raynham Test Road, str reinforcement was used and at the Walpole Test Road continuous wire fabric reinforce ment (see Figs. 1 and 2 for wire fabric styles and sizes).

Raynham Test Road

The Raynham Test Road consisted of a three-lane reinforced cement concrete pave ment 8 in. thick, placed on a 12-in. gravel base. It had bituminous macadam shoulde 3 to 5 ft wide, on each side.

The original portland cement concrete was in a good condition structurally. The majority of the 57-ft slabs had but one transverse crack with a very small percentage having two or more. The surface was scaled in places and many bituminous skin pat had been placed. The joints were about 1 in. wide and had worn considerably causing traffic to thump, but there was no evidence of major settlements.

The Raynham installations consisted of two types of strip reinforcement tests: (a) reinforcing of transverse joints with welded wire fabric sheets, and (b) reinforcing of longitudinal joints with wire fabric sheets (Table 1).

Walpole Test Road

The Walpole Test Road was a two-lane reinforced concrete pavement 8 in. thick, placed on a 12-in. gravel base. It had bituminous macadam shoulders 3 ft wide on each side. The concrete lanes were 10 ft wide, the slabs averaged 57 ft in length. The resurfacing extended over both shoulders.

The original concrete was in fair structural condition, although the slabs averaged two to three transverse cracks each. The surface was scaled and contained a few bituminous skin patches. The rough-riding transverse joints were about 1 in. wide but did not show appreciable settlement.

The Walpole installation consisted of two types of continuous reinforcing tests: (a) continuous reinforcing of entire slabs using wire fabric sheets, and (b) continuous reinforcing using 57-ft long rolls (Table 2).

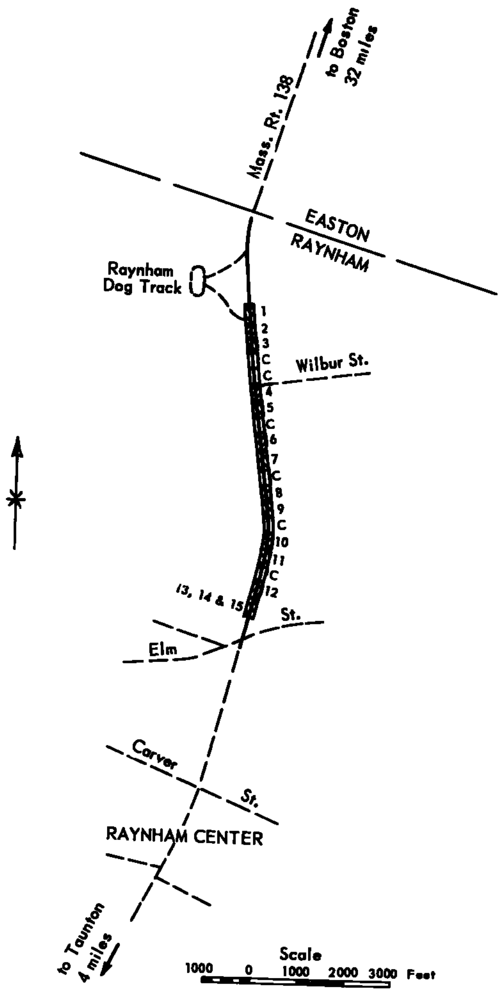


Figure 1. Location map of test road, Raynham, Mass. Numbers indicate test sections, C indicates control sections.

<u>omparison of Raynham and Walpole ites</u>

Even though the two test roads are uite similar as far as the underlying ortland cement concrete is concerned ere are some differences that should e pointed out. One is the amount of ansverse cracking. Whereas in Raynham ly one or two transverse cracks were und on each slab, Walpole had two or ree cracks per slab. This was one of e reasons why a continuous reinforcement was chosen for the Walpole Test oad. On the other hand, it must be emasized that intermediate cracking beeen joints affects the amount of horintal joint opening and closing. Therere this should be kept in mind when aking comparisons between Raynham d Walpole results.

The amount of traffic on both sites was medium heavy but in Raynham it was distrited over three lanes whereas in Walpole, due to the relatively narrow width, the ffic is concentrated in one lane in each direction.

PRELIMINARY WORK ON TEST SITES

Before the actual surfacing was started repairs were made to old road surface. The nsverse joints still contained bituminous sealer and they were untouched. In some as, where scaling of the slabs was extensive, new patches were applied. Some rk was also performed on the shoulders and new side drains were installed where essary.

TRANSPORTING AND DISTRIBUTING WIRE FABRIC REINFORCEMENT

The wire fabric reinforcement was received from the manufacturers already cut ize. In Raynham the fabric was carried to the site on a flat-bed truck and distried beside the test sections of roadway. It was then placed over the joints immediately

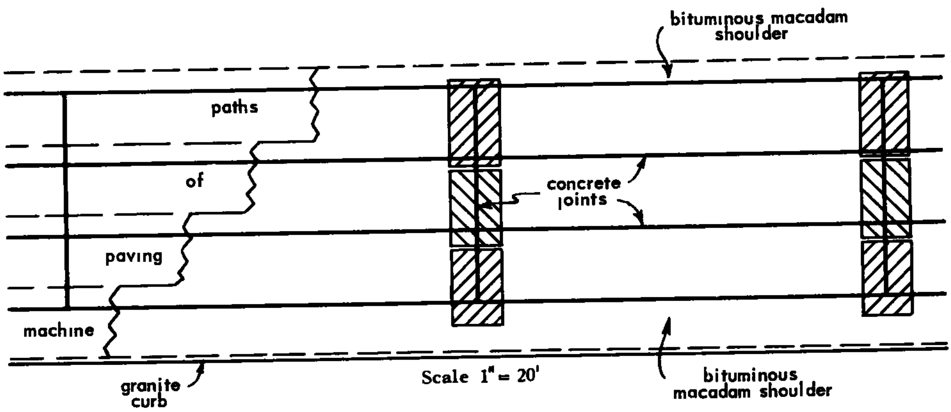


Figure 2. Placement of fabric on road. Transverse joint reinforcing with sheet fabric.

TABLE 1

WELDED WIRE FABRIC (SHEET REINFORCING) USED AT RAYNHAM TEST SITE

Test No.	Total Sheets	Fabric Style	Spacing of Wire in Inches: Transverse Dimension	Spacing of Wire in Inches: Longitudinal Dimension	Gage of Wire: Transverse Wire	Gage of Wire: Longitudinal Wire	Outside Dimensions of Sheets in Place: Transverse Dimension	Outside Dimensions of Sheets in Place: Longitudinal Dimension
(a) Transverse Concrete Joints								
1	18	3×6 10/6	3	6	6	10	11	6'6"
2	18	6×6 10/10	6	6	10	10	11'	5'
3	18	6×6 10/10	6	6	10	10	11'	6'6"
4	18	3×6 10/6	3	6	6	10	11'	5'
5	18	6×3 10/10	6	3	10	10	11'	6'6"
6	18	3×6 10/10	3	6	10	10	11'	7'6"
7	18	4×8 10/6	4	8	6	10	11'	6'6"
8	18	4×4 10/10	4	4	10	10	11'	6'6"
10	18	6×6 8/8	6	6	8	8	11'	7'6"
11	18	12×6 8/6	12	6	6	8	11'	7'6"
12	36	12×6 8/6	12	6	6	8	5'6"	10'

(b) Longitudinal Concrete Joints

Test No.	Total Sheets	Fabric Style	Spacing of Wire in Inches: Transverse Dimension	Spacing of Wire in Inches: Longitudinal Dimension	Gage of Wire: Transverse Wire	Gage of Wire: Longitudinal Wire	Outside Dimensions of Sheets in Place: Transverse Dimension	Outside Dimensions of Sheets in Place: Longitudinal Dimension
13	24	4×4 10/10	4	4	10	10	5'	12'
14	24	6×3 10/10	6	3	10	10	5'	12'
15	24	6×6 8/8	6	6	8	8	5'	12'

TABLE 2

WELDED WIRE FABRIC (CONTINUOUS REINFORCING) USED AT WALPOLE TEST SITE

Test No.	Total Sheets	Fabric Style	Spacing of Wire in Inches		Gage of Wire		Outside Dimensions of Sheets in Place	
			Transverse Dimension	Longitudinal Dimension	Transverse Wire	Longitudinal Wire	Transverse Dimension	Longitudinal Dimension
(a) Sheet Fabric								
16	96	6×12 8/6	6	12	6	8	6'	18'
	32	6×12 8/6	6	12	6	8	5'	18'
17	96	6×6 8/8	6	6	8	8	6'	18'
	32	6×6 8/8	6	6	8	8	5'	18'
18	96	3×6 10/6	3	6	6	10	6'	18'
	32	3×6 10/6	3	6	6	10	5'	18'

(b) Roll Fabric

Test No.	Total Rolls	Fabric Style	Spacing of Wire in Inches		Gage of Wire		Outside Dimensions of Rolls in Place	
			Transverse Dimension	Longitudinal Dimension	Transverse Wire	Longitudinal Wire	Transverse Dimension	Longitudinal Dimension
19	30	6×3 10/10 (1)	6	3	6	10	6'	57'
	10	6×3 10/10	6	3	6	10	5'	57'
20	30	3×6 10/10	3	6	10	10	6'	57'
	10	3×6 10/10	3	6	10	10	5'	57'
21	30	4×4 10/10	4	4	10	10	6'	57'
	10	4×4 10/10	4	4	10	10	5'	57'
23	30	6×3 10/10	6	3	10	10	6'	57'
	10	6×3 10/10	6	3	10	10	5'	57'
24	30	3×6 10/10 (1)	3	6	10	10	6'	57'
	10	3×6 10/10 (1)	3	6	10	10	5'	57'
25	Test 25 placed with total resurfacing thickness increased from normal 3" to 3 ¾", without reinforcing.							
26	30	3×3 12/12 (2)	3	3	12	12	6'	57'
	10	3×3 12/12 (2)	3	3	12	12	5'	57'
27	30	2×2 14/14 (2)	2	2	14	14	6'	57'
	10	2×2 14/14 (2)	2	2	14	14	5'	57'

lote (1) - Placed with Transverse wire on top
lote (2) - Galvanized wire

front of the trucks delivering the mix to the paving machine. Similarly, the Walpole nforcement was transported to the site and distributed along the roadside.

PLACING OF REINFORCEMENT

In both Raynham and Walpole tests the wire fabric was placed directly on the portland nent concrete with longitudinal wires up. There were a few installations where the nsverse wires were up (Tables 1 and 2).

In Raynham the wire fabric strips were nailed to the pavement using metal clips and a Remington stud driver (Fig. 10). About four to six clips were placed along the side of the fabric nearest the paver thus preventing the wire reinforcement from being caught in the machine. The transverse sheets of fabric were layed with little effort. Occasionally where the crown of the road was lowered by reducing the usual $1\frac{3}{4}$-in. thickness of the binder, an additional one or two studs and clips were used. No sleds or other hold-down devices were needed. The tracks of the Barber-Greene finisher did not cause any disturbance of the fabric except where an attempt was made to pave over strips which had not been fastened down.

On the Walpole Test Road the rolls were first flattened out to their full length of about 57 ft (Figs. 12 and 13). The ends were overlapped in the middle of the concrete slabs by about $1\frac{1}{2}$ of the transverse wire spacing. However, the curl of the rolls, when rolled out, caused the lap to have a tendency to rise up off the concrete. This was remedied by increasing the overlap to 18 in. and fastening both ends of the rolled out fabric together with hog rings. Thus a lane of smooth reinforcement was obtained.

In the three sections where continuous sheet reinforcement was laid, the sheets were overlapped about $1\frac{1}{2}$ of the transverse spacing and tied together with hog rings. One or two rings were also occasionally used to fasten the longitudinal edges of adjacent sheets together whenever they showed a tendency to curl (see Figs. 1 to 14 for laying plans and operations).

PLACING BINDER COURSE ON TOP OF FABRIC

Massachusetts Type I binder course mix with $\frac{7}{8}$-in. maximum size aggregate was used to cover the fabric with a $1\frac{3}{4}$-in. layer. Specifications for this mix are given in the Appendix. For both test roads a Barber-Greene finisher was used. The work was done under contract and, therefore, the test installations were actually laid with the same equipment and construction techniques as on the usual resurfacing job.

On the Walpole Test Road the fabric was held down by a sled made of railroad rails and attached to the Barber-Greene paving machine (Fig. 14). This sled was dragged ahead by the front frame of the finisher with the individual rails trailing behind and riding directly over the fabric for their length. One rail of the sled was always kept on the edge of the fabric to prevent it from curling up.

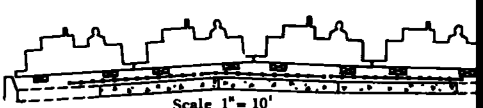


Figure 3. Cross-section showing positions of paving machine passing over fabric

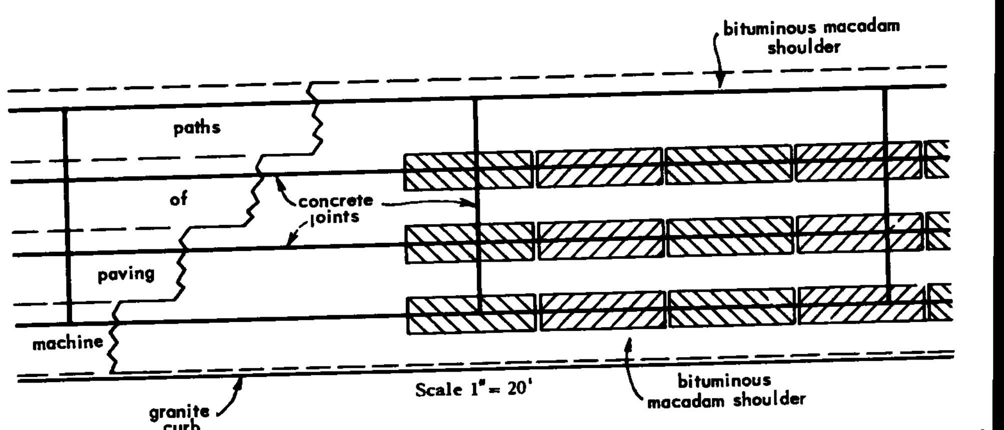


Figure 4. Placement of fabric on road. Longitudinal joint reinforcing with sheet fa- ric.

