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Foreword

The subject matter of the four papers published in this RECORD will be of interest to persons concerned with pavement and bridge maintenance.

In their book "Bituminous Materials in Road Construction," the Road Research Laboratory states that the first attempt to modify bituminous binders by adding rubber was in 1898. Laboratory research has shown that small percentages of rubber incorporated in bituminous binders have marked effect on the physical properties, and some field trials of rubber-asphalt combinations incorporated in paving materials have performed well.

McDonald, in his paper "A New Patching Material for Pacement Failures," reports his successful trial use of a high percentage of rubber combined with asphalt in a thinly spread slurry as a repair technique for bituminous pavement. Most previous researches have tested the use of rubber added in amounts of less than 10 percent of the asphalt, whereas McDonald tested mixtures containing as much as 33 percent rubber. The author feels that the high materials cost can be justified since a thin application substitutes for a much heavier overlay of conventional materials.

Full-depth concrete patching of concrete pavements entails the problems of removing old slab sections and doweling new concrete to the sections of pavement which have not been removed. Staib reports in detail the ingenious methods he has developed on the Ohio Turnpike for removing the old slab and doweling adjacent concrete to the newly placed concrete. Procedures have been worked out to insure that a failed section of pavement can be broken out, replaced full depth with new concrete and the repaired section opened to traffic during a normal workweek, thus avoiding interference with traffic during the weekend periods.

Sections of highway in North Dakota developed spalling at the joints necessitating corrective action. The repair procedure accomplished in late 1964, as reported by Rice and Kyser, consisted of removing the unsound concrete adjacent to the joint, and replacement with an epoxy resin mortar patch on one side and a PCC mortar patch on the other side of the repaired joint. The joint was resawed and a neoprene compression seal was then inserted. This procedure differed from work in Great Britain wherein irregular cracks reportedly were repaired by removing unsound concrete, placing an extended neoprene rubber strip, reinforced with malleable wire, and then replacing both sides of the crack with epoxy resin/sand mortar which bonded to the rubber strip.

It has been demonstrated that asphalt concrete pavements are not necessarily impermeable. Engineers, noting deteriorating concrete bridge decks, frequently err by assuming that an asphaltic concrete overlay will provide a waterproof covering through which damaging fluids cannot pass. In many instances concrete deterioration continues unobserved under the overlay. The Port of New York Authority, facing a bridge inspection problem, retained a consultant to develop a method for nondestructive testing of concrete pavements supporting asphalt overlays. Phelps and Cantor report the development of direct, digital reading microsecond timing instruments and accessories for routine monitoring of concrete base-slab conditions over large portions of asphalt-overlaid structures, and for determining asphalt thickness without disturbance of the material.

Contents

A New Patching Material for Pavement Failures

CHARLES H. McDONALD, Engineering Supervisor, City of Phoenix, Arizona

The increasing volume of traffic, and particularly heavy traffic, has created a severe problem on many roads and streets in the country. This problem has resulted from elastic type failures in a "chicken wire," or "alligator" pattern cracking. The cracking is caused by fatigue of the surface from repeated deflection. Repairs by overlaying are usually effective for a short period only and many other more drastic measures are too expensive and often also ineffective.

This paper is about the successful search for a material which can be used for repairing this type of failure at reasonable cost. It involves a thin application of a hot compound of asphalt and rubber that has high elasticity and flexibility together with low temperature susceptibility. The use of aggregate is not the dominant feature in this process as it is used only to prevent pick up and excessive wear. As far as the author has been able to determine, the use of the material in this manner is unique. The experiment involved has been in place long enough to demonstrate its worth, and it should prove to be a new weapon in the hands of maintenance forces against the ever-increasing deterioration of our roads and streets.

•THE so-called "flexible-type pavement" is actually not a particularly flexible structure; there are occasions when it could be classed as very brittle, particularly in cold weather or when the surface has suffered a long period of embrittlement from oxidation and age. The cracking caused by this lack of flexibility has created a tremendous problem, when considered on a nationwide scale. In traveling over the streets and highways of this country, one can seldom go more than a few miles without finding distressed pavement that is basically caused by a repeated flexing of the surface under the traffic loads.

This type of failure has been variously defined as flexure cracking, elastic-type failure, and fatigue failure. It is characterized by multiple cracking of the "chicken wire," or "alligator," type pattern without plastic deformation of the surface. The cracking is due to fatigue of the bituminous mixture from repeated deflection under load and subsequent recovery of the surface, which in turn is caused by elasticity of some member of the substructure. It is the most prevalent of the three most common types of failure occurring in flexible-type pavements. The others are the following.

1. The plastic type, which is manifested by cracking of the same character as the elastic-type of failure, but is also accompanied by plastic deformation of the surface. The surface is depressed under the loaded area and usually slightly raised at one or both sides of the loaded area. This type is usually caused by inadequate thickness of base material and is no longer a serious problem on highways or streets built under modern design criteria.

2. The surface-type failure, which is characterized by attrition, or stripping and emulsification of the asphalt in the surface of the pavement. There is raveling and loss

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of material from the surface but no significant amount of cracking. Although this type of failure is very common, it is not as serious as it can be corrected by the application of a seal coat.

Fatigue cracking resulting from the elastic-type failure is entirely different, and solutions have not only been difficult and expensive, but in many cases quite uncertain in their results because there is resilience in some member of the substructure. This resilience must be counteracted by either making the substructure or the surface so rigid that it cannot bend, or by making the surface so flexible that it will take the bending. Part of the problem lies in the fact that the deflections required to produce the elastic-type failure are so small that almost complete elimination of the resilience is required. Repeated deflections of a very small order are sufficient to produce this type of failure. Various authorities (2, 3), have given figures for a critical deflection which range from 0.010 to 0.050 in. with the certain probability that the critical deflection would vary considerably for pavements of different thickness, composition (7), asphalt grade, asphalt content, asphalt quality, prevailing temperatures, and radius of the deflection curve.

Complicating the problem is the fact that the source of such a small magnitude of elasticity may be difficult to determine. It may be either in the subgrade, subbase, or base course. An increase in the normal moisture content of even a good subgrade (for instance, by frost action) may cause it to become "quickie," resulting in a condition where the load is borne by hydrostatic pore pressure. Although such a condition does not ordinarily last for a long time, there is almost no reasonable thickness of overlying material that will prevent the deflection. The surface of a 4-ft fill over a quickie soil has been observed to visibly deflect under load. This condition also develops in densely graded base courses through frost action.

Certain materials present in soils, such as mica, have elasticity within themselves, and the economic necessity of using local materials may require that these materials be incorporated in the substructure. Such materials are often the only ones available in the particular area without incurring excessive cost. Perhaps the most common cause is entrapment of minute quantities of air (1) in fine-grained subgrade soil. Any soil which is capable of moderate capillary pressure can entrap air under certain moisture conditions by holding it in pores which are sealed on all sides by capillary moisture. The capillary pressure is sufficient to prevent the air from being expelled under traffic loading. If enough of these entrapped air cells are involved, the structure has a pneumatic character. In extreme cases such soils have an almost rubber-like elasticity when pressed between the fingers. The moisture content need only be slightly above optimum to entrap air. This type of soil is surprisingly prevalent throughout the United States.

The increasing use of cement-treated bases is, I believe, whether recognized or not, an attempt to overcome this problem of substructure elasticity by stiffening that structure. The so-called "up-side-down" method of construction in which the subbase is cement-treated, rather than the base, is a quite obvious attempt to stiffen the substructure against resilience from an underlying member. This is practiced rather commonly in New Mexico (4) and Arizona (5). Incidently, I have observed that this type of treatment has been quite successful.

The use of rigid portland cement concrete pavements has also been quite effective; however, the cost is generally prohibitive for indiscriminate use. Again, the obvious motive is to make the structure so rigid that it will not be affected by resilience of the substructure.

An attack against this type of failure has also been mounted from the other standpoint of attempting to make the bituminous mixture more flexible (6). This has been done by the use of open-graded plant mixes employing very heavy asphalt films on each particle. These mixes have large void spaces so that the high asphalt content, in relation to surface area, will not cause distress. This type of design has helped to ameliorate the situation but has not been a cure all.

Similarly, small percentages of rubber incorporated in mixes have also been used. These small percentages of rubber have undoubtedly been beneficial, although information on the degree of success obtained with these mixes for this purpose appears to be somewhat limited. It is also my opinion that the cost of these materials has prevented the use of rubber in the amounts necessary to give the pavement true elasticity.

EXPERIMENTAL APPROACH

An entirely new approach was needed, and the approach employed in the experiments described in this paper is completely different in its use of rubber from anything which I have read. This approach embodies the use of a relatively high percentage of rubber, combined with asphalt, in a relatively thin application. The purpose is to keep the overall cost in bounds but still obtain maximum elasticity of the patching material. Although this approach may be unique, it is to this date completely successful in some extremely difficult situations. The cost is not out of line with heavier overlays which are commonly used, generally unsuccessfully, in combating this problem. It must also be remembered in maintenance repair work that the cost of the materials is a relatively minor item. The big cost is the labor involved, and anything that will eliminate repeated repairs to the same distressed area is cheap.

We found in the laboratory that a rubber-asphalt material could be made that had a consistency of thick slurry when hot. By experimentation it was found that the best consistency for our purpose was obtained by heating 85-100 penetration grade asphalt to approximately 420 deg and then stirring into it partially devulcanized reclaimed rubber (a commercial product) in the proportion of 2 parts of asphalt to 1 part of rubber. Copies of this series of laboratory tests are attached in the Appendix. These tests show that the consistency depends not only on the rubber content, but also on the degree of the solution, or "jelling," of the rubber. The higher the temperature of the mixing, the greater the degree of the solution of the rubber in the asphalt and the more nearly the end product resembles the properties of rubber rather than asphalt. In other words, when the material is mixed at a temperature of 350 F it is quite fluid and has the consistency of a thin slurry. This would be very convenient for placing; on the other hand, it would be more temperature susceptible so that it would tend to bleed more readily in the summer, and it would have less elasticity than the thicker product. The same may be said for reducing the rubber content. In other words, it can be made at any consistency desired but it must be remembered that in doing so the properties of the final product will be changed. The thinner the product, the more nearly its properties will resemble those of asphalt; the thicker the product, given the same proportions, the more they will resemble those of rubber.

Other laboratory mixes were made using synthetic rubber latex mixed with emulsified asphalt and sand in various proportions so as to form a slurry seal type of material. The mixture of this type considered best for the purpose of the experiment consisted of 2 parts of SS-1H emulsified asphalt to 1 part of synthetic rubber latex emulsion, with concrete sand added in sufficient quantity to make a slurry consistency. The details on this are also given in the Appendix.

At a later date it was decided to experiment with a mixture of 85-100 penetration grade asphalt and ordinary reclaimed rubber obtained from a local vulcanizing shop. This was a finely granulated product obtained from the buffing of tires for recapping. This material was also mixed in the proportion of 2 parts of 85-100 penetration grade asphalt to 1 part of reclaimed rubber. Experiments indicated that when this material was heated over 440 F the final product was softened to an undesirable extent. This was contrary to what we had found with the partially devulcanized product. Heating to approximately 420 F produced the most desirable results (see Appendix).

FIELD SURVEY

After studying the results of this work, a field survey was made to determine the location of the most severe test conditions that could be found so that the answers to the experimental work would be quickly forthcoming. The criteria were to locate pavements where the traffic was heavy, preferably with a high percentage of heavy truck traffic, and severe elastic-type failure had already occurred. An area where poor drainage was involved was also desirable for our purpose and one of the test areas did have exceedingly poor drainage (Fig. 1).

Figure 1. General area occupied by test panels Nos. 4,5,6, and 10, showing poor drainage.

Figure 2. Test panel No. 11, showing reclaimed rubber asphalt still in good condition, whereas sand-asphalt patch is failing.

The locations, all in the city of Phoenix, selected for the various test panels were as follows.

1. Test panels Nos. 1, 2, 3, 4, 5, 6, 7, and 10 were on Seventh Street just south of Jefferson Street where the traffic volume numbered 13,200 veh/day, a large proportion of which were trucks as this street serves an industrial area. The pavement

Figure 3. Test panel No. 9 showing superior condition of rubber-asphalt repair.

Figure 4. Test panel No. 9 (in background) showing arrest of cracking.

surface was generally covered by alligator-pattern cracking in an advanced state and the drainage was extremely poor. The cracking pattern was similar to that shown in Figure 2, and Figure 1 shows the poor drainage.

2. Test sections Nos. 8 and 9 were located on north Central Avenue with daily traffic volumes of 30,800 and 38,400, respectively. Most of this traffic is of passenger type. There was severe alligator-type cracking in the wheel tracks but it was not spread as generally over the street as in the previous case. Test panel No. 9 (Fig. 3) shows par-

tially devulcanized rubber-asphalt patch in foreground and slurry seal patch, made approximately one week later, in background. The rubber-asphalt repair is clearly superior. In Figure 4 test panel No. 9 shows the arrest of cracking with less than a ¼-in. thickness of partially devulcanized rubber-asphalt compound.

3. Test panel No. 11 was placed on Washington Street which has a traffic count of 18,500 veh/day. Many of these vehicles are of the commercial and industrial type. This section was in an area which had given continuous trouble for some time. Figure 2 shows test panel No. 11 in upper center and hot mixed sand-asphalt patch in lower half; both were placed the same day but the reclaimed rubber asphalt is still in good condition whereas the sand asphalt patch is failing.

Details on all of the foregoing test panels are given in Appendix A; however, a brief resume of their performance is related here. Test panel No. 1 consisted of ½ gal/sq yd of a 50-50 mixture of SS-1H emulsified asphalt and rubber latex emulsion sanded to prevent pickup. The panel was lost due to insufficient curing time before the entry of traffic which destroyed it. It was replaced by a conventional sand asphalt mix placed by maintenance crews on the following day.

Test panel No. 2 consisted of 1 gal/sq yd of a 50-50 mixture of SS-1H asphalt emulsion and synthetic rubber latex emulsion mixed with sand to slurry consistency. Here again the panel was torn up by traffic before it could properly set. Maintenance crews placed a conventional sand asphalt mix over this patch area the following day.

Test panel No. 3 directly adjoined the first two to the south and consisted of 1 gal/sq yd of a mixture composed of 2 parts of 85-100 penetration grade asphalt and one part partially devulcanized reclaimed rubber. One-quarter inch maximum-sized, clean-cover aggregate was spread over the surface and tamped into the material. Approximately three weeks after placement the area was subjected to 58 hr of steady rain and partial inundation while being pounded by traffic. The test section was unaffected; however, the conventional sand-asphalt mixes which had been placed over the adjoining panels, Nos. 1 and 2, were cracking. They were also raveling due to partial emulsification of the asphalt. A few months later these patches were almost completely destroyed, but the rubber asphalt mixture on test panel No. 3 showed no reflection cracking from the underlying cracks or any other distress.

Test panel No. 4 consisted of the same material as No. 3 and was subjected to the same conditions. It is still in good condition. Traffic can be allowed on these materials as quickly as they have cooled—quite an advantage on a heavily traveled city street.

Test panel No. 5 consisted of 1 part of synthetic rubber latex emulsion, 2 parts of SS-1H emulsified asphalt and 6 parts of slurry sand placed at the rate of 1 gal/sq yd. Traffic was permitted to use it after 2 hr of curing. This panel failed approximately 2 weeks after placement after being under constant rain and partial inundation for 40 hr while subjected to heavy traffic. It appeared that the asphalt component had re-emulsified, leaving a porous skeleton of rubber strands which were destroyed by the traffic. The failure of the SS-1H and synthetic rubber latex with slurry sand in test panel No. 5 is not believed to be significant insofar as a normal condition is concerned. This material had very good elasticity, and, had the drainage situation been normal, there is little doubt that it would be in service today and probably resisting flexure cracking as well as the other rubber-asphalt panels. That experiment should be repeated at another location.

Test panel No. 6 was made from a proprietary emulsion of latex and asphalt (Indaco 200) applied at the rate of 1 gal/sq yd and cover aggregate was added. It was allowed to cure for 2 hr before being subjected to traffic. It shows no distress.

Test panel No. 7 consisted of the same material as test panels Nos. 3 and 4. However, it was spread at a different rate than the foregoing panels which would average approximately 0.18 in. in thickness for a rate of 1 gal/sq yd. This panel was spread to a depth of ¼ to ½ in. thick and followed with ¼-in. cover aggregate rolled with a steel roller. This panel is also in good condition as of this writing.

Test panel No. 8 was similar to test panel No. 7 as it was made of the same material and was spread to the same thickness but in an entirely different location area. It is in fair condition but shows some distress. While rolling it with a steel roller to set the cover aggregate, some of the hot mixture was squeezed up through the cover aggregate and the roller picked it up, creating a bald spot or two. This did not appear to affect the properties of the patch, but rolling the material when it was too hot caused uneven penetration into the cover aggregate, resulting in a certain roughening of the surface texture. This roughness eventually ironed out under traffic.

Test panel No. 9 again consisted of 2 parts of 85-100 penetration grade asphalt and 1 part of partially devulcanized reclaimed rubber, a commercial product, spread at the rate of 1 gal/sq yd of mixture followed by a cover aggregate. This was placed over a portion of a long strip of failed area. The remaining portion of this strip was repaired by a conventional maintenance slurry seal patch approximately a week later. The slurry seal patch had already shown distress only a week after replacement, and had completely failed after a few months (Fig. 3). It was replaced April 14, 1965, by maintenance forces with a hot asphaltic-concrete mix. However, the rubber-asphalt test panel was still in good condition except for a few spots which had been "picked up" by the roller during placement. One of these spots is right of center, on the lower edge of Figure 3. Maintenance had also placed some slurry seal patches nearby which contained 2 percent synthetic latex emulsion but these also failed early as the rubber content was insufficient to give it the required elasticity. All of the slurry seal patches had to be completely replaced approximately three months after placement. The test panel was in good condition except for a small exceptionally thin spot where reflection cracking showed up to a minor degree (Fig. 5). The ¼-in. cover aggregate as used in these tests was completely covered by the rubber-asphalt mixture after a few days under traffic. In the spot where the cracking occurred the aggregate had not been covered as there was insufficient material to squeeze up around it. Where normal thickness was obtained, there was no reflection cracking.

Test panel No. 10 consisted of 2 parts of 85-100 penetration grade asphalt and 1 part of locally obtained, unprocessed, shredded, reclaimed rubber from a local tire shop. The asphalt was heated to a temperature of 440 F before being mixed with the rubber and the mixture was rather soft on curing. It was also covered with ¼-in. cover aggregate and is in good condition.

Test panel No. 11 was the same as test panel No. 10 except it was placed in a different location and the asphalt was heated to a temperature of 420 F. This mixture was

Figure 5. Test panel No. 9, showing slight reflection cracking.

not as soft as that in test panel No. 10. At the same time that this panel was being placed, a maintenance crew was placing adjacent hot mix sand-asphalt patches. After approximately one month of service, these conventional patches were beginning to show reflection cracking. After another month, the reflection cracking was quite pronounced (Fig. 2). There has been no reflection cracking in the rubber-asphalt mixture to date.

A tack coat was used before placement of the experimental materials on all except the first three panels. It is deemed desirable for the best results.

Experimentation on other test panels was continued in the summer of 1965 for the purpose of observing the effects of placement of the material at elevated summer temperatures and to try out other rubber compounds. These panels (see Appendix) have not been in place a sufficient length of time to warrant conclusions.

Insofar as we have been able to judge there is no apparent difference in the performance of the panels made with the partially devulcanized reclaimed rubber, a commercial product, and the ordinary reclaimed shredded rubber obtained from a local vulcanizing shop. There is, however, a difference in the reaction of the two to the asphalt. The partially devulcanized reclaimed product seems to make a stiffer product when mixed at higher temperatures, whereas the reverse is true with the conventional reclaimed rubber (Appendix C). The ideal asphalt temperature for mixing either of these products appears to be approximately 420 F.

The temperature susceptibility of this rubber-asphalt material is far less than with asphalt alone. This is, of course, a tremendous advantage in achieving control of reflection cracking. It retains some flexibility down to approximately freezing temperatures and although it does soften under summer heat, it apparently does not pick up. The material will be quite soft to the touch when warm and show tracking under truck tires, but instead of shoving and rolling, it rebounds and tends to resume its original location. A somewhat leathery skin develops on the surface which is dry and resists pick up. It will pick up, however, if a tacky material such as asphalt is applied to this dry surface, because the bond with the underlying surface is not unduly strong due to the softness of the material.

Patching by this process is actually almost comparable to that of the manufacture and placing of slurry seal, in that a finely divided material is added to a liquid. The only difference between this process and the slurry seal is that the liquid must be hot; otherwise, it is the same. The material is smoothed out with a rubber squeegee in the same manner on the street. Although ours was strictly a hand operation, the mechanization would be similar to that used for slurry seal. The similarity, however, ends after smoothing with the squeegee. In the case of a rubber-asphalt compound, a cover aggregate surface is added to prevent traffic pick up and traffic may be allowed on it as soon as it cools which is almost immediately. Whereas, with the slurry seal a considerable curing time must elapse before traffic can be permitted to use it; otherwise it will be destroyed.

In conclusion, it appears that the use of either the partially devulcanized reclaimed rubber, a commercial product, or the conventional reclaimed rubber derived from the buffing of tires for recapping, together with 85-100 penetration grade asphalt in the proportions developed by these experiments, will prevent reflection cracking from elastic-type failures caused by fatigue cracking at very nominal cost. It is thought that this has never been achieved by skin patching with any other material. This should prove to be a boon to maintenance forces throughout the country who are plagued with repairing this type of failure.

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Appendix A

TEST PANELS OF EXPERIMENTAL PATCHING MATERIAL

Test Panel No. 1 — Placed: Dec. 16, 1964

This panel, located on 7th St. south of Jefferson, consisted of ½ gal/sq yd of a mix consisting of 1 part SS-1H (40 percent H_2O) and 1 part latex emulsion (33 percent H_2O).

Sanded to prevent pickup; traffic turned on it within ¾ hr after placing which was insufficient for curing as they promptly tore it up. A conventional mixed sand-asphalt skin patch was placed Dec. 17, 1964. Heavy rain Dec. 18, 1964.

Observed July 1, 1965. Sand-asphalt patches put in by street maintenance have completely failed and have been repatched with the same material. Some cracking is beginning to appear in the new patches.

Test Panel No. 2 — Placed: Dec. 16, 1964

This test panel, located on 7th St. south of Jefferson, consisted of 1 gal/sq yd of a mixture consisting of 1 part SS-1H (40 percent H_2O) and 1 part latex emulsion (33 percent H_2O) and concrete.

Sand added to a slurry consistency. Traffic turned on test panel within ¾ hr after placing, which was insufficient time for curing. Traffic promptly tore it up. A conventional mixed sand-asphalt skin patch was placed by street maintenance, Dec. 17, 1964. Heavy rain Dec. 18, 1964 (0.5 in.).

Observed July 2, 1965. Sand-asphalt patch placed by maintenance has been completely destroyed. This area has been repatched and some cracks are beginning to appear.

Test Panel No. 3 — Placed: Dec. 16, 1964

This panel, located on 7th St. south of Jefferson, consisted of 1 gal/sq yd of a mix consisting of 2 parts 85-100 penetration asphalt and 1 part partially devulcanized reclaimed rubber.

The mixture was applied at a temperature of 420 F and cover aggregate was spread over the surface to prevent pickup. Traffic was turned on it within ¾ hr. This was sufficient time for curing as the material set up on cooling. Test panel was in good condition 1 day later (Dec. 17, 1964). Heavy rain (Dec. 18, 1964) and traffic caused the test panel to loosen at the edges, indicating the need for a tack coat.

Observed Jan. 8, 1965. Unaffected by 58 hr of steady rain and partial inundation under heavy traffic. At this time the conventional sand-asphalt patch that had been placed over the site of test panels No. 1 and No. 2 were cracking and raveling out due to partial emusification of the asphalt.

Observed April 9, 1965. Excellent condition but adjoining conventional patches were almost completely destroyed.

Observed July 1, 1965. Panel in excellent condition. Material appears to be hardening. Moderate abrasion has occurred on west edge about 1 ft in width, no cracking visible.

Test Panel No. 4 — Placed: Dec. 23, 1964

This test panel, located on 7th St. south of Jefferson, was tacked with MC-250 liquid asphalt. The test panel consisted of 1 gal/sq yd of a mixture of 2 parts 85-100 penetration grade asphalt and 1 part partially devulcanized reclaimed rubber.