During the paving operations traffic
vas restricted to one side of the road.
\s soon as the binder course was com-
leted in one lane, traffic was switched
o the other side while the remaining
are area was covered with fabric and
aved.

AREAS WHERE CAUTION IS ADVISABLE

During the paving operations in Rayn-
am very few difficulties were encounter-
d in placing the binder course over weld-
d wire fabric reinforcement and 96 per-
ent of placement was completed without
ny incident. In the Walpole Test Road
everal small difficulties developed with
e continuous reinforcement. In the
ontinuous roll section sometimes a wave
as created by the paving machine as it
oved forward. This wave usually ex-
nded itself at the next lap at the middle
the slab but sometimes it curled up
d became caught in the machine. In
me locations where the binder course
ppened to be quite thin due to surface
regularities of the old portland cement
ncrete, the screed caused a relatively
gh frictional force and pushed a local-
ed wave of the fabric ahead of it. This
sulted in distortions of the reinforce-
ent and cracking in the binder behind the
isher. Some wire sections had to be
t out for this reason. There were also
few difficulties due to the curl of the
lls which sometimes caused it to rise
the binder immediately behind the paver at the lap joints. Under roller the binder
ther cracked or became springy over the lap, and the ends of the rolls tended to pro-
ude through the mix. This was remedied to a great extent by increasing the length
the lap. There were instances where the mix had been spilled on the pavement be-
re the reinforcement was laid, thus creating a high spot and causing the paver to
tch the fabric as it passed over it.

In the Walpole Test Road the top course was not placed immediately after the binder
plication and the binder course was subjected to considerable traffic use. As a re-
lt after about 24 hours a few springy spots developed in some sections, each about

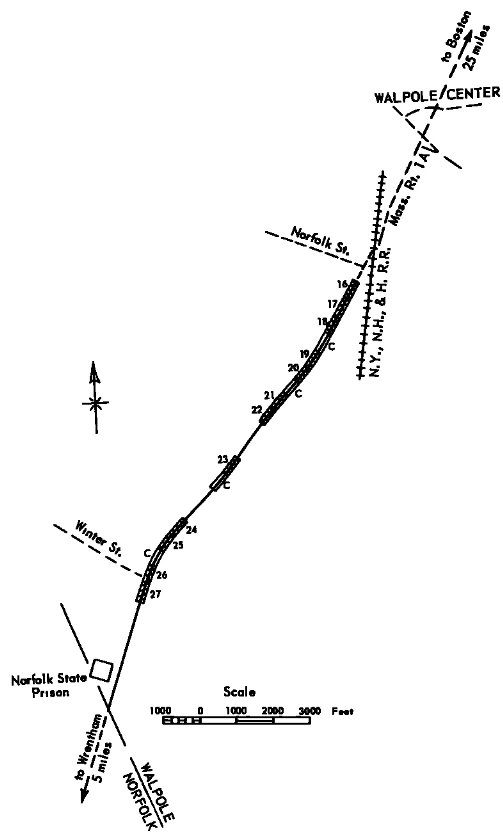
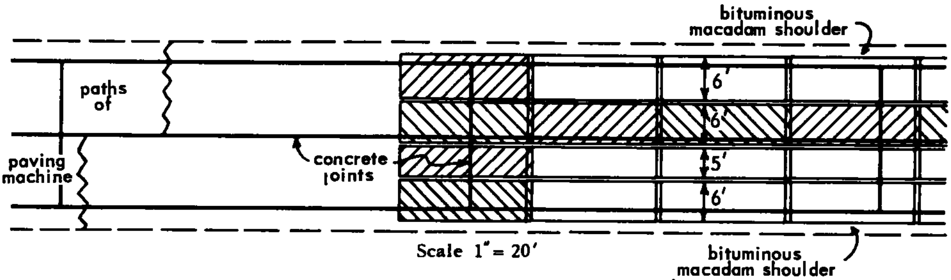


Figure 5. Location map of test road, Walpole, Mass. Numbers indicate test sections, C indicates control sections.



ure 6. Placement of fabric on road. Continuous reinforcing with sheet fabric.

Figure 7. Cross-section showing positions of paving machine passing over fabric.

12 to 18 in. in diameter. The areas in which this developed were over fabric which had curled up in the mix where the bituminous concrete binder course was relatively thin. In these springy spots the binder was removed, the wire cut out and the area patched before the top mix was placed. These defects have not shown up in the top surface so far.

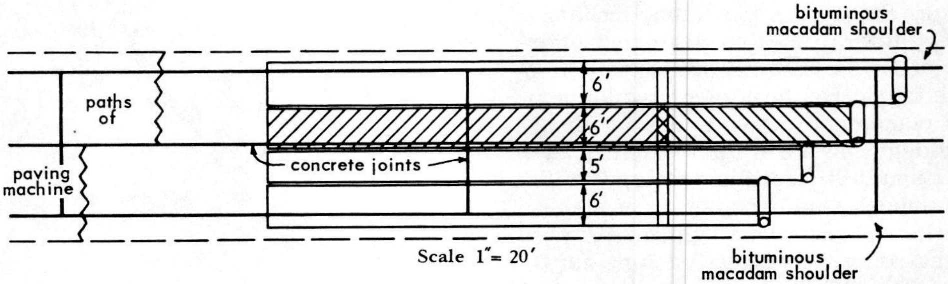


Figure 8. Placement of fabric on road. Continuous reinforcing with roll fabric.

PLACING TOP COURSE

The mix used to cover the binder course was Massachusetts Type I top mix, 1¼ in thick. This was laid by a Barber-Greene finisher without any special precautions or provisions being required. The mix specifications are given in the Appendix.

CONDITION SURVEYS AND INSPECTION

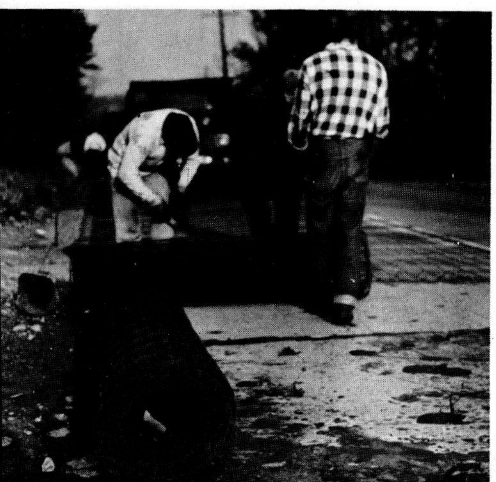
The paving operations on the Raynham test site began on September 28, 1955, and were completed on October 18, 1955. Th

Figure 9. Typical installation of transverse reinforcing over joint.

Figure 10. Remington stud driver used to hold down fabric sheets.

gure 11. Typical installation of longitudinal reinforcing over joint.

Figure 12. Placing continuous sheet reinforcing.



gure 13. Placing continuous roll reinforcing.

Walpole test was started on October 26 and completed on November 17, 1955.

The first inspection of the test roads was made in the fall of 1955. From that time on two major condition surveys were made each year: the first one during the winter months, preferably January or February, and the second one during the summer months. The surveys were done according to an established method by plotting the crack lengths and widths on forms specially prepared for this purpose. These records were then analyzed and the various types of cracks classified according to the width and type.

Several additional inspections were made using photographic and visual methods. The development of defects and cracking in the various sections has been recorded from survey data and is presented in subsequent paragraphs.

CRACK DEVELOPMENT WITH TIME

It has been observed during the past 10 years that under Massachusetts conditions flection crack development in a Type I mix is usually delayed for about a year after acing the resurfacing (8). This holds true also for the Raynham and Walpole Test oads. Only few cracks were observed on the Walpole Test Road during the winter of 56 and the first cracks were observed on the Raynham test in September 1956. The itial large-scale condition survey in February 1957, revealed considerable cracking cause the winter was severe and the crack development was unusual as compared to st experience. During this second winter more than 50 percent of the transverse ints in the control sections had cracks, whereas other test sections showed lower percentages or none at all.

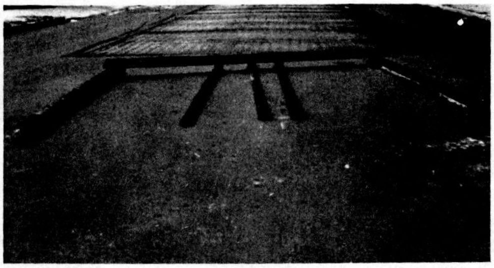


Figure 14. Sled used on Barber-Greene paver for holding down continuous reinforcing.

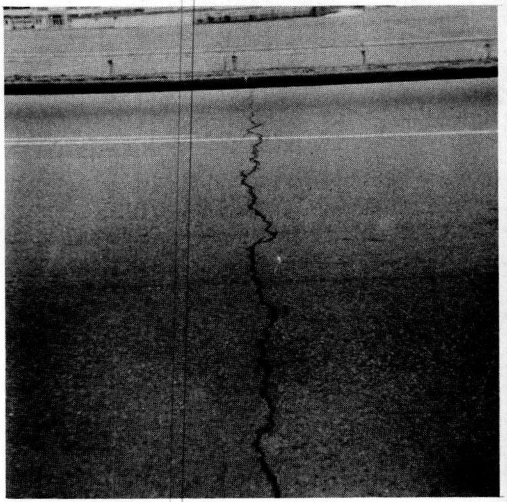


Figure 15. One of the cracks in non-reinforced section.

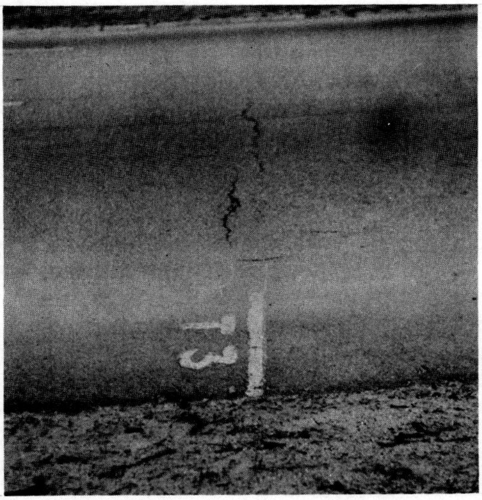


Figure 16. Crack above joint in reinforced section, Raynham.

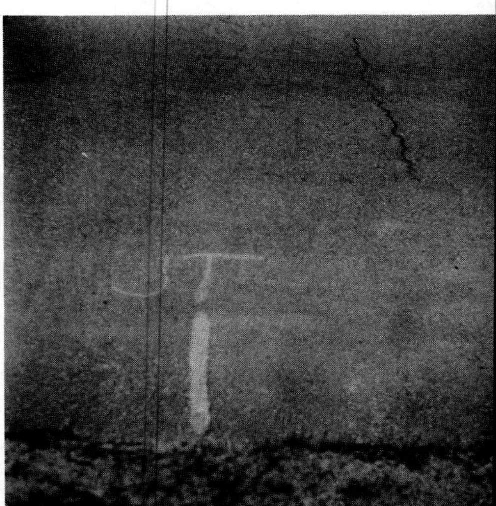


Figure 17. Crack at edge of reinforcement, Raynham.

FIVE-YEAR FIELD RESULTS

All sections have been subjected to five years of traffic and weathering. Data compiled from winter and summer surveys indicate that the differences between the amount of cracks during the warm period and the cold period are not great except for the crack width. Some small cracks tend to close up during the summer especially during the first years. In this report comparisons between the various test sections will be made on basis of winter surveys when the cracks are at their widest opening.

Raynham—Transverse Joint Strip Reinforcement

The width of the strip reinforcement in Raynham varied between 5 and 10 ft. The 5-ft wide strips are herein called narrow, the 6½- to 7½-ft strips are called medium wide and the 10-ft strips are designated as wide. The percentage of total reflection cracking above the transverse joints is given in Figure 19. This figure contains six

Figure 18. Longitudinal crack narrows down and stops at reinforced section (bottom), Walpole.

separate bar graphs which were necessary o compare each of the reinforced test sections with its adjoining control section. t can be seen that the amount of cracking both in the test sections as well as in the controls increased with time, with the control sections developing cracks at a aster rate than the test sections. Because the width of the cracks varied from ne area to another, the distinction beween "wide cracks" and "narrow cracks" vas set at ⅛ in. The cracks wider than ⅛ in. are represented by the shadowed art of the bar, whereas narrow cracks ave not been shaded. This dividing oint at the ⅛-in. width was selected because usually cracks that are ⅛ in. and ess in width are not sealed in maintenance operations. During the actual field urveys the narrow cracks were subdivided nto two categories: (a) hair cracks and) cracks of about ⅛-in. width. The wide racks had several other recorded subclassifications. The records of these are vailable, but for the sake of simplicity nd brevity, the detailed data have not been included in this paper.

In the case of the strip reinforcement the cracks that are found appear in two disnct places: (a) immediately above the old joint and (b) at the edge of the reinforceent strip. In practically all cases the cracks appeared either above the joint or at e edge and very seldom in both places. In those few instances where two parallel racks had appeared the lengths of the cracks were counted as one, and only the crack idth was increased in the classification. In this manner the total percentage of crackg cannot exceed 100 percent.

Figure 20 shows a division between the amount of edge cracks and cracks right over e joint. Here again, the hatched area designates wide cracks, whereas the blank rt of the bar denotes the narrow cracks. The arrangement of the bar graphs is ightly different in Figure 20 than in Figure 19, showing for each test section the development or increase in cracking with time. The control sections are not included in is comparison because they contain only one kind of crack; that is, only those immediately above the joint.

ynham—Strip Reinforcement over Longitudinal Joints

As given in Table 1, three test sections were established using 5-ft wide strip reinrcement over the longitudinal joints only. The relative comparisons between the conol and the three tests are given in Table 3. The percentages are based on about 240 ft longitudinal joint for each test section.

alpole—Continuous Reinforcement Sections

As given in Table 2, the Walpole test sections were constructed using continuous inforcement not only over the whole area of the portland cement concrete slabs but so extending over each side onto the shoulders (Figs. 6, 8, and 12).

The cracking percentages for the transverse joints are shown in Figure 21. Three the test sections contained wire fabric in sheets, whereas the rest of the test had the inforcement in rolls of approximately the same length as the slabs. In Figure 21 e various test sections are compared with the adjacent control sections and, therere, each section of the bar graphs has to be studied independently. For instance, sts 20 and 21 have an adjoining control section, C-3, which cannot be directly

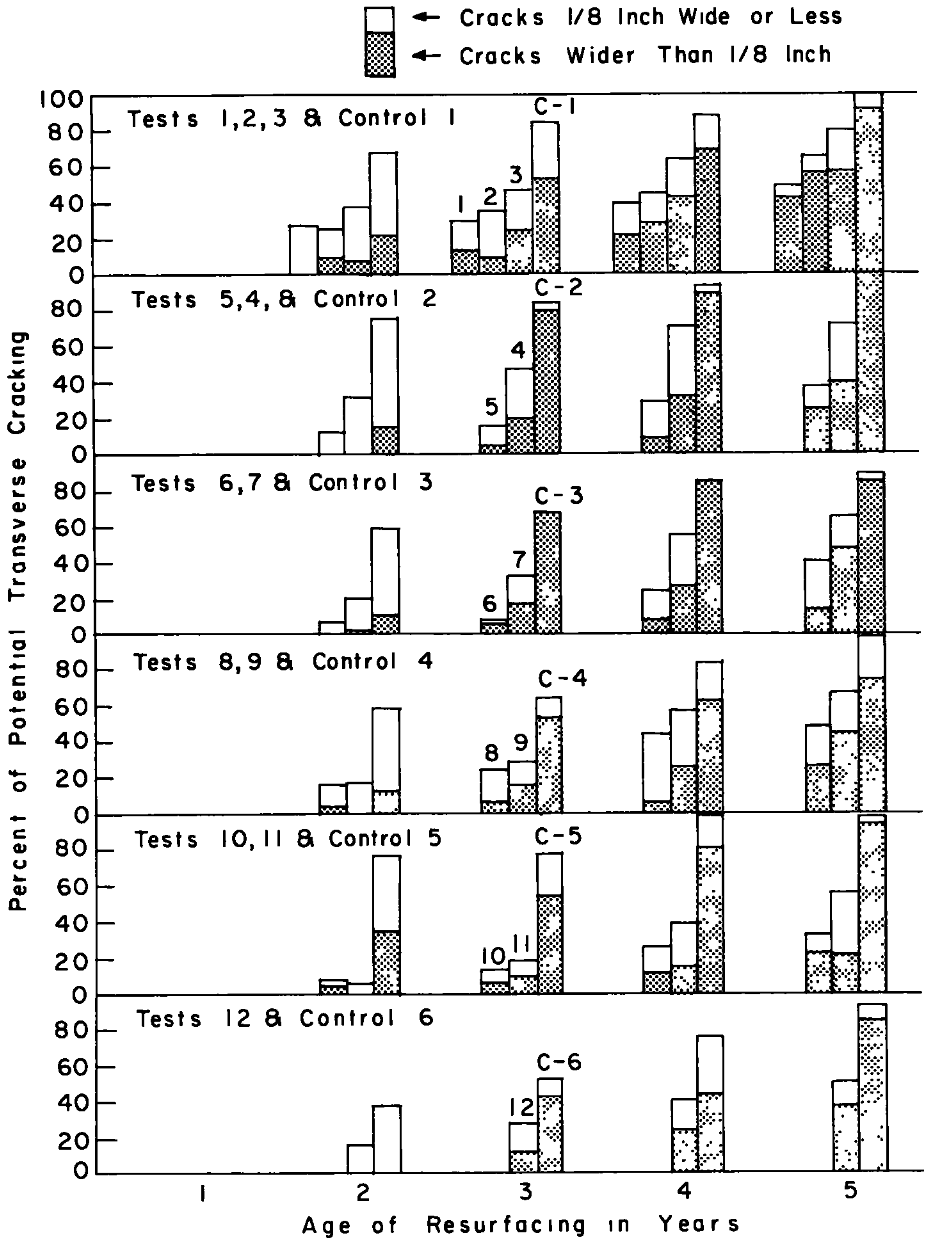
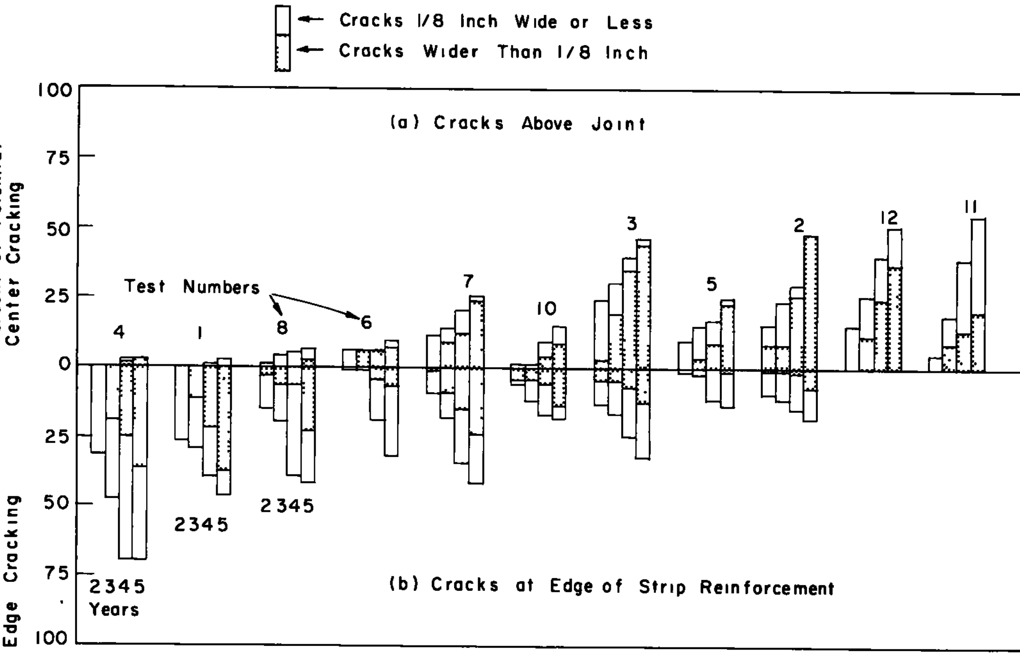


Figure 19. Comparison of transverse crack development for the Raynham Test Road using strip reinforcement.

compared with tests 16 and 17 because the control section adjacent to tests 20 and 21 shows about 70 percent cracking, whereas the control section next to tests 16 and 17 has cracked 100 percent. Again, as in the Raynham test results, the hatched area of the bar graphs denotes the amount of wide cracks, whereas the blank area gives the percentages of narrow cracks.

Figure 22 shows a comparison for cracks along the edges of the pavement between the slab and the shoulder. Comparing Figure 21 with Figure 22 it is apparent that th amount of longitudinal cracking in Walpole is very small as compared to the transver cracking.



igure 20. Comparison between the two types of transverse cracking in Raynham: (a) racks right above the underlying joint and (b) cracks at the edge of the strip reinforcement.

TABLE 3

COMPARISONS OF LONGITUDINAL CRACKING BETWEEN ADJACENT SLABS IN THREE TEST SECTIONS IN RAYNHAM

est	Fabric Style	Potential Cracking at Various Ages (%) 1 Yr	2 Yr	3 Yr	4 Yr	5 Yr
3	4 x 4, 10/10	0	0	15	21	31
4	6 x 3, 10/10	0	0	6	6	7
5	6 x 6, 8/8	0	0	0	0	0
	none	0	7	14	22	38

Besides cracks immediately above the old joints, there are also a few cracks in eas where the overlap of the wire fabric sheets or rolls occurs. The relative amount this kind of cracking as compared to the longitudinal and transverse cracks is very nall and the length of cracks for 1,000 sq ft, both for wire fabric in sheets and rolls, given in Tables 4 and 5.

The amount of longitudinal cracking between two adjacent slabs in Walpole has not en shown in any of the comparisons. The reason for this is that many cracks in the nter of the road had been caused by the paint stripes and it was virtually impossible separate these cracks from the joint cracks. It can also be seen from Figure 8 that butting of two adjacent rolls or sheets occurred about 6 in. on one side of the longitinal center joint. Observations in cracking pattern indicate that reinforcement at ch a short distance from a joint leaves the reinforcement ineffective against longitinal joint crack appearances. Therefore, due to the closeness of the reinforcement

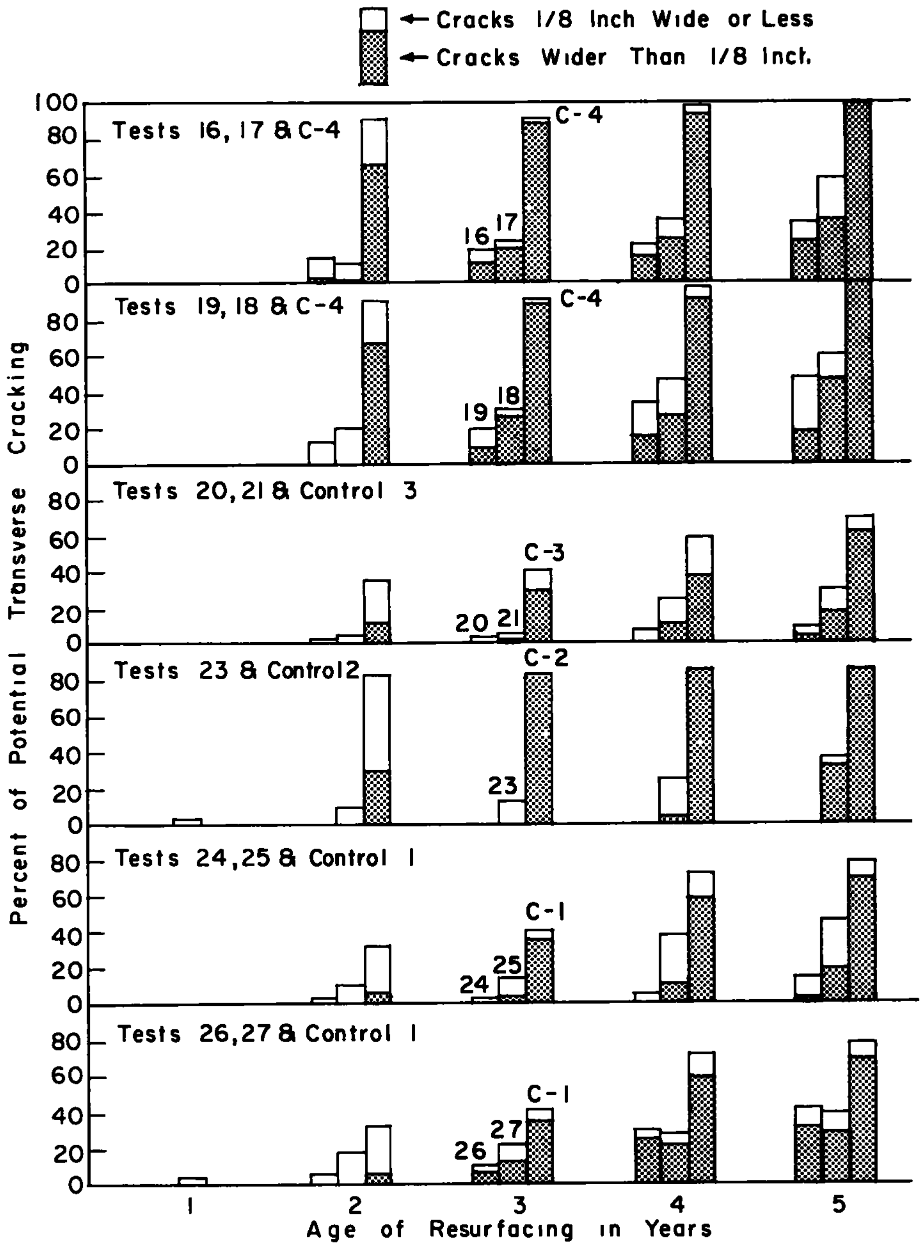
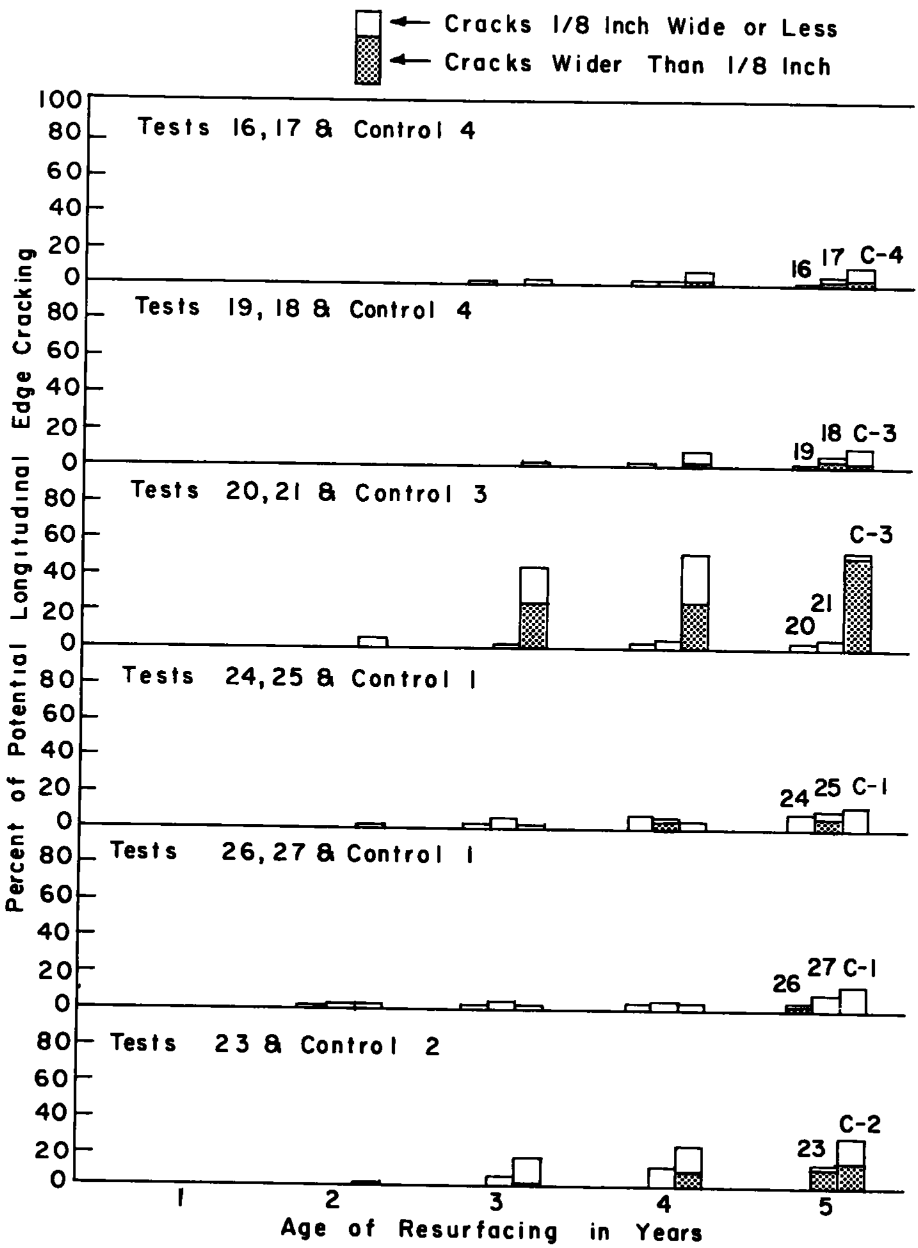