The mixture was supplied at a temperature of 420 F and cover aggregate was spread over the surface. It would have been ready for traffic as soon as it cooled but it was held off for 2 ± hr to allow adjoining sections containing emulsion to cure. Rained (Dec. 28, 1964, 0.25 in.).

Observed Dec. 28, 1964. Condition good.

Observed Jan. 8, 1964. Unaffected by 58 hr of steady rain and partial inundation under heavy traffic.

Observed July 2, 1965. Test panel in excellent condition; no stripping has occurred; no cracking present. Material is harder than more recently placed test panels. Test panel has spread to a small extent in the direction of traffic.

Test Panel No. 5 — Placed: Dec. 23, 1964

This test panel, located on 7th St. south of Jefferson, was tacked with MC-250 liquid asphalt. The test panel consisted of 1 gal/sq yd of a mixture consisting of 1 part latex emulsion (33 percent H_2O), 2 parts SS-1H emulsified asphalt (40 percent H_2O), and 6 parts slurry sand.

Latex emulsion was mixed into the SS-1H mixture. It was the consistency of slurry. Traffic was turned on it after 2 hr of curing. Rained Dec. 28, 1964 (0.25 in.). Observed Dec. 28, 1964, condition ok.

Jan. 7, 1965. This panel partly failed this morning after being under constant rain and partial inundation for 40 hr while subjected to heavy traffic. It apparently partially re-emulsified the asphalt as pieces scattered about showed a porous skeleton of rubber strands.

Observed July 2, 1965. This test panel has stripped about 75 percent. Remaining material is in excellent condition.

Test Panel No. 6 — Placed: Dec. 23, 1964

This panel, located on 7th St. south of Jefferson, was tacked with MC-250 liquid asphalt. This test panel consisted of 1 gal/sq yd of a proprietary asphalt-latex emulsion mixture (Indaco 200) normally used as an elastic joint sealer. Cover aggregate was added. Traffic was turned on it after 2 hr of curing. Rained Dec. 28, 1964 (0.25 in.).

Observed Dec. 28, 1964. Condition good.

Observed Jan. 8, 1965. Unaffected by 58 hr steady rain and partial inundation under heavy traffic.

Observed July 2, 1965. Excellent condition. Test panel has spread in the direction of traffic; no cracking present. Somewhat softer than test panel No. 4. Resembles a rubber mat.

Test Panel No. 7 (4- by 8-ft) — Placed: Jan. 5, 1965

Two hundred to 208 ft south of fire hydrant at the southeast corner of 7th Street and Jefferson, and 9 to 13 ft west of the west edge of sidewalk on 7th Street. Tacked with 4 parts of 85-100 penetration grade asphalt to 5 parts of kerosene.

This panel consisted of 5 gal of 85-100 penetration grade asphalt and 21 lb of partially devulcanized reclaimed rubber (2 parts of 85-100 penetration grade asphalt to 1 part rubber by weight). Temperature of the asphalt was 420 F when mixed with the rubber. The mixture was spread to an area of $3\frac{1}{2}$ sq yd and about $\frac{1}{4}$ to $\frac{1}{2}$ in. thick. The entire test panel was completely covered with aggregate and rolled with a steel roller.

Observed July 2, 1965. Excellent condition. Test panel feels quite soft due to thicker mat than others. Some spreading has occurred in the direction of traffic.

Test Panel No. 8 ($3\frac{1}{2}$- by $13\frac{1}{2}$-ft) — Placed: Jan. 5, 1965

Turney Avenue and center lane of the west half of Central Avenue 19 ft east and 1 ft north from the drop inlet at the northwest corner of Central Avenue and Turney Avenue to the southwest corner of the test panel. Tacked with 4 parts of 85-100 penetration grade asphalt and 5 parts of kerosene.

This test panel consisted of 5 gal of 85-100 penetration grade asphalt and 21 lb of partially devulcanized reclaimed rubber (two parts of 85-100 penetration grade asphalt to 1 part rubber and cover aggregate). Temperature of the 85-100 penetration grade asphalt was 430 F, when mixed. The mixture (rubber and asphalt) was spread to an area of 5.30 sq yd and about ¼ to ½ in. thick. The test panel was then rolled with a steel roller after placing cover aggregate.

Observed July 8, 1965. Condition fair. Test panel very soft and spreading in the direction of traffic and to the side causing thin spots with some minor resultant reflection cracking. This test panel appeared to be softer than the other panels. This is probably due to a combination of an excess of tack coat and greater thickness of the rubber-asphalt material resulting in slower curing. Ideal average thickness seems to be 0.18 in. ±, obtained by spreading 1 gal/sq yd of the combined materials. The entire street was recently fog sealed and this seems to have had an adverse sealing effect on the rubber-asphalt. After the fog seal cured, it could not respond to elastic movements of the rubber and cracked badly tending to propagate cracking downward over the areas where the rubber-asphalt overlaid cracked and moving areas in the original pavement.

Test Panel No. 9 (3½- by 51-ft) Placed: Jan. 8, 1965

East half of Central Avenue south of Indian School Road west lane 1 ft east of east face of median curb, 210 ft south of south curb on Indian School Road to 261 ft south (opposite 4041 North Central Avenue).

Work was done by 3-man maintenance crew.

Tacked at 11:45 A.M., with 4 parts of 85-100 penetration grade asphalt and 5 parts of kerosene. Asphalt was heated before arrival on the job.

85-100 penetration grade asphalt, 400 F
Mixed and was spread at 11:55 A.M.
Cover aggregate applied at 12:10 P.M.
Rolled at 12:15 P.M.

Consisted of 15 gal of asphalt and 62 lb of partially devulcanized reclaimed rubber = 20 sq yd. Material cost $12.00 ± or $0.60/sq yd. 1 gal/sq yd of a mixture applied and followed by a cover aggregate which was applied.

January 21, 1965. A conventional slurry seal patch was placed January 17, 1965, extending south from test panel No. 9. It had already failed by the time of this observation and test panel No. 9 was ok. Photos taken. Slurry seal patches containing 2 percent latex, placed Jan. 17, 1965, on opposite side of median had also failed.

April 14, 1965. The slurry seal patches extending south from panel No. 9 were replaced with conventional hot mix. Some hot mix placed in a few spots on test panel where "pulling" had occurred, probably by roller at time of placement. Test panel showed no cracking except for a small spot where it was so thin that the cover aggregate was exposed in relief. Generally, the cover aggregate normally becomes buried under traffic action in this process.

May 17, 1965. Test panel was inadvertently destroyed by maintenance force. It was still functioning perfectly in preventing crack reflection.

Test Panel No. 10 (3- by 6-ft) Placed: Mar. 2, 1965

Location: 75 to 81 ft south of fire hydrant at the southeast corner of 7th Street and Jefferson Street, and 9 to 12 ft west of the west edge of sidewalk on 7th Street (adjacent to test panel No. 6).

Tacked with 4 parts of 85-100 penetration grade asphalt to 5 parts of kerosene.

This test panel consisted of an application of 1 gal/sq yd of a mixture of 2 parts of 85-100 penetration grade asphalt to 1 part of locally obtained, unprocessed, shredded, reclaimed rubber by weight. Temperature of the asphalt was 440 F, when mixed with the rubber. The mixture was spread over an area of 2 sq yd and at an average thickness of 0.18 in. This material was then completely covered with ¼ in. seal coat aggregate and turned over to traffic.

Observed July 2, 1965. Excellent condition. Some spreading has occurred in the direction of traffic. No cracking has occurred. Test panel resembles rubber mat. No stripping has occurred. More elastic than test panels Nos. 3, 4 and 6.

Test Panel No. 11 (3- by 6-ft) Placed: Mar. 2, 1965

Location: 61 to 67 ft west of east end of median curb on east Washington at 26th Street and 8 to 11 ft north of north edge of median curb.

Tacked with 4 parts of 85-100 penetration grade asphalt to 5 parts of kerosene.

This test panel consisted of an application of 1 gal/sq yd of a mixture of 2 parts of 85-100 penetration grade asphalt to 1 part of locally obtained, unprocessed, shredded, reclaimed rubber by weight. Temperature of the asphalt was 420 F, when mixed with the reclaimed rubber. The mixture was spread over an area of 2 sq yd and at an average thickness of 0.18 in. This material was then completely covered with $\frac{1}{4}$-in. seal coat aggregate.

March 9, 1965. An observation made this date showed the test panel to be in good condition but hot mix sand-asphalt patches adjacent to the test panel, and applied the same day, showed the beginning of crack reflection. Photos taken.

Observed July 8, 1965. Test panel is in excellent condition except for an 8-in. hole in the center that was dug for test purposes. The test panel is not as soft as some others nor is there any spreading or cracking evident.

Test Panel No. 12 (3- by 6-ft) Placed: June 22, 1965

Location: 208 to 214 ft south of fire hydrant at the southeast corner of 7th Street and Jefferson Street, and 10 to 13 ft west of the west edge of sidewalk on 7th Street (adjacent to test panel No. 7).

No tack was applied.

This test panel consisted of an application of approximately 1 gal/sq yd of a mixture of 2 parts of 85-100 penetration grade asphalt to 1 part of partially devulcanized reclaimed rubber by weight. Temperature of the asphalt was 430 F, when mixed with the rubber. The mixture was spread over an area of 2 sq yd. This material was then completely covered with $\frac{1}{4}$-in. seal coat aggregate and turned over to traffic.

The atmospheric temperature was 101 F, and the pavement temperature was 138 F at the time of application.

June 23, 1965. Heavy thunder showers on this date resulted in nearly an inch of rain. The test panel showed no sign of damage, although it was subjected to traffic in a completely submerged condition for many hours.

Observed July 2, 1965. Excellent condition. No stripping or cracking. No spreading, and the surface is smooth.

Test Panel No. 13 (3- by 6-ft) Placed: June 22, 1965

Location: 214 to 220 ft south of fire hydrant at the southeast corner of 7th Street and Jefferson Street, and 10 to 13 ft west of the west edge of sidewalk on 7th Street (adjacent to test panel No. 12).

No tack was applied.

This test panel consisted of an application of approximately 1 gal/sq yd of a mixture of 2 parts of 85-100 penetration grade asphalt to 1 part No. 9306 company designation No. 30 mesh ground whole tire rubber (a commercial product of U. S. Rubber Reclaiming Company) by weight. Temperature of the asphalt was 450 F, when mixed with the rubber. The mixture was spread over an area of 2 sq yd. This material was then completely covered with $\frac{1}{4}$-in. seal coat and turned over to traffic. The atmospheric temperature was 101 F, and the pavement temperature was 138 F, at the time of application. Consistency was too thick for proper spreading. It shows disconnected areas. It appears that the mixture should consist of a relatively greater amount of 85-100 penetration grade asphalt for best workability with this material.

June 23, 1965. Heavy thunder showers on this date resulted in nearly an inch of rain. The test panel showed no sign of damage, although it was subjected to traffic in a completely submerged condition for many hours.

Observed July 2, 1965. Condition excellent. Surface is a little rough but seems to be ironing out under traffic.

Test Panel No. 14 (3- by 6-ft) Placed: July 2, 1965

Location: 220 to 226 ft south of fire hydrant at the southeast corner of 7th Street and Jefferson Street and 10 to 13 ft west of the west edge of sidewalk on 7th Street (adjacent to test panel No. 13). No tack was applied. This test panel consisted of an application of approximately 1 gal/sq yd of a mixture of 3 parts of 85-100 penetration grade asphalt to one part company designation No. 9306, No. 30 mesh ground whole tire rubber (a commercial product of the U. S. Rubber Reclaiming Company) by weight. Temperature of the asphalt was 400 F, when mixed with the rubber. The mixture was spread over an area of 2 sq yd. This material was then completely covered with concrete sand and turned over to traffic. The official atmospheric temperature was 107 F, the atmospheric temperature 3 ft above the pavement was 114 F, and the pavement temperature was 156 F, at the time of application.

Test Panel No. 15 (3- by 6-ft) Placed: July 2, 1965

Location: 226 to 232 ft south of fire hydrant at the southwest corner of 7th Street and Jefferson Street and 10 to 13 ft west of the west edge of sidewalk on 7th Street (adjacent to test panel No. 14). No tack was applied. This test panel consisted of an application of approximately 1 gal/sq yd of a mixture of 2 parts of 85-100 penetration grade asphalt to one part company designation No. V-17 asphalt soluble rubber (a commercial product of the U. S. Rubber Reclaiming Company) by weight. Temperature of the asphalt was 410 F when mixed with the rubber. The mixture was spread over an area of 2 sq yd. This material was then completely covered with concrete sand and turned over to traffic. The official atmospheric temperature was 107 F, the atmospheric temperature 3 ft above the pavement was 114 F, and the pavement temperature was 156 F, at the time of application.