Figure 21. Comparison of transverse crack development for the Walpole Test Road using continuous reinforcement.

butting and the occurrence of paint cracks immediately above the joint, the comparison between the test sections and the control sections in this paper has not been made. The actual observed amount of cracking was similar to the adjacent control sections.

DISCUSSION OF FIELD RESULTS

The main purpose of the two test roads was to compare the effectiveness of various strip reinforcements placed above the transverse joints and continuous reinforcement



ure 22. Comparison of longitudinal edge crack development for the Walpole Test Road.

ich covers the whole road surface. The types and sizes of the wire fabric used in rious sections were selected according to practical considerations and availability the various wire fabric styles. Following is a discussion of the field performance two test roads.

ynham—Strip Reinforcement

The main emphasis in the Raynham test section was placed on the performance of nforcement over transverse joints because the strains are relatively higher at these

TABLE 4

CRACKING OVER LAPS BETWEEN SHEETS AND ROLLS IN WALPOLE TEST ROAD

Test	Cracking (ft/1,000 sq ft) 1 Yr	2 Yr	3 Yr	4 Yr	5 Yr
16	0	2.4	3.1	3.7	5.1
17	0	1.1	1.4	2.2	2.3
18	0	1.2	2.0	3.1	3.1
19	0	1.4	2.0	3.1	3.9
20	0	0	0	0.3	0.3
21	0	0	0.1	0.8	0.8
23	0	0	0	0	0.1
24	0	0	0	0.2	1.8
26	0	0	0	0	0.2
27	0	0	0	0	0

TABLE 5

RANDOM CRACKING OVER INTERIOR OF SLABS IN WALPOLE TEST ROAD

Test	Cracking (ft/1,000 sq ft) 1 Yr	2 Yr	3 Yr	4 Yr	5 Yr
16	0	0.3	0.8	5.1	5.
17	0	1.8	1.4	3.5	3.
18	0	0.8	0.8	0.8	0.
19	0	1.6	1.1	3.3	4.
20	0	0	0	0.2	0.
21	0.2	0.4	0.4	1.2	1.
23	0	0	0.6	1.6	1.
24	0	0.4	0.6	0.7	1.
25	0	0	0	0.3	1.
26	0	0	0.2	0.2	1.
27	0.7	2.8	4.5	4.9	8.
C-1	0	0.2	1.1	2.9	4.
C-2	0	0.4	0.5	1.2	2.
C-3	0	0	0.2	1.3	1.
C-4	0	0.2	1.0	3.2	4.

areas as compared to the longitudinal joints. Figure 19 shows that all test sections and controls had cracks after two years of traffic and exposure to weather. In all cases the cracks increased in width and length with time. The most undesirable cra are the wide ones because they require continued maintenance. Comparing the amou of wide cracks as well as the total cracking in Figure 19 it is apparent that the contr sections in all cases had more of both types of cracking than any of the wire reinforced tes tions. Thus, it can be said that any of the wire fabric reinforcement over the trans- verse joints will help in a degree to prevent reflection cracking. The lowest per- centage of reflection cracks was found in test 10 (fabric style—6 x 6, $^{8}/_{8}$) which had about one-third of that in a comparable control section. If the percentage of the wid cracks is defined as the most important, test 6 (fabric style—3 x 6, $^{10}/_{10}$) is the best. In general, tests 5, 6, 8 and 10 are the best performers with total cracking from ab ne-half to one-third and wide cracks between one-half to one-sixth of that in the conrols.

The two types of transverse cracks encountered in Raynham test section are comared in Figure 20. It is apparent that the type of reinforcement plays a great role in he structural behavior of the bituminous concrete and reinforcement combination. aboratory studies show that the design and placement of the reinforcement for transerse joints is an engineering problem which requires balancing of "strength" to resist he forces of nature and traffic (11). For instance, if the strip reinforcement is made f stiff wire closely spaced and the width of the strip is not adequate, there will be no racking immediately above the joint but the strongly reinforced section will transfer he strain to the edge of the reinforcement causing a crack there. On the other hand, the sizes of the wire fabric are small, the spacing between the wires is considerable, nd the strip of wire fabric placed above the joint is wide, the likelihood of reflection racks occurring immediately above the joint is high. Laboratory tests and theoretical udies indicate (10, 11, 17), that the behavior of welded wire fabric in the bituminous oncrete is different from that of reinforcement placed in portland cement concrete. hen a reinforced bituminous concrete layer is subjected to slow tension the longidinal wires offer very little bond resistance. The main resistance to the strain of e reinforced overlay is exercised by the transverse wires which undergo a shear ovement through the mix. The amount of shear movement depends on stress, temperure of mix, time of loading and other factors. Part of this movement is recoverable astically, whereas the other part becomes a permanent deformation. The main facrs that appear to play a role in deciding whether a crack will appear immediately ove the joint or at the edge of the strip reinforcement are as follows:

1. The percentage of the longitudinal steel area affects the amount of strain that is stributed along the longitudinal wire for a given stress. The thinner the longitudinal ires or the less of them per unit area the easier it will be to "stretch" the fabric.
2. The transverse wire length between two longitudinal wires is of great importance. ne longer the spacing the more flexible is the reinforcement system and the more ely center cracks can occur. This flexibility is influenced also by the transverse re diameter.
3. The shear resistance of the transverse wires which is influenced by the transrse wire diameter is another factor that affects the wire fabric reinforcement bevior. The smaller the transverse wire diameter the easier the wire will cut into e mix or undergo shear movement under the given load and more likely allow cracks develop immediately above the joint.
4. Road tests show that the width of the wire fabric strip reinforcement is imrtant in deciding whether there will be cracking or not and if so where this cracking ll occur. For narrow strip reinforcement the likelihood is an edge crack.

It must be remembered, however, that the wire fabric reinforcement alone is not ing to determine whether the reinforced joint will have a crack or not. The condition the underlying slab, especially surface roughness as well as other factors, will y a role in defining where the crack will form—if it does. By outlining the previousdiscussed reinforcement factors for each of the Raynham test sections, a fairly constent explanation of why certain sections had cracks at the edges, whereas the others l not, can be obtained. For instance, test 12 has a relatively low longitudinal wire el area, the longitudinal wire spacing is 12 in., the transverse wire diameter is atively large but the spacing is 6 in., and the strips are 10 ft wide which is wide cording to the authors' classification. Therefore, it is likely that cracks will delop along the center of the reinforcement or immediately above the joint. On the er hand, test 4 has a high longitudinal steel area, narrow spacing of longitudinal res, and the strip width is only 5 ft. Therefore, if any cracks would appear they e likely to be at the edge of the strip. This is true for both installations (Fig. 20). nilar comparisons for the other test sections will indicate the same trend. Although s difficult at this time to assign definite numbers for the aforementioned factors, it assumed that future research will reach conclusions as to their relative significance. e Raynham Test Road indicates that under Massachusetts conditions and with the styles of strip reinforcement used, reflection cracking can be considerably reduced but not prevented. The main reason for this is the fact that strip reinforcement needs balanced design which is difficult to achieve where slabs are of considerable length an the horizontal joint opening and closing is large. This question has been discussed at greater length in a previous publication (11).

In addition to the 11 tests of transverse joints, three areas were covered with strip reinforcement for longitudinal joints only. The cracking index for these three section are given in Table 3. Because of the relatively short length of these tests any conclusions should be drawn with caution. Test 13 (fabric type 4 x 4, $^{10}/_{10}$) has unusually high crack index which is unexpected and cannot be explained at this time. Test 15 (fabric type 6 x 6, $^{8}/_{8}$) which has the same fabric as test 10, has no cracking.

Walpole—Continuous Reinforcement

The Walpole Test Road contains three test sections with the continuous sheet of reinforcement and seven sections where rolls were used. Figure 21 summarizes the amount of transverse cracking and again it must be emphasized that comparisons can be made only between the tests and the adjacent control sections. Thus tests 20 and 2 should be compared with control 3 rather than with other controls. This calls for a caution when direct comparison between various sections are made.

The basic idea behind the continuous reinforcement is to make the resurfacing independent of the underlying slab movements. In a continuously-reinforced surfacing the possibility of so-called edge cracking (as shown in Raynham) is no longer a problem; and instead of balancing the various forces on both sides of the joint, the design involves optimum reinforcement to obtain the strength necessary to avert any excessi strain in the resurfacing. The results in Figure 21 indicate that the best performanc was obtained with 3 x 6, $^{10}/_{10}$ welded wire fabric in rolls. There are two test sections with this type of wire fabric and both of them show good results (tests 20 and 24). Th difference between the two is the manner in which the wire was layed: in test 20, the longitudinal wires were on the top of the fabric, whereas in test 24 this was reversed The total amount of cracking as compared to the control is about one-seventh for test 20 and about one-fifth for test 24. The number of wide cracks in both test sections is low and very few cracks are noticeable driving over these sections. It can be said that after five years, transverse cracking in these two sections is negligible from a maintenance standpoint.

Tests 16, 17 and 18 were reinforced with various types of fabric in sheets and the performance is less impressive than with similar wire fabrics of a continuous type o reinforcement. The possible reason for this is that even though the sheets were tied together by hog rings there actually was never a continuous-type reinforcement in su an arrangement. In several cases the laps of the joints were very close to the old portland cement concrete joints thus causing a plane of weakness near the joint. In test sections where rolls were used the longitudinal wires in all cases had to be gage 10 or higher (thinner) because of the difficulty in flattening the rolls on the road. As a consequence no heavier gage than 10 was used on a test section. Test 26 had 3 x 3 $^{12}/_{12}$ wire fabric style in rolls and test 27 was 2 x 2, $^{14}/_{14}$ wire fabric. Both sections were difficult to place mainly because of the narrow wire spacing and the $^{7}/_{8}$-in. aggr gate used in the binder course. Test 25 contained no wire but the thickness was increased to 3$^{3}/_{4}$ in. instead of the conventional 3 in. so that the cost would equal that o wire fabric. The transverse cracking in this section has reached 50 percent but is still low compared to the adjacent control.

The percentages of longitudinal cracking between the edge of the slab and the shoulder on each side are given in Figure 19. As can be seen, the relative number of cracks found is small compared to the previously discussed transverse cracking. Even the control sections (except for control 3) have low cracking index. The reinforcement in the bituminous concrete overlay was extended over already existing bituminous concrete shoulders. Therefore, the amount of settlement so far has been very small which might account for the relatively low cracking indexes. There were a few instances where drainage was placed along the edge of the old pavement prior

) resurfacing. The disturbed and then compacted soil and gravel support has settled bout $\frac{1}{4}$ to $\frac{1}{2}$ in. in these locations and edge cracking has taken place. These areas, owever, had to be eliminated from the test compilations as they could not be compared with test sections where such disturbances were not present. The high perentage of cracks in control 3 is not explainable from data available.