Appendix B

TEST A

CONSISTENCY EXPERIMENT FOR PARTIALLY DEVULCANIZED RECLAIMED RUBBER - 85-100 PENETRATION GRADE ASPHALT MIXTURE

One hundred grams of 85-100 penetration grade asphalt were weighed into each of 4 beakers. The beakers were labeled No. 1, No. 2, No. 3, and No. 4, and the contents were heated to 350 F, 400 F, 450 F, and 500 F, respectively. Fifty grams of partially devulcanized reclaimed rubber were mixed with the contents of each beaker containing the 85-100 penetration grade asphalt at their respective temperatures.

1. Temperature at time of mixing - 350 F.
   Consistency - thin slurry.
   Mixed for 2 min before observing consistency.
   Consistency after 16 hr curing at 140 F - soft, sticky, and stringy.
   Consistency after 4 hr curing at 250 F - soft.
2. Temperature at time of mixing - 400 F.
   Consistency - slurry.
   Mixed for 2 min before observing consistency.
   Temperature after mixing - 300 F.
   Consistency after 16 hr curing at 140 F - soft, sticky, and stringy.
   Consistency after 4 hr curing at 250 F - soft.
3. Temperature at time of mixing - 450 F.
   Temperature after mixing - 330 F.
   Mixed for 2 min before observing consistency.

Consistency - thick slurry.
Consistency after 16 hr curing at 140 F - semi-soft, not sticky.
Consistency after 4 hr curing at 250 F - soft.

4. Temperature at time of mixing - 500 F.
Temperature after mixing - 350 F.
Consistency - very thick slurry.
Mixed for 2 min before observing consistency.
Consistency after 16 hr curing at 140 F - spongy, not sticky.
Consistency after 4 hr curing at 250 F - soft.

General notes. Elasticity of the cold material was better with samples mixed at the higher temperatures and best with sample mixed at 500 F, but hot workability was poor at that temperature. The best compromise would seem to be a mixing temperature of 400 to 450 F.

Material was somewhat brittle at 190 F, but ductile at 36 F.

TEST B

FLOW TEST OF PARTIALLY DEVULCANIZED RECLAIMED RUBBER - 85-100 PENETRATION GRADE ASPHALT MIXTURE

The four samples from test A were cooled to room temperature. A portion of mixture, the size of a pea, was taken from each of the four samples. This pea-sized sample was placed on a shiny piece of tin plate, 1 in. apart and in a line on the upper ⅔ of the tin plate. The tin plate was placed in the oven at an angle of 30 deg. The amount of flow for each pea-sized sample was observed under various temperatures.

The following data were obtained.

Flow at 170 F for 2 hr

Sample No.	Flow
1 (350 deg)	very small
2 (400 deg)	none
3 (450 deg)	none
4 (500 deg)	none

Flow at 210 F for 4 hr

Sample No.	Flow
1 (350 deg)	very small
2 (400 deg)	none
3 (450 deg)	none
4 (500 deg)	none

Flow at 330 F for 2 hr

Sample No.	Flow
1 (350 deg)	4 in.
2 (400 deg)	2 in.
3 (450 deg)	3 in.
4 (500 deg)	none

Appendix C

TESTS ON COMPRESSION AND RECOVERY OF RUBBER-ASPHALT MIXTURE FOR THE PURPOSE OF DETERMINING ELASTICITY

On March 2, 1965, two additional test panels, Nos. 10 and 11, were placed consisting of 2 parts of 85-100 penetration grade asphalt and 1 part by weight of shredded scrap reclaimed rubber obtained from a local tire recapping shop.

When placing test panel No. 10, the temperature of the 85-100 penetration grade asphalt was 440 F just before mixing in the rubber, and 420 F for test panel No. 11.

Approximately 1 gal of rubber-asphalt mixture from each test panel was molded into a concrete cylinder can. After the mixture had cooled, the concrete cylinder can was removed and two test specimens 6 in. in diameter and 7 to 8 in. in height were obtained.

The test specimen molded from material from test panel No. 10 (on 7th St. south of Jefferson) was designated test specimen A, and the other test specimen molded with material from test panel No. 11 (east Washington and 26th Street) was designated test specimen B.

The test for elasticity (compression and recovery) was performed as follows. The height of each test specimen was determined. Each test specimen was compressed by applying a vertical load until a 2-in. displacement was observed. The load was removed in one test immediately, and after 5 min in another. The height of each test specimen was measured at intervals of immediately, 1 hr, and 12 hr, after the load had been removed. The recovery in height of each test specimen was determined as an indication of the elastic properties of the material. The recovery in inches, as a percentage of 2 in. (the length of displacement), was designated as the percent of recovery of the material.

The following results were obtained.

Test Specimen A

Mixing temperature of asphalt: 440 F.
Height of test specimen before loading: 7.75 in.
Height of test specimen with load: 5.75 in.
Height of test specimen 1 hr after immediate release of load: 7 in.
Percent recovery = 1.25/2 = 63 percent.
Height of test specimen before loading: 6 in.
Height of test specimen when loaded for 5 min: 4 in.
Percent recovery (1 hr after removing load) = 0.5/2 = 25 percent
Height of test specimen 12 hr after removing load: 5 in.
Percent recovery 12 hr after removing load = 1 in./2 in. = 50 percent.

Consistency Test

Cured for 24 hr at 140 F.
Observation: sticky and soft.

Test Specimen B

Mixing temperature of asphalt: 420 F.
Height of test specimen before loading: 7 in.
Height of test specimen with load: 5 in.
Height of test specimen 1 hr after immediate release of load: 6.75 in.
Percent recovery 1 hr after release of load = 1.75/2 in. = 88 percent.
Height of test specimen when loaded for 5 min: 4.75.
Height of test specimen 1 hr after release of load: 6 in.
Height of test specimen 12 hr after removing load: 6.50 in.
Percent recovery 1 hr after removing load applied for 5 min = 1 in./2 in. = 50 percent.
Percent recovery 12 hr after removing load applied for 5 min = 1.5 in./2 in. = 75 percent.

Consistency Test

Cured for 24 hr at 140 F.
Observation: gummy and firm.

Summary

It appears that the temperature of the 85-100 penetration grade asphalt just before mixing with the shredded reclaimed rubber is important. Test specimen B showed more resiliency and elasticity than did test specimen A indicating some damage to the rubber at the higher temperature.

Full-Depth Concrete Pavement Repairs on the Ohio Turnpike

FRANCIS C. STAIB, Superintendent of Maintenance, Ohio Turnpike Commission

Full-depth concrete patching is used to repair damaged sections of pavement on the Ohio Turnpike. Because of heavy traffic demands, methods have been developed to reduce the amount of time normally involved in accomplishing this type of repair.

One major time-saving results from sawing of the concrete to be replaced into 2- by 3-ft pieces which are removed intact; another from a rapid method for drilling holes in the vertical faces of the exposed concrete for placement of dowels. A program of rigid control of the concrete mix and the placing, finishing, and curing of the concrete results in high quality repairs that are expected to be permanent.

The work is scheduled so that all lanes are completely open to traffic during the week-end periods.

•THE OHIO Turnpike consists of 241 miles of four-lane roadway built in 62 contract sections by 27 different contractors during 1954 and 1955. The resultant variety of construction practices is reflected today in the extent and location of pavement repairs.

The maintenance department consists of two divisions, each headed by a superintendent and an assistant. Each division covers four maintenance sections. Each section is under a foreman and two crew chiefs directing from 16 to 20 workmen. A section clerk, mechanic and a building custodian are also employed. This work force is equipped and skilled in the performance of all maintenance activities necessary to maintain a modern controlled-access highway facility.

Except for a comparatively few spots, the pavement on the Ohio Turnpike is in relatively good condition. Where repairs to damaged and broken areas are necessary, full-depth concrete patches are used. Temporary measures, such as covering the broken areas with bituminous mixtures, do little toward restoring the pavement strength. When patching is needed, it is done at the earliest opportunity.

On heavily traveled highways, all lanes must be available to traffic as much of the time as possible. Full-depth repair work is carried out during the periods of least traffic. The actual repair work is scheduled for Monday, Tuesday, and Wednesday, thus leaving sufficient time for curing of the newly placed concrete before the reopening to traffic by 2:30 p.m. Friday. The method is simple and straightforward. Maintenance crews, once properly trained, do quality work economically without constant supervision.

The first experience of the Ohio Turnpike with full-depth patching was in 1957, using maintenance forces. A pavement breaker (drop hammer) was rented and maintenance equipment, on hand at that time, was used in the demolition work. A Gradall and an air compressor with pneumatic paving breaker were the only special equipment employed on this job. Considerable difficulty was experienced in removing this pavement because of the reinforcing steel. The job was started on a Monday with overtime that day, and continued for the full day of Tuesday when concrete was placed, finished by hand screeds, and the job completed by evening. As a result of this experience, con-

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siderable study was given to developing a better and more efficient means of doing this type work.

During 1960, it became apparent that an extensive patching program would have to be formulated. During that fall and the early spring of 1961, the necessary equipment was procured and during the spring a program was instituted to train crews to do this work. Through cooperation with the Portland Cement Association engineers, workmen were given careful preparatory training which helped correct misconceptions they might have had about concrete. The consultation provided at this critical time helped to get the work off to a good start.

DESIGN OF PATCH

The laying out of the patch involves consideration of the existing pavement conditions, the shape and dimensions of a patch, and its position in the pavement with respect to

Figure 1. Patch design and position in pavement with respect to joints and edges.

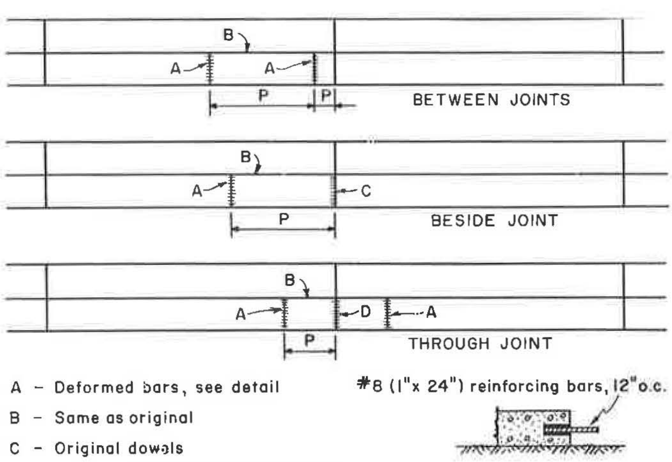

Figure 2. Ten-foot pavement repair.

joints and edges (Fig. 1). These items have a direct relation to the ability of the patch to stand up under traffic.

During the construction of the turnpike, it was decided that a minimum of 10 ft would be replaced when, because of some construction fault, it became necessary to replace a portion of the pavement. This policy was carried out in replacing defective pavement during the construction phase at several locations. An inspection of these patches showed no failures, and in fact, they were all in excellent condition. Following a study of these areas, we decided to continue using the minimum length of 10 ft for pavement repair (Fig. 2).

In constructing a full-depth patch it is necessary to determine the condition of the pavement on either side of the repair. To restore pavement smoothness, it is sometimes necessary to remove a considerable length of pavement. Adjacent settlements can be corrected by mudjacking before the pavement repair. It is generally a good practice to end a repair area no less than 10 ft from a joint or crack. If inspection of the crack indicates that it has not faulted, and the appearance is good, then the repair can end as close to it as necessary. Some repairs have ended approximately 6 ft from a crack. If the crack is open or faulted, the repair area is extended approximately 6 in. beyond the crack or the distance required to square the horizontal joint with existing pavement. In cases involving a failure at a joint, we have relocated a joint as much as 5 ft from its original location. Generally, these have been full-pavement width replacements.

REMOVING OLD PAVEMENT

Single Lane Width

After the size of the repair area has been determined, the pavement is sawed to a depth of approximately 6 in., to cut the reinforcing steel (Fig. 3). A cut is made on both ends of the repair area and approximately 2 in. in from, and parallel to, the centerline. A cut is made approximately 8 in. in from, and parallel to, one of the end cuts. The remaining cuts are made in such a manner as to result in pieces of approximately 2- by 3-ft.