In summary it can be said that several types of wire fabrics are capable of restrictng the longitudinal edge reflection crack development for at least five years.

Table 4 summarizes the number of cracks which have developed at the wire fabric aps. Similarly, a few cracks that have developed over the interior of the slabs are iven in Table 5. The highest occurrence of these two types of cracks is found in ests 16, 17 and 18 where the sheet type of reinforcement was used. The length of the rack for each area, however, is very small and of little importance.

aynham Compared to Walpole Tests

The comparison between the two has to be made with some caution because of the elative differences between the test roads and also due to slight differentials in the racking indexes of the control sections. If a "rough" comparison is assumed to be alid, all but two of the wire tests have performed better than the best control. In n over-all comparison a continuous type of reinforcement has performed better than e strip type of reinforcement with 3 x 6, $^{10}/_{10}$ wire fabric in continuous rolls showing e most outstanding performance. The ability of the 3 x 6, $^{10}/_{10}$ welded wire fabric resist formation of cracks above transverse joints is also clearly indicated by the erformance of tests 1, 4 and 6 in Raynham. Although they have cracks at the edges the reinforcement, center cracking is below ten percent; that is, comparable to e 3 x 6, $^{10}/_{10}$ sections in Walpole.

CONCLUSIONS

When comparing these results, the variations in the condition of each test section well as in the physical environment of the test sites over the 5-yr period must be pt in mind. Observations and measurements indicate the following:

1. It is easy to place the types and styles of welded wire fabric used in Raynham and alpole tests in 3 in. of bituminous resurfacing when the fabric is placed directly on e surface of the old concrete pavement.
2. All types and styles of fabric used had the effect of reducing the amount of reection cracking to less than that in control sections.
3. Where cracks did appear at reinforced transverse joints, they were of smaller erage width than those in the comparable control sections.
4. The most outstanding resistance to reflection cracking was shown by 3 x 6, $^{10}/_{10}$ ntinuous welded wire fabric reinforcement in rolls (test 20). For example:
   - (a) Only nine percent of the total length of potential transverse reflection cracks appeared. This is nearly one-eighth that in the comparable control section.
   - (b) Only four percent of the cracks are wide (more than $\frac{1}{8}$ in.), which is about one-fifteenth of the wide cracks found in the control section.
   - (c) Only about two percent of possible longitudinal edge cracking occurred with only narrow cracks present.
5. In the strip reinforcement test a strip of 6 x 6, $^{8}/_{8}$ welded wire fabric 7.5 ft wide oved the best. Total cracking was only one-third (33 percent) of that in the control. st 6 using 3 x 6, $^{10}/_{10}$ fabric showed the lowest percentage of wide cracks; that is, out one-seventh than that of the control.
6. The performance of welded wire strip reinforcement is evidently influenced by e longitudinal steel area, longitudinal wire spacing, transverse wire diameter, and dth of the strip reinforcement. Strip reinforcement requires a balanced design (not) strong and not too weak) where relatively large horizontal joint openings are excted (about 0.1 in. or more).
7. While the choice of width of welded wire fabric reinforcement strips is a function several factors, under the conditions tested 6- to 8-ft wide reinforcement for

transverse joints and about 5 ft for longitudinal joints appears satisfactory.

8. The amount of longitudinal edge cracking was small in the Walpole Test Road in all reinforced sections.

9. If settlement of the resurfacing over a shoulder is more than about $\frac{1}{4}$ in., longi tudinal cracks will occur between the slab and the shoulder even with reinforcement.

10. The $3\frac{3}{4}$-in. thick resurfacing in test 25, costing the same as a continuously-reinforced surfacing 3 in. thick, had a transverse crack incidence five times greater than in test 20 where 3 x 6, $\frac{10}{10}$ welded wire fabric reinforcement was used in the 3-in thickness.

11. Field observations show that $\frac{7}{8}$-in. maximum size aggregate appears to be the upper size limit if the reinforcement opening is 2 x 2 in. In other words, the minimu fabric wire spacing in the map should be at least twice and preferably three times the maximum dimension of the aggregate used in the mix.

RECOMMENDATIONS

The work described in this paper has been aimed at systematic gathering of data concerning welded wire fabric reinforcement in bituminous concrete resurfacings so that meaningful conclusions could be reached. This should be continued in order to find solutions for varied situations:

1. The amount of horizontal and vertical joint movements should be measured and data compiled in each state.
2. In areas where the conditions are similar to Massachusetts large installations using the best types of continuous reinforcement should be undertaken.
3. Strip reinforcement should be tried in areas where the maximum horizontal joint movement is not as great as in Massachusetts and where the main purpose is to reinforce against shear and flexure at the joints.
4. Some samples removed from the test road showed evidence of rusting of the wire fabric. Although this does not appear to have influenced the performance of the reinforcement, ways of protecting the steel should be investigated, such as sandwichi it between layers of bituminous concrete.
5. Improvements in methods of laying fabric including the development of equipment for this purpose should be studied.
6. Theoretical studies of welded wire fabric reinforcement in a viscoelastic material (bituminous concrete) should be pursued.
7. Laboratory studies to prove theories and to accumulate knowledge on the importance of factors like longitudinal wire cross-sectional area, spacing, transverse wire size and strip width should be continued.
8. Rheological studies of various types of bituminous concrete reinforced with welded wire fabric should be continued in the laboratory.

ACKNOWLEDGMENTS

The cooperation and help of many organizations and individuals was necessary to make the first phase of the project a success.

The Committee on Welded Wire Fabric Reinforcement Research of the American Iron and Steel Institute sponsored the research.

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The Pittsburgh Steel Products Company, American Steel and Wire Division of Uni States Steel Corporation, and Wickwire Spencer Steel Division of The Colorado Fuel nd Iron Corporation manufactured the fabric, much of which was in small lots of pecial styles.

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The Norfolk Construction Company and Thomas Brothers Corporation were the contruction contractors.

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Appendix

SPECIFICATIONS FOR MASSACHUSETTS TYPE I BITUMINOUS CONCRETE MIX

	Standard Sieves		Percent by Weight			
			Standard Bottom Course		Standard Top Course	
Material	Passing	Retained	Min	Max	Min	Max
Coarse aggregate[a]	-	7/8"	-	0	-	-
	7/8"	1/2"	30	50[1]	-	0
	1/2"	No. 4	15	30	25	40[1]
	No. 4	No. 10	5	15	15	25
Fine aggregate and mineral filler	No. 10	No. 20	2	8	4	12
	No. 20	No. 40	4	10	6	16
	No. 40	No. 80	4	10	6	16
	No. 80	No. 200	2	6	4	10
	No. 200	-	1	4	4	6
Bitumen (sol in CS_2)			$4\frac{1}{2}$	$5\frac{1}{2}$	6	7
Total				100		100
Total fine agg. and min. filler			20	30	35	45

[a]Not more than 1/5 of the 7/8- to 1/2-in. fraction in the bottom course shall be retained on a 3/4-in. sieve. Not more than 1/4 of the 1/2 in. to No. 4 fraction in the top course or dense mix shall be retained on a 3/8-in. sieve.

Effect of Pavement Breaker Rolling on Crack Reflectance in Bituminous Overlays

PAUL G. VELZ, Research Engineer, Minnesota Department of Highways, St. Paul

THIS STUDY was conducted on S. P. 6511-10 and S. P. 6512-01 (T. H. 212) located etween Bird Island and Stewart. The project was a typical widening and bituminous esurfacing project, in which the bituminous mixtures were placed directly on the old oncrete pavement.

The 1931 concrete pavement, like a number of the older pavements in Minnesota, ad warped panels, cracks, and faulted joints to the extent that the riding qualities had ecome somewhat objectionable, especially for trucks. Experience on other bituminous esurfacing projects indicated that many of these objectionable features will eventually e reflected in the bituminous overlay surface. Usually the joints and cracks in the d pavement cause cracks in the new surface within a very short time. Then longi- dinal cracks appear at the edges of the old pavement and many times at centerline; nd, ultimately, slab movements cause recurrence of general roughness. Maintenance osts go up, and the serviceability of the surface is reduced.

In the past, one solution has been to provide lifts of granular material over rough d pavements before placing the bituminous surface. The added thickness retarded e reflectance of cracks and roughness; and, when thick enough, lifts actually elimina- d most of the effects of the old pavement defects. However, this type of construc- on was costly because of the large quantities of granular materials needed and because the additional grade widening usually required. It also might be considered extrava- nt, in that the full potential of the old pavement as a base course was not used and be- use it consumed such large volumes of good base aggregate—an undesirable feature any case, but especially so in areas of gravel scarcity.

A different solution to the problem of crack and roughness reflection from old pave- ents; namely, pavement breaker rolling, was tried experimentally on this project. he experiment was limited to a 1½-mi section (Sta. 950 to 1025) located about 1 mi est of Stewart. This section was rolled with a 59-ton roller to break the old pave- ent, and was constructed to three different design sections. The variables were: -in. leveling course, 3-in. bituminous base and 6-in. bituminous base, as compared a 1½-in. leveling course on the rest of the project. These variables were coupled th a 1½-in. binder course, a 1½-in. wearing course and a standard widening section complete the reconstruction.

To assist in the study, four comparison sections were selected as follows: Sta. 0 to 130, one of the roughest portions of the old pavement; Sta. 800 to 810, an area typical roughness; Sta. 925 to 950, immediately adjacent to the beginning of the ex- rimental section; and Sta. 1025 to 1033, immediately adjacent to the end of the ex- rimental section. (The comparative value of this last section was partially lost be- use rolling was extended to Sta. 1029.)

This report includes the results of the evaluation studies made during construction 1959 and subsequently to April 1960, a period of six months after completion of the tuminous surfacing. Information is included on such items as the design of the old vement, the immediate effects of the rolling on the old pavement, typical sections, sts of the reconstruction and the performance of the project to date.

SUMMARY OF FINDINGS

The project is not old enough to draw positive conclusions regarding the effects of e pavement breaker rolling and the performance of the various design sections. wever, a number of interesting facts and observations are disclosed by the study to date. These findings, discussed in detail in this paper, are summarized as follows

Pavement Breaker Rolling

1. Contact tire pressure was 83.5 psi based on the gross contact area of the 18.00 x 25, 24-ply tires.
2. Vertical slab movements at joints were variable, ranging from $\frac{1}{8}$ to $\frac{1}{2}$ in. and averaging about $\frac{1}{4}$ to $\frac{3}{8}$ in.
3. Ten passes of the 59-ton roller provided optimum cracking on this project. An additional ten passes increased the number of cracks only slightly.
4. After rolling, all cracks (new plus old) averaged 16.4 per station in each lane. When the joints are included with the cracks, the total openings averaged 19.1 per station in each lane—an average spacing of 5.2 ft.
5. Cracking in 20-ft panels was comparable to cracking in 40-ft panels; however, the 20-ft panels had 4.9 more openings per station, mostly on account of more joints.
6. Pavement breaker rolling caused only minor permanent changes in the profile of the concrete pavement. Generally, the pavement was permanently depressed from 0.01 to 0.05 ft over most of its length. Some portions of the pavement were unchange some were raised slightly and some were depressed greater amounts, up to 0.13 ft (about $1\frac{1}{2}$ in.).
7. Pavement breaker rolling caused only a slight decrease in roughness in one lan (160 to 154 in. per mile) and no change in the other lane (154 in. per mile) as measur by the road roughness recorder.
8. Pavement breaker rolling cost $271.44 per mile for 10 passes of the roller ove the 20-ft pavement.

Typical Sections and Costs

9. Resurfacing sections included a $1\frac{1}{2}$-in. wearing course, a $1\frac{1}{2}$-in. binder cours and widening plus the following variables, all at the total indicated costs per mile:

$1\frac{1}{2}$-in. leveling course (std. sec. for project)	$ 38,340
2-in. leveling course (sta. 1000 to 1025)	39,908
3-in. bituminous base (sta. 950 to 975)	40,427
6-in. bituminous base (sta. 975 to 1000)	45,001

The latter three sections include pavement breaker rolling.

Performance

10. Sections subjected to pavement breaker rolling have had less cracking in the bituminous surface to date than unrolled sections. Where rolled, 5 to 42 percent of the transverse joints and two cracks were reflected as compared to 48 to 100 percent of the transverse joints and one crack reflected where not rolled.
11. Reflectance cracking occurred over the 1-in. expansion joints sooner than ove contraction joints on both the rolled and unrolled sections.
12. No cracking over the edge of the old pavement has occurred on the rolled sec- tions, whereas from 2 ft to 364 ft has occurred on unrolled comparison sections.
13. Cracking over the centerline of the old pavement has occurred on all sections except the 6-in. bituminous base section (rolled) which showed very little cracking of any kind. Some of this centerline cracking may have been associated with frost actio
14. Frost heaving occurred on all the sections, with the maximum measured heav being 2.4 in.
15. Differential heaving, though slight in many cases, was noticeable at a conside able number of cracks. Where every joint was cracked, such as between Sta. 120 an Sta. 130, the heaving at cracks caused a slight warped panel effect.
16. Roughness on the rolled sections was 50 in. per mile after construction and 56 in. per mile in April 1960. Project averages were 56 and 60 in. per mile at these same times.
17. No rutting or displacement of the bituminous mixtures in the wheel tracks has occurred on this project to date.

18. It appears from all the data, that pavement breaker rolling has had a beneficial ffect in retarding reflectance cracking during the first six-months performance of this roject.

OLD PAVEMENT

The old concrete pavement was constructed in 1931. The slab was 20 ft wide, 9 in. iick at the edges and tapered to 7 in. thick 4 ft from the edges, a typical 9-7-9 section. 'anels were generally 40 ft 4 in. long, with every other joint being an expansion joint. 'he 1-in. expansion joints had $^{3}/_{4}$-in. dowels with steel sockets on one end. The dum- iy-type contraction joints had $^{3}/_{4}$-in. dowels which were greased on one end. Slab einforcement consisted of two $^{5}/_{8}$-in. bars along each edge and one along each side of enterline, one $^{5}/_{8}$-in. bar along each side of the joints, and $^{1}/_{2}$-in. by 4-ft tie bars cross centerline. The pavement design is shown in more detail in Figure 1.

In minor portions of the project, the pavement was modified to 20-ft panels. Some these shorter panels occur in the experimental and comparison sections.

Just prior to construction, in April of 1959, the road roughness recorder showed an erage roughness of 138 in. per mile, with 163 in. being the roughest mile recorded id 121 in. being the smoothest mile recorded. This roughness, combined with the arped panels, caused very unsatisfactory riding qualities on a considerable length of e project.

PAVEMENT BREAKER ROLLING

The pavement breaker rolling was performed July 23 and 24, 1959 on the $1^{1}/_{2}$-mi perimental section near the east end of this project, Sta. 950 to Sta. 1025. The ecial Provisions required that each 10-ft lane be covered by 10 passes of a 59-ton ller having four wheels on one transverse axle and tire air pressure of 90 psi. These ovisions were followed, except that 20 passes of the roller were made in the west- und lane between Sta. 949 and 964+13, and with the further exception that the rolling as extended beyond Sta. 1025 to an entrance at Sta. 1029 to facilitate turning the roller.

ire Contact Pressure

The roller was a Bros Compactor loaded to 118,000 lb (59 tons) and fitted with four,

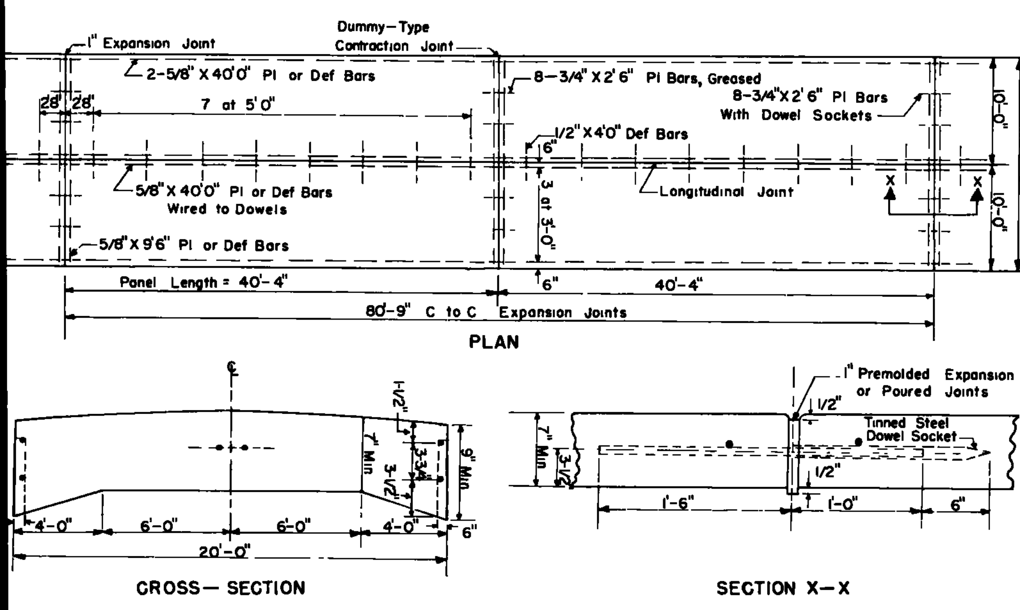
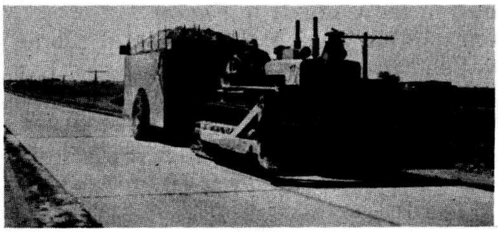


Figure 1. Concrete pavement details.

18.00 x 25, 24-ply, diamond tread tires inflated to 90-psi air pressure. The rolling width, measured from outside to outside of tire contact, was 8 ft 8 in. An Internation TD 24 tractor was used to pull the roller and did an excellent job of controlling it at all times. The roller-tractor combination is shown in Figure 2.

To measure the tire contact areas, a length of 30-in. wide paper was placed across a clean portion of pavement slab, and the roller was pulled forward until the tires wer on the paper. Then using pressurized cans, paint was sprayed completely around the contact periphery of each tire, being sure to cover the tire and the paper at all places. When the roller was moved ahead, the four contact areas were outlined on the paper a shown in Figure 3. Later, these were measured as follows:



Tire	Gross Contact Area (sq in.)
Left outside	354.3
Left inside	344.7
Right inside	384.8[a]
Right outside	329.3[b]
Total	1,413.1 sq in.

[a]Noticeably larger. [b]Noticeably smaller.

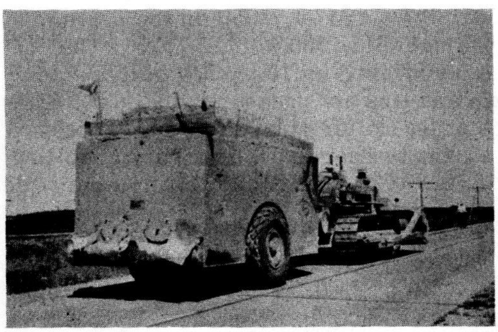


Figure 2. Fifty-nine-ton pavement breaker roller.

Using this total area the average conta pressure was computed as 83.5 psi.

Vertical Slab Movements

From visual observations, it was note that there was considerable variation in slab movement at joints from panel to panel and place to place. It seemed that the slab ends at expansion joints moved more when the roller passed than they di at contraction joints. This seemed logic considering that some aggregate interloc was still effective in resisting vertical movement at the contraction joints. How ever, the few actual measurements taken do not bear this out.

Four joints were checked for vertical movement as follows:

		Vertical Movement (in ft)			
		At Centerline		At Left Edge	
Station	Joint Type	Panel 1	Panel 2	Panel 1	Panel 2
959+00	Expansion	0.015	0.030	0.025	0.030
959+41	Contraction	0.010	0.030	0.025	0.025
959+82	Expansion	0.030	0.015	0.040	0.020
960+22	Contraction	0.020	0.025	0.025	0.010

These measurements seem typical of the movements observed—varying from abou $\frac{1}{8}$ to $\frac{1}{2}$ in. with most approximately $\frac{1}{4}$ to $\frac{3}{8}$ in. However, the measured movements were not significantly larger at the expansion joints.

Cracking

Visual cracking of the pavement slab did not occur until after several passes of th

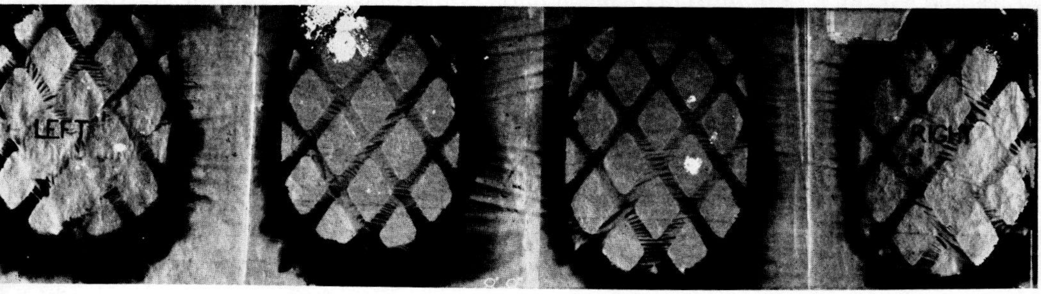


Figure 3. Tire contact areas, 59-ton pavement breaker roller.

oller, and cracks became more visible as rolling continued. When they first occurred, racks were extremely fine and difficult to observe. As rolling progressed, the top dges spalled slightly, and the cracks became more visible. It was also observed that raffic caused a similar spalling of the cracks. Much of this spalling was very minute, eing just enough to show whitish dots along the path of the crack, although a few spalls ltimately were an inch or more in diameter. Practically all cracks, new and old, vere transverse cracks with the exception of some diagonal cracks in frost heave areas.

Prior to rolling, there were 368 cracks in the left (W.B.) lane and 339 cracks in ne right (E.B.) lane. Rolling caused 933 and 935 new cracks in the respective lanes. Vhen 420 half-width joints were added to the cracks, there were a total of 2,995 open-ngs in the two lanes.

Converting these figures to cracks or openings per station, the following comparisons an be made:

	Left	Right	Both Lanes
ld cracks per Sta.	4.7	4.3	9.0
ew cracks per Sta.	11.9	11.9	23.8
otal cracks per Sta.	16.6	16.3	32.9
otal openings per Sta. (including joints)	19.3	18.9	38.2
verage spacing between cracks and joints (ft)	5.2	5.3	-

Between Sta. 949+00 and Sta. 964+13, there were 10 passes of the roller in the ght lane and 20 passes of the roller in the left lane. The difference in cracking was s follows:

	Left	Right
	20 Passes	10 Passes
umber of old cracks	77	67
umber of new cracks	207	184
otal No. of cracks	284	251
otal No. of openings (including joints)	322	289
ld cracks per Sta.	5.1	4.4
ew cracks per Sta.	13.7	12.2
otal cracks per Sta.	18.8	16.6
otal openings per Sta. (including joints)	21.3	19.1

The 20 passes on the left lane caused only 23 more new cracks than the 10 passes on the right lane. This small increase in cracking indicates that, for this project, 10 passes of the roller were sufficient to develop the optimum amount of cracking.

On this project, the standard joint spacing was 40 ft. However, there was a 580-ft section of 20-ft panels in the rolled area. Therefore, a comparison of cracking can b made between these two joint spacings as follows:

	Cracks in Both Lanes	
	20-ft Panels	40-ft Panel
Old cracks per Sta.	10.9	8.9
New cracks per Sta.	22.2	24.0
Total cracks per Sta.	33.1	32.9
Total openings per Sta. (including joints)	42.8	37.9

The new cracking caused by the rolling was slightly greater for the 40-ft panels, though not a significant amount when you consider that there were more old cracks in the 20-ft panels. Combining the new and old, the 20-ft panels had slightly more total cracks after rolling. When the joints are added, the 20-ft panel section averages 4.9 openings more per station that the 40-ft panel section, a significant difference.

Permanent Slab Deflections and Roughness

Profiles were taken 8 ft right and left of centerline on the old concrete pavement before and after the pavement breaker rolling at three locations. The results were generally as follows:

Sta. 950 to 955.—There were only minor changes in the profile of the old pavemen In a few places, the pavement was higher by 0.01 to 0.04 ft after rolling. In other places, there was no measurable change in elevation. However, in a majority of plac the rolling depressed the old slab from 0.01 to 0.06 ft.

Sta. 980 to 990.—Through most of this section the profile after rolling was within 0.02 ft of the original profile. Generally, the rolling depressed the slab, but not at all places. The old pavement was depressed more than 0.02 ft only in isolated areas

Sta. 1010 to 1025.—Rolling caused a somewhat more consistent lowering of the pa ment by 0.02 to 0.05 ft through this section. However, there were areas of little or no change and areas where the slab was slightly higher after rolling. At one point, at a crack, the slab was depressed 0.13 ft. This was the maximum permanent defle tion caused by rolling as measured by the profiles.

Roughometer readings before and after rolling show a slight decrease in average roughness in the westbound lane and no change in the eastbound lane. The westbound lane averaged 160 in. per mile in April prior to rolling and 154 in. per mile in July after rolling. The eastbound lane averaged 154 in. per mile at both times.

RESURFACING SECTIONS AND COSTS

The typical resurfacing section for the project consisted of a 1½-in. leveling cour a 1½-in. binder course and a 1½-in. wearing course over the old concrete pavement This section was varied over the rolled experimental portion of the project to provid for a 2-in. leveling course, a 3-in. bituminous base and a 6-in. bituminous base as alternates to the 1½-in. leveling course on the standard section. These features plu the widening designs are shown in detail in Figure 4.