Two- or More Lane Width

The sawing is the same as for a single lane width with the exception of the centerline cut. In this case, the centerline cut is made approximately 8 in. into the adjoining lane to provide space for setting forms.

Figure 3. Sawing of pavement to cut reinforcing steel.

Figure 4. Drilling of hole for lift pin which removes concrete piece.

Figure 5. Trench in shoulder beside slab to accommodate forms.

When sawing is completed, a hole $1\frac{1}{2}$ in. in diameter is drilled in the approximate center of each 2- by 3-ft piece for inserting the lift pin for removal of the piece (Fig. 4). A trench is excavated in the shoulder beside the slab to the depth of the slab and as long and wide as necessary to accommodate the forms (Fig. 5). For patches longer than 10 ft we have built short lengths of forms to add to the standard 10-ft forms. Pneumatic paving breakers are used to remove the 8-in. piece at one end of the repair area. After this piece is removed, the drop hammer is used to break out the first adjoining row of 2- by 3-ft pieces (Fig. 6.). Usually one hit of the hammer on the inside corner (where saw cuts cross) will break a piece loose. This is done one row at a time. Experience has shown a tendency to overbreak with the drop hammer. The more hits of the drop hammer the more spalls and chips to remove by hand. Sufficient material is blown out with an air blow pipe to provide room for inserting the lifting pin. The pin is inserted and the high-lift used to lift the piece of slab out (Fig. 7).

After the first row of pieces has been removed, the remaining spalls and chips are removed before removing the next

Figure 6. Drop hammer breaking out 2- by 3-ft concrete pieces.

Figure 7. High-lift used to lift out piece of slab.

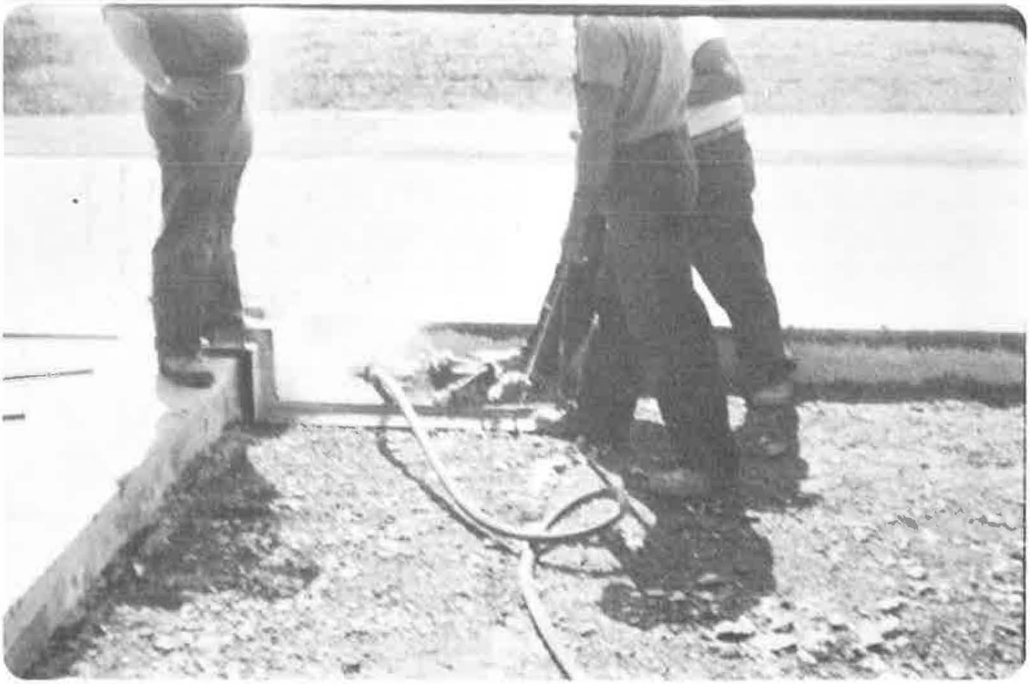

Figure 8. Removal of spalls and chips.

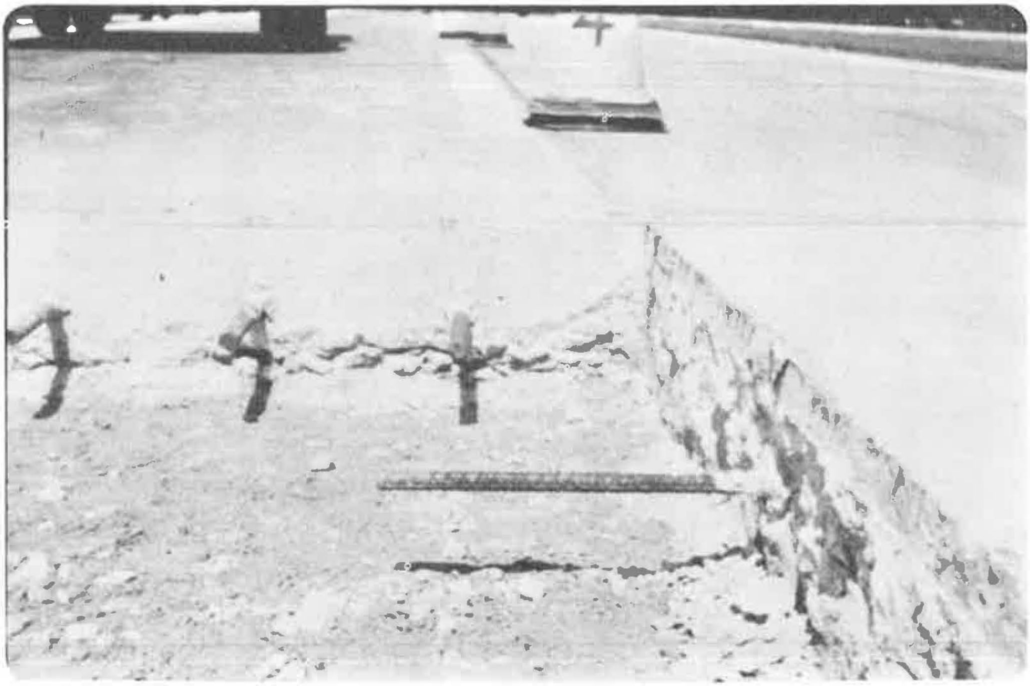

Figure 9. Replacement of joint, using steel bars to provide the working joint.

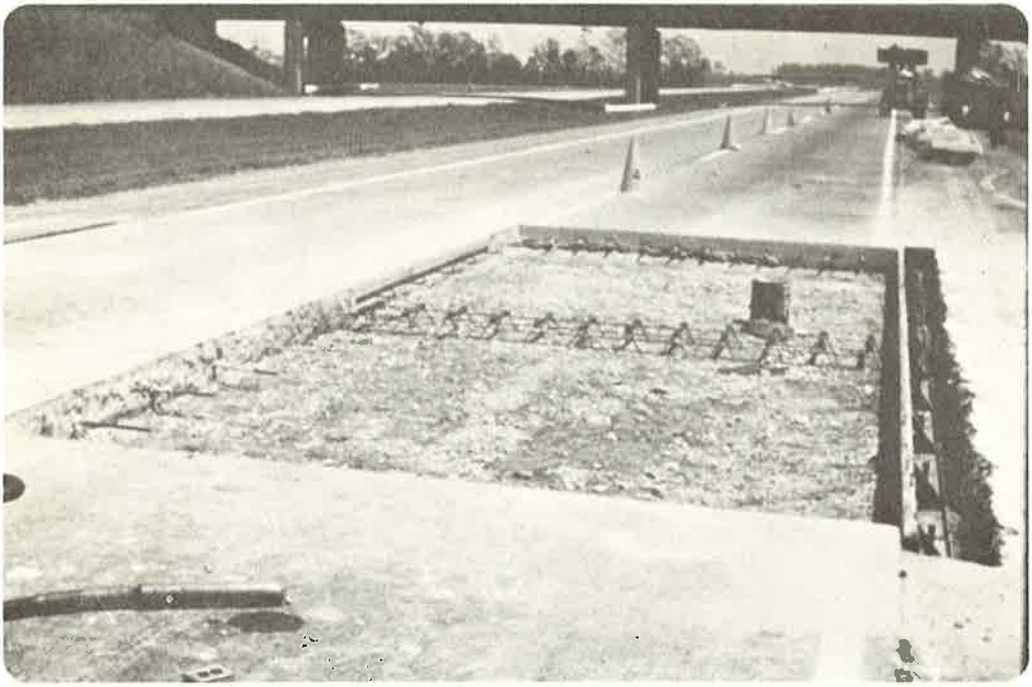

Figure 10. Standard hook bolt key joint.

row, to insure a minimum of subbase disturbance (Fig. 8). When all the old material is removed, $1\frac{1}{2}$-in. holes are drilled on 12-in. centers, 12 in. deep, into the vertical faces of the adjacent pavement at the transverse joints. In instances where the repair is an internal one (not at a contraction joint) No. 8 (1-in.) deformed bars are used for load transfer. Some locations involve the replacement of a joint where, for the most part, the full pavement width is replaced. The joint is replaced, using smooth, hot-rolled, steel bars $1\frac{1}{8}$in. in diameter to provide the working joint (Fig. 9). One and one-half inch holes are drilled 12 in. deep on 30-in. centers on the longitudinal joint. Where the repair is full pavement width, a standard hook bolt key joint is used (Fig. 10).

A quick setting mortar is used to grout the bars into the slabs. When smooth bars are used, they are grouted into the old pavement and the exposed ends greased before the new concrete is placed in the repair area. All dowels are 24 in. in length. After placement of the dowels, the form is set and the subbase dressed, as required. Seldom, if ever, is any material added to the subbase.

PLACING OF CONCRETE

Ready-mixed concrete, containing 6.5 sk of cement per cubic yard, with between 5 and 6 percent entrained air and No. 4 and No. 5 coarse aggregate (AASHO M 43), is ordered from a local batch plant (Fig. 11). The material is delivered to the job site dry. Since most of the delivery trucks do not carry sufficient water, it is provided at the job site to complete mixing. The concrete is mixed long enough to insure thorough blending and full generation of entrained air. A dry low-slump concrete is used (Fig. 12). Concrete is tested for sufficient air before being placed in the repair area and test beams are cast. No admixture other than the air-entraining agent is added to the concrete.

When the concrete is mixed and the air test completed, the concrete is placed in the repair area to a depth of approximately 7 in. (Fig. 13). A vibrator is used for consoli-

Figure 11. Placement of ready-mix concrete.

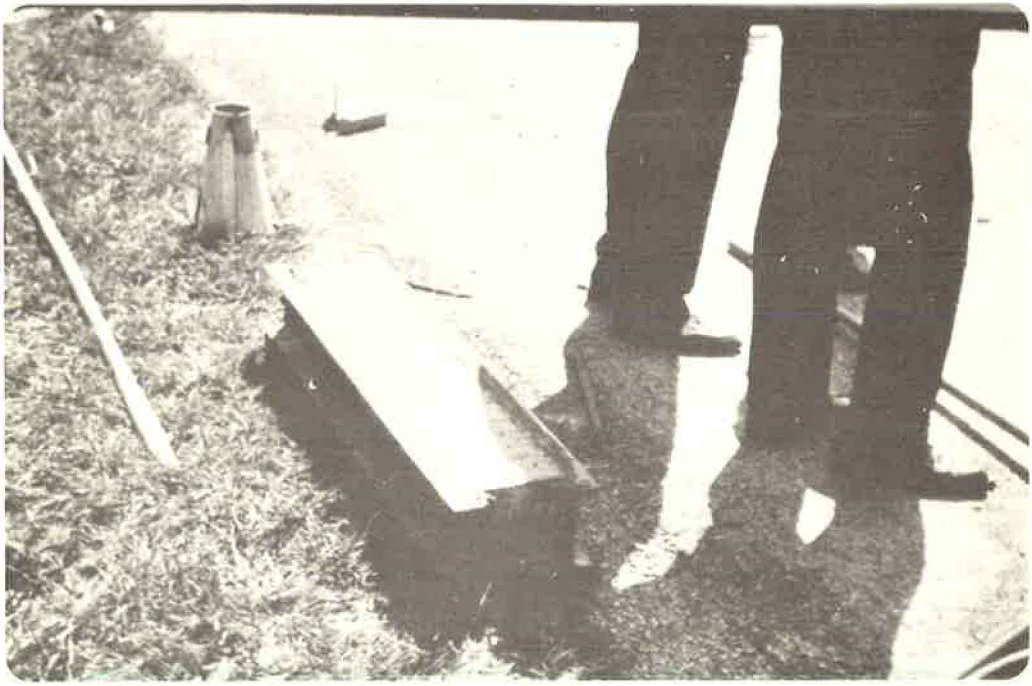

Figure 12. Dry, low-slump concrete.

Figure 13. Placing of concrete in repair area.

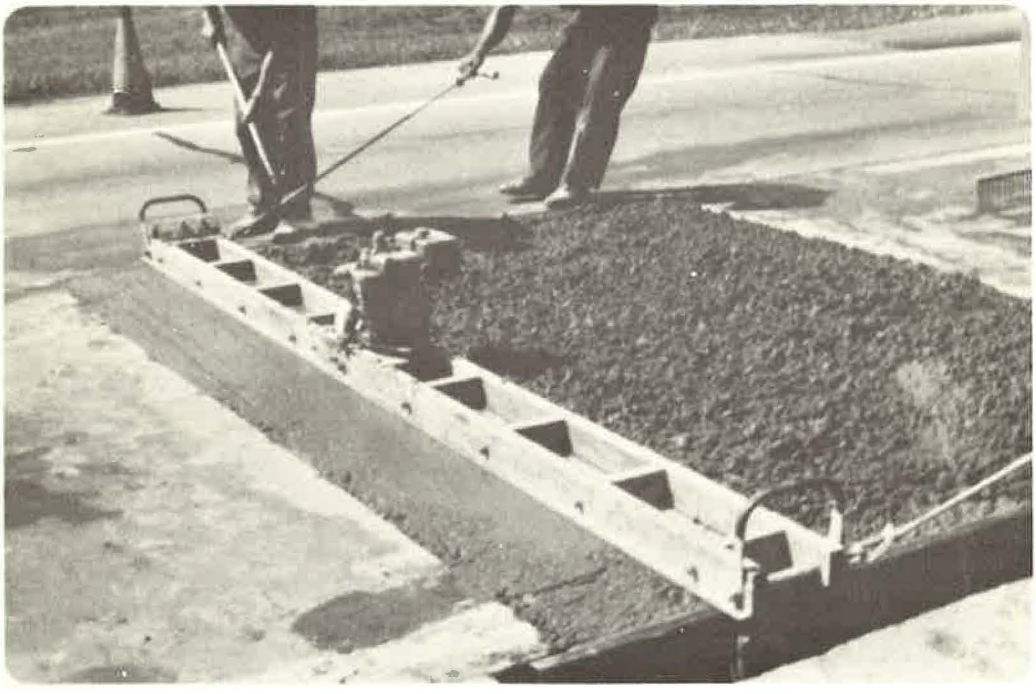

Figure 14. Addition of remaining concrete after placement of steel mesh.

Figure 15. Spraying of white-pigmented compound on concrete.

dation around the dowel bars. The steel mesh is placed and the remaining concrete added (Fig. 14). A vibrating screed is used to strike off the concrete, and several passes with the vibrator are usually sufficient. Hand work is kept to a minimum; generally, only a float and a straightedge are required to finish the surface. Burlap is used to texture the surface.

Immediately on completion of the finishing operation a white-pigmented compound is sprayed on (Fig. 15). The forms are usually removed after 24 hr. A trench drain is installed from the trench excavated for the forms to the adjacent shoulder and the form trench is back-filled and surfaced. Each maintenance division is equipped with a beam breaker. When the beams cast at the repair area indicate 500-lb flexural strength, the pavement is reopened to traffic.

COST AND TIME INVOLVED

Sawing is generally completed the day before the removal of the concrete to be repaired. Sawing time for the average size repair is 2 to 3 hr. Removing the old pavement and placing forms and dowels generally requires between 1½ to 2 hr with another hour required to place and finish the concrete.

The cost varies in relationship to the material cost and the size of the area being repaired. The average cost of labor and material, in place, is $12.00 per sq yd.

EQUIPMENT USED

The equipment used includes the following items:

- Trucks, as needed, to remove material and equipment;
- Front-end loader;
- Concrete saw (Clipper Model 300);
- Water tank, 900-gal capacity;
- Drop hammer (Henry Model 810, mounted on an IHC Model 300 tractor);

Air compressor;
Jack hammer and rock drill;
Device to hold rock drill for drilling horizontal holes;
Lift pins;
Vibrating screed (Thor Model FS);
Concrete vibrator (Wyco square head No. 991-G 7); and
Miscellaneous small tools such as wheelbarrows, brooms, trowels, edgers.

CONCLUSIONS

Enough of these repairs have been made on the Ohio Turnpike to demonstrate that the principle is sound. Rigid controls are set up for the use of this repair method, and when followed carefully, the end results are good and the repairs are expected to be permanent.

NOTE: A color movie, shown at the 45th Annual Meeting of the Highway Research Board, has been developed by the Portland Cement Association to illustrate in detail the procedures developed.

Restoration of Joint and Spall Failures on PCC Pavement

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C. E. RICE, Research Engineer, North Dakota State Highway Department

•IN April 1961, the North Dakota State Highway Department let a contract for portland cement concrete pavement on highway US 83 through the city of Minot. This contract called for a 4-lane divided highway with both depressed and raised medians. The PCC pavement was 9 in. thick with standard 6-by 12-in. wire mesh reinforcement. Slab lengths were 61 ft 6 in. with doweled contraction joints. The contract was completed in the fall of 1961. Before the pavement had a chance to completely cure, and in less than 30 days after completion, freezing weather and icy conditions required that the city use a mixture of sand and salt on the pavement to reduce traffic hazards. The pavement was constructed of air-entrained concrete but we believe the curing period was so curtailed by the freezing temperatures that the pavement's susceptibility to resist salt action did not have a chance to really become effective.

In the spring of 1962, areas of the pavement began to scale. By midsummer joints began to spall as the pavement expansion stressed the edges of the joints. Spalled areas showed evidence of salt concentrations in the plane of the spall indicating that the early application of salt had had a damaging effect on the pavement.

During 1962 and 1963 the spalls became larger, deeper, and more numerous. Joint seals had failed entirely and were allowing incompressible material to enter the joints. This added to the stresses on the pavement joint edges as the slabs expanded in hot weather (Fig. 1). Two years after the pavement was constructed, some joints had spalled so deeply that the dowel bars were visible. Some spalled areas 1 to 2 ft wide extended the full width of the street and were becoming traffic hazards (Fig. 2).

COURSE OF ACTION

Our research department was enlisted to determine a course of action. After studying available literature and making a physical inventory, it was decided that all transverse joints on the project would be resawed and resealed. All spalled areas would be restored to their original condition insofar as possible.

The decision was made to resaw all joints to a width of $\frac{5}{8}$ in. and seal with $1\frac{1}{4}$-by $1\frac{3}{4}$-in. neoprene compression seal. (It was proved later that the joints had to be sawed to $\frac{21}{32}$ in.)

The physical inventory showed that the spalled areas varied in size from small chip-like areas to areas 2 ft wide, 4 to $4\frac{1}{2}$ in. deep with some areas extending the entire width of the pavement. It was decided to separate the spalled areas into two categories as to type of repair. Spalled areas up to 36 sq in. were to be patched with an epoxy mortar and all spalled areas over 36 sq in. were to be patched with a portland cement mortar (Fig. 3). It was estimated there would be 127 sq ft of epoxy patching and 3,429 sq ft of portland cement patching.

The epoxy patching material, Thiokol or equivalent was to be mixed and placed in accordance with the manufacturers, recommendations.

The portland cement patching material was to be 1: 2 mortar mix with no more than 4 gal of water per sack of cement.

Paper sponsored by Committee on Maintenance of Portland Cement Concrete Pavements and presented at the 45th Annual Meeting.

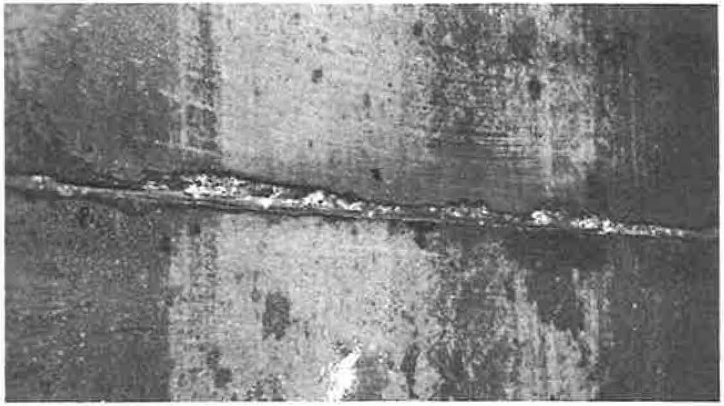

Figure 1. Slab expansion in hot weather.

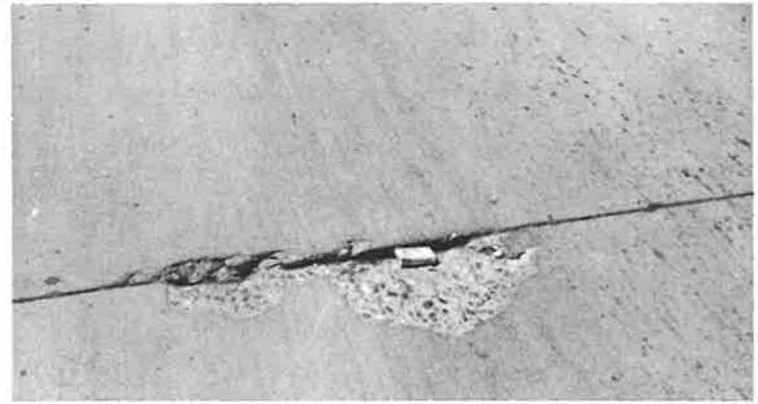

Figure 2. Spalled area at joint 2 years after construction; notice cigarette pack for reference.

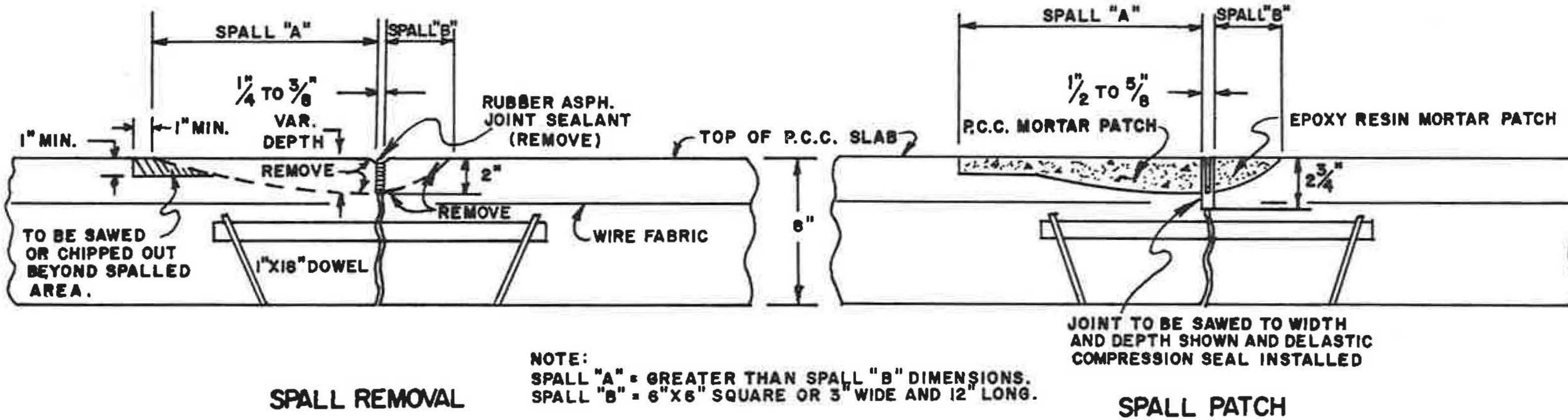

Figure 3. Spall and joint repair detail.

PROJECT AND CONSTRUCTION METHODS

In August 1964, a contract was let for this restoration work at a contract price of $41,700.00. The engineer's estimate of the cost of the work was $28,000.00. After awarding contract, the first step in the work was to locate and outline all unsound, or loose concrete in the area of each joint. The loose portions of concrete were easily detected by a hollow sound when rapped with a hammer. The outline of each area was then painted (Fig. 4) on the pavement. The joints were then sawed to the dimensions required for the preformed neoprene seal. A 1-in. deep saw cut was made following the painted outline of the spalled areas.