Costs of the various sections constructed on this project were computed on the ba of average job data. All items pertaining to the construction from shoulder to should including the excavation for the widening, were taken into account. Items such as cu vert extensions, subgrade treatments, sodding and seeding were not included. On th basis, the costs of the various sections are given in Table 1. The cost of the standa section (1½-in. leveling course) was $38,340 per mile. The costs of the other secti

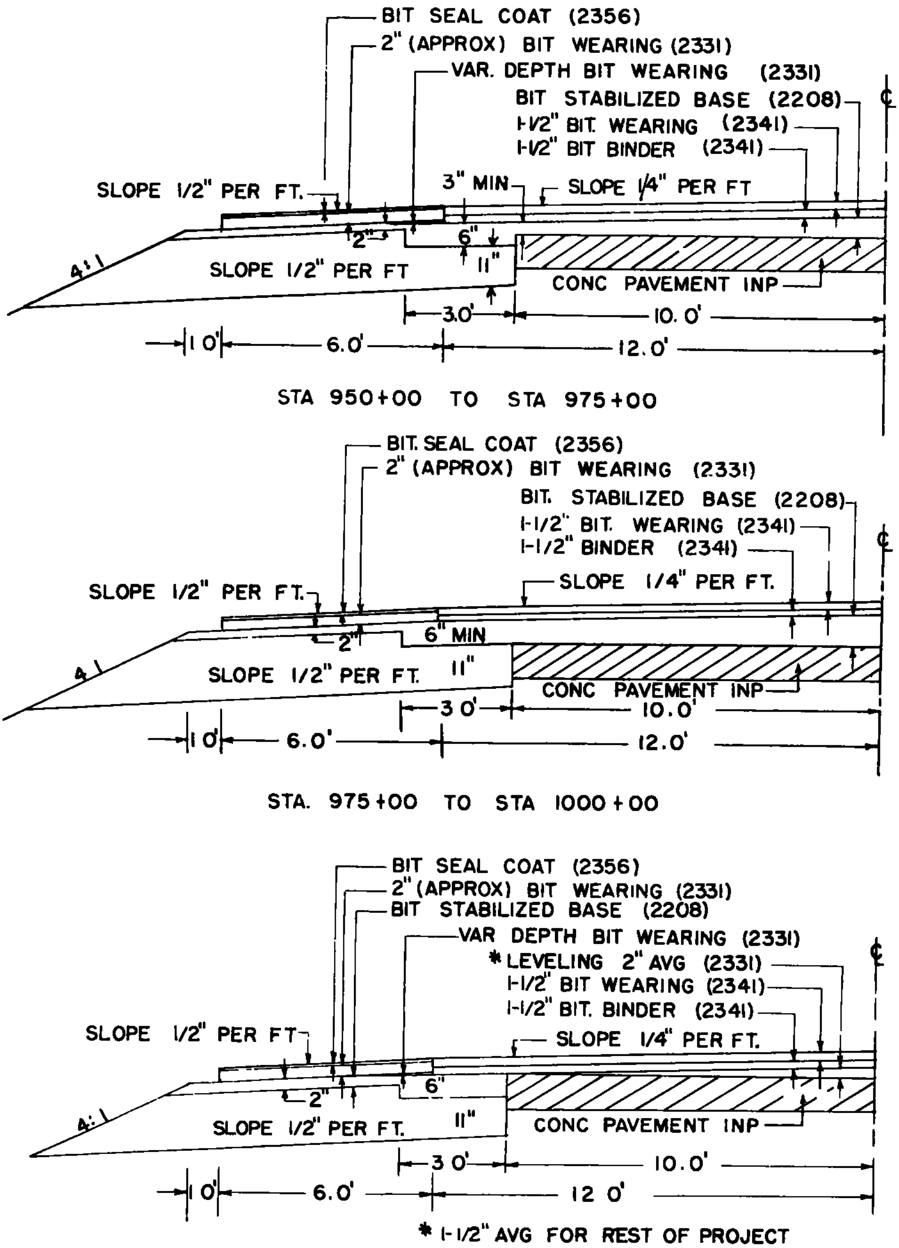


Figure 4. Typical sections.

re increasingly greater as the thickness of bituminous mixture was increased: the n. leveling course section being $39,908; the 3-in. base section being $40,427; and 6-in. base section being $45,001. These latter costs include $271.44 per mile for pavement breaker rolling.

PERFORMANCE OF THE PROJECT

The performance of the bituminous resurfacing has been evaluated and compared in

TABLE 1

RELATIVE COSTS OF RESURFACING SECTIONS

Station	Wearing Course (in.)	Binder Course (in.)	Leveling Course (in.)	Bituminous Base (in.)	Pvt. Breaker Rolling[a]	Cost per Mile[b] ($)	Date Surface Complete
950-975	1.5	1.5	-	3.0	Yes	40,427	9/28/59
975-1000	1.5	1.5	-	6.0	Yes	45,001	9/28/59
1000-1025	1.5	1.5	Av. 2.0	-	Yes	39,908	9/28/59
Rem. of Project	1.5	1.5	Av. 1.5	-	No	38,340	10/9/59[c]

[a]$271.44 per mile for 10 passes.
[b]Complete section including widening.
[c]Based on Sta. 925 to 950 as a comparable area.

several ways. Crack surveys have been made before rolling, after rolling, and seve times after completion of the surfacing. Profiles were taken before and after rolling after completion of the surfacing, and in February 1960, to show the effects of frost heaving. Detailed cross-sections of the bituminous surface have been taken to detect rutting in the wheel tracks. And, as previously mentioned, several comparison sections were selected with which the performance of the experimental sections could be compared and evaluated.

The information on cracking, profiles, and roughness before and after rolling have already been discussed in relation to the rolling. This data will not be repeated here but it must be pointed out that the condition of the pavement after rolling is the condition which will influence the performance of the surfacing in the experimental section

Reflectance Cracking

The results of crack surveys on each of the sections, comparison and experimenta are discussed separately, covering the sequence and amount of cracking. This information is also summarized in Table 2, where it will be noticed that the rolled sections have less cracking than the comparison sections. The rolled section between St 975 and Sta. 1000 with the 6-in. bituminous base is the least cracked.

Sta. 120 to 130.—This section is located 2.5 mi east of Bird Island and was selected for study because it represented one of the roughest portions of old pavement o the project. The average roughometer reading was 166 in. per mile on this section.

The surfacing was completed during July, and, according to reports from field personnel, cracking started a day or two after the mat was laid. Earlier cracking ha also been reported in the binder course before the wearing course was placed. It is probable that this cracking—which occurred over much of the project completed durin July and early August—was caused by excessive joint movements due to high tempera tures during this period.

In September, when the first condition survey was made on this section, 100 perce of the transverse joints were reflected as cracks in the new surface, and there were

TABLE 2

SUMMARY OF REFLECTANCE CRACKING

					Condition Surveys																		
							Cracks																
					No. Sts.		11/19/59				2/9/60				4/14/60				Length (ft)		
							No.		%		No.		%		No.		%				
Station	Treatment	Pvt. Rolling	Const. Cost ($/mi)	Date Surf. Compl.	L	R	L	R	L	R	L	R	L	R	L	R	L	R	Left Edge	Right Edge	Cent line
120-130	Std. Sec.	No	38,340	July	35	35	34	35	97	100	34	35	97	100	34	35	97	100	230	134	446
800-810	Std. Sec.	No	38,340	Oct. 1	25	25	2	7	8	28	12	12	48	48	12	12	48	48	0	2	20
925-950	Std. Sec.	No	38,340	Oct. 9	63	63	25	6	40	10	32	30	51	48	34[a]	30	52	48	38	22	10
950-975	3-in. Bit. Base	Yes	40,427	Sept. 28	59	59	0	0	0	0	15	17	25	29	25[a]	27[b]	41	42	0	0	80
975-1000	6-in. Bit. Base	Yes	45,001	Sept. 28	64	64	0	0	0	0	0	0	0	0	3	4	5	6	0	0	0
1000-1025	2-in. Bit. Level	Yes	39,908	Sept. 28	73	73	0	0	0	0	9	7	12	10	19	21	26	29	0	0	30

[a]Includes one crack over crack in old pavement.
[b]Includes two cracks over cracks in old pavement.

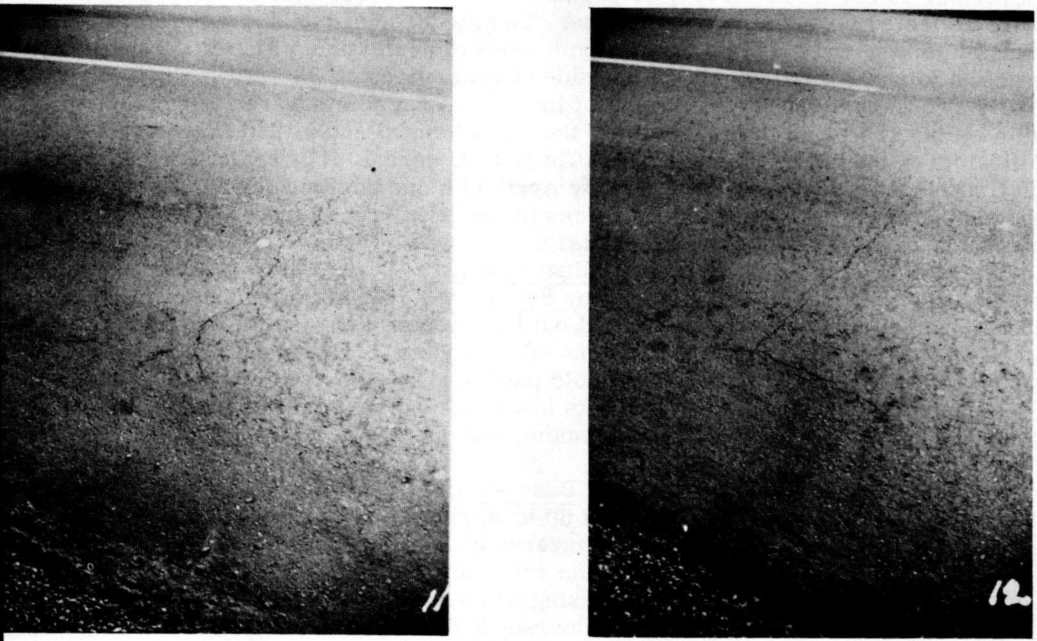


Figure 5. Typical transverse and widening cracks, Sta. 120 to 130.

er 200 ft of longitudinal cracking on centerline. Widening cracks along the edge of e old slab extended up to 5 ft from many of the transverse cracks. Figure 5 shows pical examples of this cracking.

By November 19 two transverse cracks had faulted significantly, 20 percent of the nsverse cracks had extended across the widening, and the centerline and widening ngitudinal cracks had extended slightly. Cracking progressed so that on February 9, 60, 57 percent of the cracks extended across the widening. This increased to 66 rcent by April 14th. All transverse cracks were over the pavement joints except e which occurred at a construction joint in the bituminous surface in the left lane. wever, by April 14 the longitudinal cracking had progressed to the point where there re 446 ft on centerline, 230 ft over the left edge and 134 ft over the right edge of the l pavement. This section has cracked more than any of the other sections studied detail.

Sta. 800-810.—This section is located about 1 mi east of Buffalo Lake and repre- nts old pavement of average roughness, 150 in. per mile. The binder course was d on September 15, and the wearing course was completed on October 6, 1959. On ptember 22 there were no cracks in the binder course. However, on November 19 e-third of the transverse joints were showing as cracks in the wearing course. Some these cracks were only on one-half of the road, and none extended into the widening. s quite certain, although not verified, that all of these cracks are over expansion nts.

By February 9, 1960, 48 percent of the joints had reflected as cracks in the bi- minous surface. All of these were the full width of the old pavement and many ex- ded into the widening. There was no change in the number of cracks up to April h except that nearly all the transverse cracks had progressed into the widening. ere were only four instances where the cracks did not extend into the widening on one e.

Sta. 925 to 950.—This section is located adjacent to the beginning of the experimental lled) section and was selected for comparison purposes. The binder course was npleted on September 17, and the wearing course on October 9. At the time of the

November 19 condition survey, 39 percent of the transverse joints in the old pavemen were reflected through the new surface. Twenty-three of the 25 cracks were over ex- pansion joints, and only six were on both sides of centerline. The two cracks over co traction joints were also only on one side of centerline.

By February 9, 1960, 51 percent of the transverse joints were reflected as cracks with all but two being the full width of the old pavement and all but three extending the full width of the bituminous surface. There also were 10 ft of centerline cracking and 60 ft of edge cracking at this time. By April 14th one additional joint and one crack i the old pavement had reflected as cracks in the left (W.B.) lane. Otherwise, there w no change from the condition in February.

Sta. 950 to 975 (3-In. Bituminous Base—Rolled). —There were no cracks in this section on November 19, 1959; but, by February 9, 1960, 29 percent of the transver joints had reflected as cracks, and all but five extended across the widening. There were 80 ft of centerline cracking and no edge cracking. One short crack in the bitum nous surface was over a crack in the old pavement.

By April 14, 46 percent of the joints had cracked, one more crack appeared which probably was over a crack in the pavement, and there was no change in the centerline or edge cracking.

Sta. 975 to 1000 (6-In. Bituminous Base—Rolled). —This rolled section with the 6- bituminous base has cracked the least up to April 14, 1960. No cracking had occurre up to February 9th, and only four transverse cracks occurred over joints by April 14 One of these was only on one-half of the road, and none of the cracks extended into th widening. No centerline or edge cracking had occurred up to this date.

Sta. 1000 to 1025 (2-In. Leveling Course—Rolled). —At the time of the February 9, 1960 condition survey, 13 percent of the transverse joints had reflected as cracks in the bituminous surface. Of the 10 cracks occurring, six covered the width of the con crete slab, and only 3 extended into the widening. Approximately 30 ft of centerline cracking had occurred in a frost heave area and there was no edge cracking.

By April 14, 28 percent of the transverse joints and one crack had reflected throu the bituminous surface. Only four cracks had extended into the widening on one or b sides. There was no longitudinal cracking over the edges of the old slab, and there no increase in the longitudinal cracking on centerline since February.

Profiles and Frost Action

On all experimental and comparison sections profiles were taken 8 ft right and 8 ft left of centerline for the purpose of evaluating roughness and distortion of the new bituminous surface with age. The first profiles of the new surface were taken shortl after placement. The second set were taken on February 18 and 24, 1960, to show t effects of frost. It is planned to take more profiles to follow the progression of dis- tortion.

The February profiles showed that frost heaving was occurring in most of the are The maximum measured heaving amounted to 0.20 ft or 2.4 in. Most of the heaving was quite uniform, however, differential heaving at some of the transverse cracks o joints occurred to various degrees. The conditions on each section were as follows:

Sta. 120 to Sta. 130 (Comparison Section). —Heaving was more pronounced on the portion between Sta. 124+60 and Sta. 130 than on the westerly 460 ft of this section. However, nearly every cracked joint was raised by heaving, particularly where pane were 40 ft long rather than 20 ft. These raised joints were as much as 0.06 ft highe than the center of the 40-ft panels. Where the panels were 20 ft long, the differentia was on the order of 0.02 to 0.04 ft. Because practically every joint was cracked on section, the differential heaving at the joints was creating a slight although noticeabl reflection of the severe warping that existed in the old pavement prior to resurfacing

Sta. 800 to Sta. 810 (Comparison Section). —The heaving on this section was gene ly slight except for a few spots where it approached 0.05 to 0.08 ft. Here again, the cracks over joints showed a slight differential heave in many cases, but not to the de gree found on the previously discussed section. Cracks were further apart so the warped panel effect was not noticeable at this time.