All unsound or loose concrete within each saw cut area was removed to a minimum depth of 1 in. Jack hammers were used to remove this concrete in the larger areas. A small chipping hammer was used in the small, shallow areas (Fig. 5). The resulting

Figure 4. Painting outline of unsound concrete.

Figure 5. Using a small chipping hammer in small, shallow areas.

areas to be patched had straight vertical sides, but the bottom of each was left very rough to aid in obtaining a good bond between the patch and the old concrete. Each area was then cleaned of all dust and loose particles of concrete by brooming and by a small vacuum cleaner. No attempt was made as yet to clean the joints. A 25 percent solution of muriatic acid etch was applied to each patch area with a stiff brush and allowed to remain until all acid action appeared to cease. This takes only a few moments. The acid was flushed out with water. A few of the patched areas have since shown evidence of a poor bond. When the poorly bonded material was later removed it was found that the acid residue had not been washed out well; therefore, if an acid etch is used, all traces of the acid must be thoroughly removed by washing, preferably under pressure. (The same contractor is now working on an air base just north of the city and is sand blasting after acid etch.) The areas to be patched were allowed to dry thoroughly before patching.

The epoxy patching mortar was composed of Presstite No. 1190 concrete adhesive in combination with Ottawa sand. One part epoxy to 3 to 5 parts sand was used as trial mixes. The mixture with 5 parts sand to 1 part epoxy was favored because it cured faster. The mortar was mixed in batches just large enough for a two-man crew to place and to finish the batch before it became unworkable.

A coat of epoxy adhesive was carefully brushed onto each face of the exposed area (Fig. 6). Then each depression was filled with mortar. The mortar was tamped into place. The surface was troweled and a light dusting of portland cement applied so that the surface color blended with the surrounding pavement. Before placing a patch, a wooden form was placed in the joint to form a wall of the joint (Fig. 7). The forms were not removed for 72 hours or more until the patches cured enough to allow traffic over them.

Several methods were used in an attempt to hasten the hardening or curing process but none was successful. In one, insulation was placed over the patches to retain heat. Various means of heating the patches were also tried including heated sand in the mortar. The 72-hr time for curing was found necessary.

The portland cement mortar consisted of 1 part Type III portland cement to 2 parts concrete sand with a water content of 3 gal per sack of cement. Air-entrainment was added to maintain an air content of from 8 to 10 percent. Calcium chloride at a rate of

Figure 6. Applying brush coat of epoxy adhesive.

Figure 7. Placing wooden form.

one percent of the cement was added to the mortar as an accelerator. Three-day compressive strengths of this mortar exceeded 3,000 psi. Prior to placing these patches, forms were placed in the line of the joint to form the joint wall. The patch area was painted with a portland cement grout, consisting of 1 part cement to 1 part sand. The grout was applied with a stiff brush to be sure it was scrubbed into every indentation of the area. The patching mortar was then tamped into the patch (Fig. 8), finished to blend in with the surrounding pavement surface, and sprayed with a white, liquid curing agent. All forms were removed in 72 hours after the patch was placed. The area was then opened to traffic.

Figure 8. Tamping mortar into patch.

Before installing the neoprene seal, all joints were re-sawn and cleaned of all dust and debris. Each joint was checked to be sure it was dry. The seal was installed according to the manufacturer's instructions.

CONCLUSION

There had been some doubt that the thinner, smaller epoxy patches would hold. As of this time only three very small epoxy patches have failed in bond. These failed due to the action of a steel-wheeled roller having been driven over them. The neoprene seals have functioned perfectly through the cold weather of the winter of 1964-1965 which had temperatures as low as -31 F. At this time we are highly optimistic that the project was completely successful and we believe time will support our optimism.

When the project was completed, there were 1, 639 (232. 7 sq ft) and 4, 844. 94 sq ft of mortar patches.

Detection of Concrete Deterioration Under Asphalt Overlays by Microseismic Refraction

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Nondestructive testing of structural materials, particularly concrete, has received increasing attention from highway engineers and other members of the civil engineering profession during the last decade. Application of nondestructive test methods to existing, and aging, highways and structures offers apparent economic advantages over methods that require defacement and repair, or interruption of normal service. A special problem in the monitoring of concrete deterioration is presented by concrete slabs overlain by asphaltic surface courses. Such concrete is not visible for inspection. On bridge decks and in similar exposed locations, considerable slab deterioration can occur without producing visible evidence at the asphalt surface. Until recently, test procedures have been limited to core sampling, or removal of asphalt for inspection. Both are costly. A method is presented for determination of asphalt thickness and estimation of concrete structural soundness by measurement of the direct and refracted microseismic wave velocities produced by low-energy hammer impacts on the surface of the material. Thickness is determined by solution of the classical seismic refraction formula. Concrete condition is estimated by comparison of the measured velocity in the concrete with standard velocity of concrete having known properties. Laboratory-type instrumentation used in evaluation of the test method and field-type instruments developed for general application are described. The theoretical basis for the method and the detailed procedure for field tests are given in the Appendix.

•HISTORICALLY, test methods for the determination of concrete quality have been destructive when carried out on specimens. The disadvantages of destructive methods for the investigation of in-service structures are obvious. Considerable time and expensive facilities are required for obtaining core samples and testing of specimens in the laboratory. Interruption of service on the structure and repair of defacement caused by specimen procurement also add to costs. Furthermore, only a single test result is yielded by destructive methods for each specimen, so that a large number of specimens are necessary to give statistically representative data over large structural areas.

These factors have spurred intensive study and increasing application of nondestructive methods for testing of concrete. One example is the resonant frequency method developed for the determination of dynamic modulus of elasticity. This has contributed greatly to the study of deterioration of concrete during freezing and thawing cycles.

Other nondestructive methods are based on the measurement of elastic (seismic) wave velocity in concrete. Wave velocity methods have wide applicability because they

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may be applied to in-service structures without damage or defacement and with little interruption of service. A number of test procedures, using a variety of measuring apparatus, have been developed in the United States (1), Canada (2), England (3), France (4), and Denmark (5). Work is continuing, in these countries and others, on studies of wave velocity in concrete and similar materials and on the correlation of wave velocity with the physical properties of the material homogeneity, such as compressive strength, dynamic elastic moduli, aging processes, and degree of deterioration with age in various environments.

As the store of empirical correlative data increases, the application of wave velocity measurements to increasingly varied field testing problems will become both possible and economically rewarding.

One such problem, of special interest to highway and bridge maintenance engineers, is the monitoring of the structural condition of concrete pavements and deck slabs supporting overlays of asphaltic material. Because of its asphalt cover, the surface of such concrete is not visible for routine inspection. It has been found that considerable, even dangerous, deterioration of the concrete can occur beneath the asphalt in structures exposed to severe environmental conditions before evidence of the damage becomes visible on the asphalt surface. Asphalt is a relatively plastic material, particularly when warm, and it can undergo a certain amount of deformation in compliance with loss of underlying support without exhibiting surface cracking or weathering. Thus it masks the unsatisfactory condition of the underlying concrete. Even with the asphalt-concrete interface bond broken and the concrete surface pitted, the asphalt can span the damaged area for some time when it is of the thickness normally used for heavy-traffic surface courses. In many cases, the asphalt thickness is not accurately known because of heavy wear over protracted periods or absence of original construction information.

A method for the rapid and economical determination of asphalt thickness and estimation of the structural condition of the underlying concrete over large areas therefore has obvious value. In the summer of 1964, the Port of New York Authority retained the consulting engineering firm of Joseph M. Phelps & Associates to develop the basis for such a method, and to assist the Port Authority's Engineer of Materials Research in evaluation of the method's feasibility on an existing structure. Laboratory-type equipment suitable for the evaluation program was developed in the laboratories of DynaMetric, Inc., Pasadena, Calif.

Engineers from the three organizations then conducted tests on portions of the asphalt-surface concrete deck of the Outerbridge Crossing, a large steel cantilever-truss bridge spanning Arthur Kill, between Perth Amboy, N.J., and Staten Island, N.Y. This structure was chosen for evaluation purposes because it was undergoing extensive repairs to and widening of the concrete deck system, both on the main span and on the two girder approaches. Tests were conducted at locations ahead of the field work, so that visual examination could later be made at the same spots. Tests were also made on new concrete decks with freshly laid asphalt surfacing. In addition, core samples of the old materials were taken at the test locations and evaluated in the strength of materials laboratory of the Port Authority.

This paper presents a brief discussion of the basis for the test method; a description of the laboratory-type equipment used in evaluation; an outline of the test procedure; a discussion of the correlative examinations used in evaluation of the method; and a brief description of new portable, self-powered instruments and accessories being developed for general application of the test method. The Appendix gives the theoretical basis for microseismic refraction and a selected list of references.

BASIS FOR TEST METHOD

Refraction seismology has been used by exploration geophysicists for many years. Its principal purpose is the detection of subsurface materials, such as rock or shale strata, and the determination of their depths below ground surface. For deep exploration such as that for potential sources of petroleum, seismic waves are created in the earth by high-explosive detonation, and their arrivals at a series of geophones at or near the surface are electrically recorded. Travel times of the waves and the velocities indicated thereby are determined by examination of the records. More recently, seismic refraction has been employed for the shallower depths of interest to engineering geologists and foundation engineers. For this application, elaborate recording equipment has been replaced by portable timers reading directly in milliseconds (msec), and the seismic waves are created by striking the ground (or a steel plate or ball resting on the ground) with a sledge hammer, which is wired to start the timer counting operation. Survey lines for shallow exploration of this type are usually 100 to 200 ft long. A geophone array is not used—only a single geophone. The hammer makes a series of impacts at successive stations along the line to produce a series of travel times needed to plot the travel-time graph from which depth to subsurface materials and velocities in those materials can be calculated.

Microseismic refraction essentially takes the process of miniaturization one step lower. As applied to the determination of asphalt thickness and concrete investigation, the survey line becomes only some 3 ft long. Blows from a small hammer are used, at impact stations only some 3 in. apart. Because of the very short distances involved, and the high seismic velocities in asphalt and concrete, the travel times must be measured in microseconds (μsec). And because of the small amplitudes and much higher frequencies of the arriving waves, the detection geophone must be replaced by a transducer of higher capability. Basic principles of the method, however, remain the same. They are covered in all standard texts on exploration seismology and are briefly discussed in the Appendix. The necessary formula for depth calculation is given, and determination of the quantities used in the formula is explained.

Two of the quantities used in the dpeth formula are the velocities of the seismic wave in the asphalt and in the concrete. Velocity in the concrete is of special interest in the present application because it serves as a basis for estimation of the structural condition of the concrete. Strong, homogeneous concrete will exhibit a seismic velocity much higher than that shown by the asphalt. As representative values, velocity in good quality concrete will range from 11,000 to 14,000 ft/sec; whereas velocities in good quality, unfractured asphalt will be in the 8,000- to 10,000-ft/sec range. Velocities in damaged or weathered concrete will rarely be as high as those in good asphalt and may be much lower, depending on degree of deterioration.

The velocity characteristics of the two materials thus make it possible to detect areas where concrete deterioration has occurred and to gain a fairly accurate determination of the depth of weathering or other damage. For example, on a slab covered by a nominal 3-in. thickness of asphalt surfacing, areas may be found where the calculated depth to concrete is, for example, 6 in., indicating that the concrete has so deteriorated that its velocity is as low as, or lower than, that of asphalt for a 3-in. depth in the concrete slab. This would be enough to render top reinforcement, if any, ineffective due to loss of bond strength.

In other cases, where concrete deterioration is just beginning or has not deeply penetrated the slab, the surface concrete may exhibit a velocity higher than that of the asphalt, but not as high as the strong concrete deeper in the slab. In such cases, the asphalt/concrete interface may appear as a first horizon (see Appendix) and the surface of the strong concrete as a second horizon. Any material at depth, having a seismic velocity higher than the velocities in materials above it, will appear in the refraction survey. Materials having velocity lower than the velocities of overlying materials will not appear on the travel-time graph, but may affect the so-called breaks in the graph.

LABORATORY-TYPE EQUIPMENT FOR TEST EVALUATION

Initial analysis of the problems involved in the miniaturization of the seismic refraction method for application to high-velocity materials such as asphalt and concrete, showed that it would be necessary to measure travel times in the range from 25 to 500 μsec with an accuracy of ± 5 μsec over the shortest base lines. Because of the very short time differentials involved, the rise time of the arriving seismic signal must be steep, corresponding to frequencies in the 2-kc range, in order to get repeatable time detection. The conclusion that ordinary geophones, which rapidly lose sensitivity at frequencies above 500 cps, could not be used, was confirmed in the laboratory. Studies also showed that use of an impact type shock switch to start time measurement would be unreliable due to unacceptable nonuniform time delay in the triggering circuit.