Sta. 945 to Sta. 950 (Comparison Section). —The heaving on this section averaged om 0.02 to 0.06 ft except for an old heave area at Sta. 948+10 where the maximum ave amounted to 0.16 ft. This heaving has caused the reflection of diagonal cracking the new surface. Heaving at cracks over joints was not noticeable in this area.

Sta. 950 to Sta. 955 (Rolled Section—3-In. Bituminous Base). —Heaving was general- 0.02 to 0.06 ft through this area with no major differentials. Four out of six cracks er joints showed slight differential heaving, but this was only minor at this time.

Sta. 980 to Sta. 990 (Rolled Section—6-In. Bituminous Base). —Between Sta. 980 d 987+40 heaving was generally less than 0.04 ft and quite uniform. There were no acks in this section in February. From Sta. 987+40 almost to Sta. 990, heaving was ore pronounced and not as uniform. The maximum rise was about 0.12 ft. Diagonal acking in the old pavement indicated this to be a potential heave area. One transverse ack over a joint was found in this area in April 1960, and could have resulted from e frost effects.

Sta. 1010 to 1025 (Rolled Section—2-In. Leveling). —Between Sta. 1010 and Sta. 19+80 the heaving was minor ranging from practically nothing to 0.04 ft. There was e crack over a joint in this area which was raised slightly. From Sta. 1019+80 to a. 1025 the profiles show an area of differential heaving which was also indicated by agonal cracks in the old pavement. This heaving ranged up to 0.20 ft. In the area of eatest heave (Sta. 1023+80), cracking was found on centerline in February. Oddly, transverse cracking was found in this heave area in April as might have been ex- cted.

ughness

Roughness measurements were made with the road roughness recorder shortly after e surface was completed in October 1959, and again in April 1960. The average re- lts obtained on the entire project and on the various sections are given in Table 3.

TABLE 3

SUMMARY OF ROAD ROUGHNESS READINGS

Section	Roughness (in./mi) October 1959	Roughness (in./mi) April 1960
oject average	56	60
a. 120 to 130	54	62
a. 800 to 810	53	58
a. 950 to 1025[a]	50	56

avement breaker rolling on this section.

These figures show an average increase of 4 in. per mile for the project as a whole om October to April. The individual sections show slightly more increase in rough- ss than the average, but the differences are still so small that they are not very gnificant.

rface Cross-Sections

Cross-sections of the bituminous surface were taken to determine distortion of the own in the wheel tracks. Such distortion (wheel tracking) has occurred on other pro- cts due to vertical and lateral displacement or movement of the bituminous mixture. cause various thicknesses of mixtures were used on this project, these cross-sections re taken and will continue to be taken as a measure of the stability of the mix under pical highway traffic.

The cross-sections were taken at established locations on the various sections three nes, October 1959, and February and April 1960. To date, no rutting was found to exist. This was more or less expected, because the weather has been cold during thi time, and most displacement of the bituminous mixture would occur during warm wea ther. The plots of the February cross-sections did show the frost heaving. These cross-sections were taken with a leveled straight-edge and scale and were measured to 0.01 in.

ACKNOWLEDGMENTS

The Materials and Research Section of the Minnesota Department of Highways, an the author in particular, gratefully acknowledge the assistance of the employees of District 8 who prepared profiles, made cost estimates, and provided general enginee ing which contributed much toward the completion of this study. We wish to thank C. A. Thompson, District Engineer, and O.T. Olson, Resident Engineer, for their coo ation in arranging for this assistance.

Discussion

J.L. STACKHOUSE, Maintenance Engineer, Washington State Department of Highwa —This discussion was requested by V.G. Gould, Chairman of HRB Committee on Sal vaging Old Pavements by Resurfacing, and H.E. Diers, Chairman, HRB Department of Maintenance, primarily because this paper is closely associated with the work re- viewed by the writer in a paper reported in the 1959 HRB Proceedings describing a similar type of construction performed on a Washington highway.

Mr. Velz is to be congratulated on the excellence of his paper generally and the presentation of details of the preliminary work performed with the variable load com pactor on the old cement concrete. The paper presents each step undertaken on all sections of the experimental construction on Minnesota highways and evaluates the re sults.

The writer was keenly interested to note, after reviewing this paper, the similari of results of rolling old concrete pavement with a variable load compactor with nearl identical loads of 59 tons gross weight on both projects; also, the agreement of other items noted in the report from the sections in Minnesota and Washington. The write recently had the unexpected opportunity to consult with Mr. Wade Faulk, Resident Engineer on the Ethel to Salkum project in Washington. Mr. Faulk and the writer concur on the following comparison of results as noted in this paper:

Pavement Breaker Rolling.—The vertical slab movements noted on the Washingto project agree with that found by Mr. Velz. The maximum number of passes with the compactor to produce optimum cracking was found to be from 8 to 10 which substanti agrees with the author's findings. The crack pattern of the old concrete pavement a ter rolling, as noted in this paper, was about the same as observed by Mr. Faulk an some portions of the pavement remained unchanged, whereas other portions were raised and others subsided from 1⅛ to 2 in., which concurs with the Minnesota find- ings.

It is noted that slightly greater compaction of the old pavement slab occurred at centerline rather than at the edge on the Minnesota project, whereas the edges of the slabs were compacted or subsided slightly more, generally, than the center area on the Washington section. Some areas were not compacted from their original profile However, there seemed to be no uniformity in compaction as to slabs or locations as reported by Mr. Faulk.

The delay of small cracks appearing after the start of rolling until several passes with the compactor had been made, as reported by Mr. Velz, checks closely with th action noted by Mr. Faulk. As the rolling progressed the cracks became more visib accompanied by a crushing, grinding sound in the slab as the compactor passed over the pavement.

Old Pavement.—Inasmuch as the age of the old pavements is not the same (the Mi esota pavement was constructed in 1931 and Washington's section in 1924), and the pavement on which Mr. Velz reports was reinforced with two bars at the edges and each side of the center longitudinal joint of each 10-ft slab and the Washington sectio was unreinforced with the exception of dowel bars tying the two 9-ft slabs together a enterline, it is significant to note that the results of the variable load compactor were milar as previously noted. The Minnesota section was slightly thicker in cross-secon. It is indicated the compactor was heavy enough to overcome this difference in nstruction design and to break down bridging of the pavement on the subgrade.

Performance. —It is interesting to note the radical reduction of percentage of transerse reflection cracks, reported by Mr. Velz, that appeared in the asphalt concrete rface which was laid on the concrete pavement rolled by the compactor as compared the control sections which were not rolled. This observation was made after six onths of use by traffic. The findings of Mr. Velz that less reflective cracks appeared the section constructed with the 6-in. leveling base course checks with the experience Washington's resurfacing projects the past 20 years where old cement concrete paveents have been widened and resurfaced. It is axiomatic with the writer and engineers this department's Construction Division that the thicker the asphalt concrete pavement nstructed, the less reflective cracks will develop and the longer it will take them to pear.

The performance of the Ethel to Salkum section in Washington has been excellent. ter a visual inspection made on December 8, 1960, or 3½ years after the pavement s opened to traffic, no transverse or longitudinal reflective cracks were observed ile walking and driving slowly over the pavement. Credit for the absence of reflecon cracks in the wearing surface of the Washington pavement is only partially given to e action of the compactor roller and the major part of this lack of cracks is believed be due to the 4-in. average thickness of base course and 2-in. thickness of top course rfacing material that was used to level up the old cement concrete pavement after mpaction. In the construction of these uncemented, crushed gravel surfacing courses processing with a motor patrol blade, wetting and rolling with both steel-tired and eumatic rollers, some fine particles of the aggregate were worked down into the rger cracks of the old pavement, thereby firmly wedging the broken slab pieces in ace and precluding any possibility of further rocking under loads. It is believed the anular material is the greatest deterrent to crack prevention in insulating the wearing rface from the contracting and expanding stresses of the underlying concrete pavement d eliminating any possible chance of such stresses being transmitted to the asphalt vement. The subject sections in Minnesota and Washington are not comparable in sign in this respect, inasmuch as the Minnesota pavement under discussion did not ve a crushed gravel or rock layer between the concrete pavement and the asphalt veling and wearing surface. It is also possible that the foregoing comments are inngrous in view of the policy of the Minnesota Highway Department as to design and ent and therefore the comparison may not be valid. The policy of the Plans and ntracts Division of the Washington Highway Department is that when traffic volumes licate a design standard of the ultimate width of wearing surface, shoulder and ditch-, providing funds are available, the old roadway will be reconstructed similar to the bject section in Washington. Likewise if it is the intention to improve the old highway a tolerable standard with future long-range plans to reconstruct the roadway to the imate design standard, then the old pavement is usually widened to 22 or 24 ft when s resurfaced and less than standard shoulders provided. No improvement is made areas outside the shoulders. The thickness of asphalt concrete leveling and wearing rface is usually a total of 3 in. with the expectation that reflective cracks will apr within a year in the new wearing surface. It is the present policy in Washington, en reconstructing a highway with existing concrete pavement, to break the pavement h a compactor and place leveling courses of base and top course surfacing materials er the concrete before constructing the asphalt cement pavement as described in the shington report.

Comment is made on the third paragraph of Mr. Velz' paper in discussion of the mer use of granular materials to overlay rough old pavements and particularly, also might be considered extravagant, in that the full potential of the old pavement a base course was not used and because it consumed such large volumes of good e aggregate—an undesirable feature in any case, but especially so in areas of gravel rcity."

The quantities of base aggregate to which Mr. Velz refers are not known; however, if the thickness of the overlay aggregates did not exceed 12 in., it would not seem extravagant, to the writer, if the added thickness would provide more than adequate fou tion or more than the bare needs of the wearing surface to resist forces of heavy traf fic using the highway. In the past, designers of highway foundation sections have pro vided thicknesses of materials that theoretically would develop the full potential of th base without providing for any factor of safety to allow for extremely wet conditions, frost heaves, or unusual conditions. It is common practice for designers of portland cement concrete highway structures to provide at least a factor of safety of 2½ in the structures to insure against failure. Then why is it not economical to construct foun tions under pavements that are more than just adequate to carry the current maximu loads under all conditions? Prior to 1947 the foundations on several projects in Was ington state were designed on the apparent theory of "cutting the suit pattern to fit th cloth" because of shortage of funds. The end result was that after the first winter th wearing surface failed; after much criticism and many complaints the project was re surfaced and the wearing surface was replaced. The blame for the failure, in these cases, was placed on the wearing surface by most of the uninformed citizens affecte whereas the real cause of the failure was the inadequate foundation which would not support the loads using the highway.

Attention is called to the fact that where it is indicated, leveling an old pavement require more than 6 to 7 in. average thickness of material, including 3 to 4 in. of as phalt pavement. A saving can be obtained by using granular material (such as ⅝-in. or smaller top course) for the first 3 in. As an example, the unit contract cost on the subject Washington project was $6.65 per ton for asphalt concrete leveling and wearing course and $1.95 per ton for crushed gravel surfacing or a saving of 3.4 ti the cost of asphalt concrete for the purpose of leveling.

Costs.—The comparison of cost of use of the variable load compactor of both projects indicates the rolling, as stated in Mr. Velz' paper, was computed to be $271.4 per mile for a 20-ft pavement width. This compares with $331.28 per mile for rolli an 18-ft wide concrete pavement. A comparison cost of the asphalt pavement of all courses of the two sections cannot be fairly made because of the difference of metho of leveling up the old pavement; however, Mr. Velz reports the total cost per mile f a 1½-in. wearing course over a 1½-in. binder course and the 1½-in. leveling cours was $38,340 per mile. Comparable costs on the Washington project for a 1¾-in. le ing course under a 1¼-in. course wearing surface was $17,705.53 per mile. The width of both wearing surfaces was 24 ft. Although the plan for the total thickness o the Washington project is shown to be 3 in., the thickness of this pavement was actu approximately 4½ in., due to the fact that the quantities were computed on an averag specific gravity for aggregate and the aggregates used in the mixture were slightly less. The total quantities of tons of mixtures set up under the contract were used, which resulted in a larger volume of mixture and consequently a thicker layer. Although Mr. Velz reports that a 1½-in. leveling course was used, it is believed that the average thickness in leveling up the old concrete pavement was greater than this figure.

Comments.—It is of interest to note that on one 4.2-in. contract completed in 19 in eastern Washington, where a compactor was specified to break down the old conc pavement constructed about 1932 with a 9-in. uniform slab thickness, the compacto was unable to break the slabs and eliminate the bridging effect of the pavement. Aft a trial with the 59-ton compactor, which had no crushing effect on the pavement, a 20-cu yd Euclid scraper unit was loaded to full capacity and operated over the old pa ment at a speed of from 15-20 mph. The bouncing effect of the approximately 50-to load accomplished the desired effect of breaking the slabs that were rocking under t compactor.

As a result of the experience gained on Washington highways during the past five years, it has been concluded that where a highway is to be reconstructed to ultimate standards and the existing pavement is a portland cement concrete type, the most su cessful method for obtaining an asphalt pavement wearing surface that will remain f of reflection cracks and maintain a uniform profile grade is by the use of a variable load compactor to first break down the old concrete pavement to eliminate void area and level up with a granular material.

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