Since the frequency of the elastic wave generated by an impact is a function of the resonant frequency of the spring-mass system formed by interaction of the impacting device with the elastic boundary of the deformed surface material at the impact point, it was felt that design of the impacting system should incorporate a coupler mass of the proper size and shape to generate an efficient, high-frequency shock effect.

Impacting Device

Mereu, Uffen and Beck (6) have shown that conversion of impact energy into seismic energy can be effected by proper selection of coupler mass and shape. Calculations based on coupler theory indicated that use of a small hardened-steel sphere as a coupler, struck sharply by a light steel hammer, would produce sufficient energy of the required frequency range. The mass of the coupler must be such that the product of its mass and coefficient of restitution are of the same order of magnitude as the mass of the falling weight, in order to minimize production of complex wave forms. A number of couplers (rods, cones, flat plates) were evaluated in the laboratory. Results showed the sphere most efficient, as theory indicated.

Triggering Device

The impact device, coupler and hammer, was used as a simple contact to trigger the operation of the timing instrument. This was accomplished by the design of the coupler. A brass rod was inserted through diamond-drilled holes in two ¾-in. diameter steel ball bearings. One ball was secured to each end of the rod by end nuts and lock nuts. One end of the rod was connected electrically to the timing trigger circuit. The hammer was also wired to the trigger circuit; thus contact between hammer and either of the balls on impact closed the circuit at the instant of impact and initiated the timing sweep. The ball not in use served as a handle. The device is shown in operation in Figure 1.

Transducer

A review of several available types of high frequency-response accelerometers indicated that one suitable for the work would be a Model 2217 accelerometer manufactured by Endevco Corp. This transducer has a resonant core frequency of 30 kc, a flat frequency response of 2 cps to 6 kc when driving into 1000-megohm impedance, and a flat frequency response from 20 cps to 6 kc when driving into 100 megohm. Its use at 100

Figure 1. Evaluation of microseismic refraction method: (a) observing travel times on portable oscilloscope triggered by impacts; (b) refraction survey line, impact device and accelerometer.

megohm requires an amplifier and a power supply, also manufactured by Endevco. These instruments were procured for the investigation and used in laboratory development and field evaluation.

Timing Instrument

Because of the time and cost involved in the design and production of any direct-reading digital counter, it was decided not to engage in such development before evaluation of the test method. Laboratory work confirmed that the arriving seismic wave generated a signal that could be seen and measured on a cathode-ray tube oscilloscope with sufficient accuracy, although use of an oscilloscope is not satisfactory for regular field testing because of operator fatigue. A Tektronix, Type 321, self-powered portable oscilloscope was used for both laboratory studies and field tests. This is a small but precise oscilloscope that can be operated from an a-c line, from an external d-c supply, or from its own self-contained and rechargeable batteries. It weighs 16 pounds. It provides Miller-integrator sweeps from 0.5 μsec per division to 0.5 sec per division in 19 calibrated steps. It can be externally triggered by from 2 to 50 volts with an input impedance of 10 pf, paralleled by 100 K, plus or minus trigger slope, a-c or d-c coupled. The scope display area is marked in 6 vertical and 10 horizontal divisions, each $\frac{1}{4}$ in. wide. The sweep rate found most useful in the microseismic measurements was 50 μsec per division. The cathode-ray tube of this oscilloscope has a long retention time that

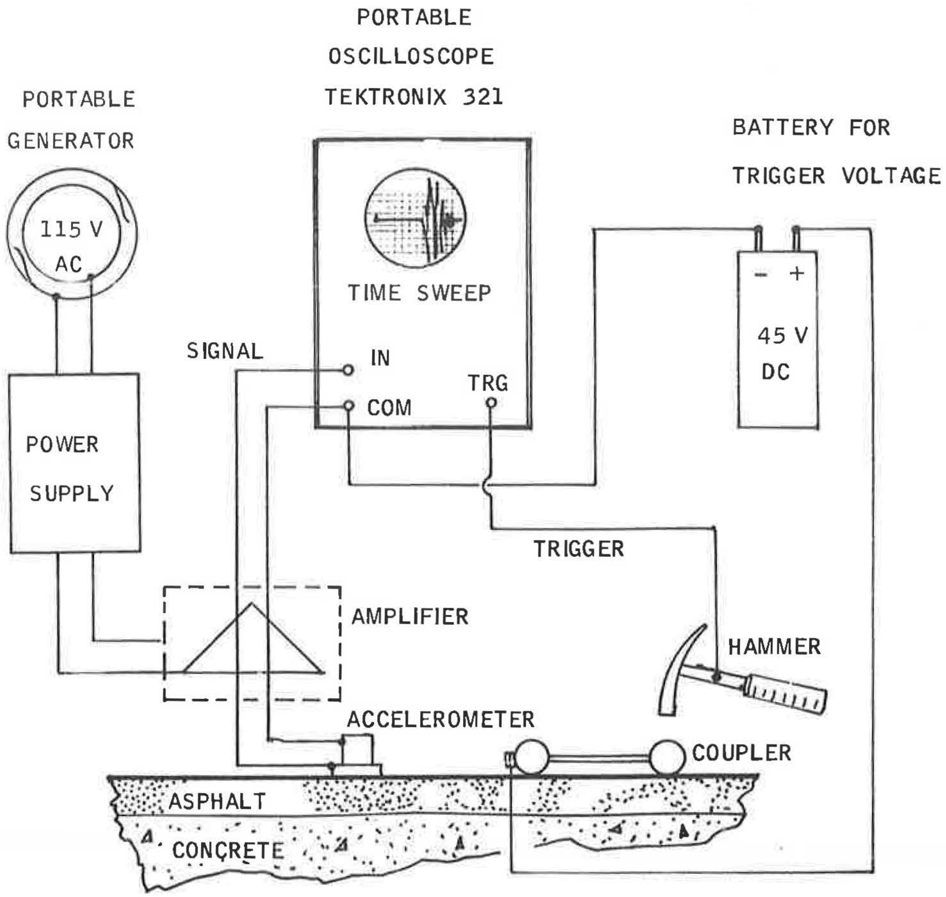

Figure 2. Functional diagram of laboratory-type equipment.

permits visual retention of the extremely fast trace and greatly assists in reading of time intervals.

During part of the field work, a large laboratory oscilloscope, powered by a portable generator was used. The large scope gave equally good results, but was more difficult to read. There may be some question as to the stability of its time base when powered by a portable generator, whereas the bistable Schmitt trigger multivibrator of the portable oscilloscope is independent of any a-c supply or line fluctuations. A block diagram of the system components is shown in Figure 2.

TEST PROCEDURE

The microseismic refraction method for determination of depth to sound concrete comprises four distinct steps:

1. Lay out a refraction survey line on the surface of the pavement. The line, as used in the evaluation of the method, was normally 3 ft long. (It was laid out at a 45-deg angle with the centerline of roadways, since steel reinforcement existed in the slabs in the normal and transverse directions. Steel carries sound waves at velocities of 16,000 to 17,000 ft/sec and so would mask the effect of the concrete refraction wave if the survey line paralleled the steel. By running at 45 deg, the sound in the steel must travel a longer path by the factor of $\sqrt{2}$, hence does not short-cut the concrete refracted wave.) Generate seismic waves in the pavement by hammer blows on a small steel sphere, at impact stations spaced at equal intervals (usually 3 in. apart) successively farther from a detector at the zero end of the line. Measure the travel time (in μsec) of the seismic wave from each impact station to the detector. Use repeated hammer blows at each station to get repeatable confirmation of the time reading, and record the lowest repeatable travel time.

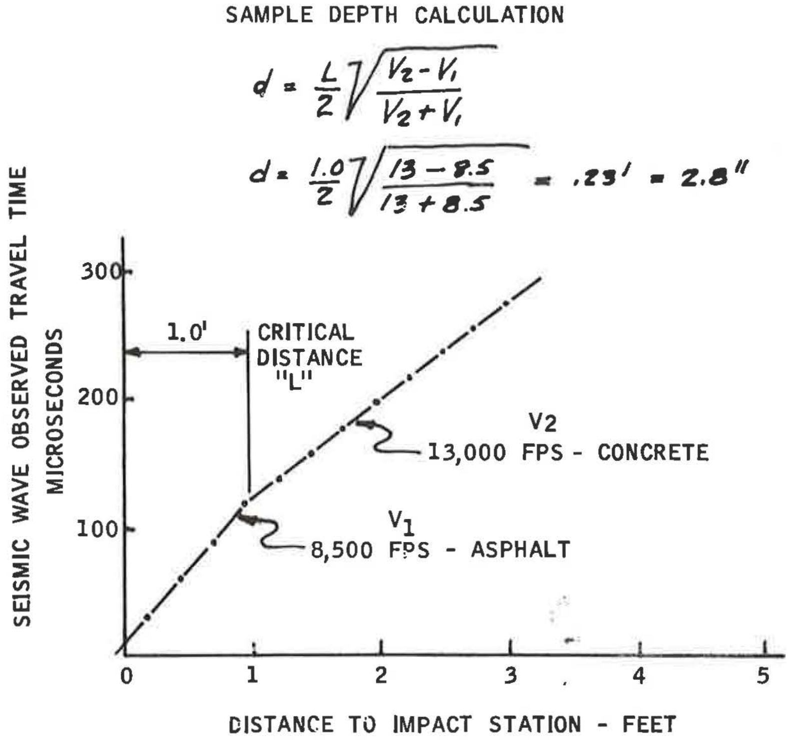

Figure 3. Typical travel-time graph.

2. Using the observed travel times and the known distances to successive impact stations, plot a travel-time graph, with time as the ordinate and distance as the abscissa (Fig. 3).

3. Observe the point on the travel-time graph at which the line changes slope. From all stations more distant than this point, the observed arriving waves have traveled through the concrete by refraction, rather than directly through the asphalt. Obtain the seismic velocity in the asphalt from the slope of the first portion of the graph, and the seismic velocity in the concrete from the slope of the second portion of the graph. Calculate the depth to sound concrete (asphalt thickness, or asphalt plus bad concrete thickness) using the measured velocities and the distance to the point at which the graph changes slope (critical distance—see Appendix).

4. Estimate the structural condition of the concrete by comparing the measured seismic velocity in the concrete with velocities known to be exhibited by good quality concrete of that type. For correlation purposes, tests can be run on known good concrete of the same type in the same structure at locations not covered by asphalt. Additional information about the nature of the surface of the concrete and the asphalt/concrete interface may also be obtained by special interpretation of aberrations in the travel-time graph.

The refraction method is not necessarily limited to cases of concrete covered by asphalt overlays. The velocity determination works equally well on exposed concrete. If surface deterioration of the concrete is visible, the depth of such damage can be determined by refraction. Recently placed concrete can also be checked for velocity increase correlative with increase of compressive strength and homogeneity during curing. One powerful advantage of the refraction method is the determination of velocity from the slope of a graph plotted through several stations, which eliminates the effect of any system delay in the timing devices and gives true velocity both in surface materials and in the underlying materials if such exist.

FIELD EVALUATION OF THE TEST METHOD

In May 1964, field tests of the microseismic refraction method were conducted, using the laboratory-type equipment, on the concrete deck system of the Outerbridge Crossing. Preliminary observations were made at a location on the New Jersey approach span where removal of old asphalt and concrete was in progress and structural conditions could be inspected visually. Velocity was measured on the surface of exposed, old concrete which appeared to be in good condition, and was found to be 11,500 ft/sec. Velocity was measured on the surface of pieces of old asphalt which had been removed from the deck, but did not appear to be cracked. The asphalt was isolated from the concrete deck by a cloth pad to avoid refracted concrete transmission. Asphalt average thickness was 2.5 in. The asphalt velocities obtained were from 6,500 to 7,000 ft/sec. These velocities are lower than those later found for undisturbed traffic-compacted asphalt well bonded to good concrete.

A refraction survey was then made on the surface of old asphalt in an area where considerable concrete slab deterioration was evident from chip tests and visual inspection. No velocity break was obtained in the travel-time graph, indicating that the concrete velocity was no higher than that in asphalt, in this case 8,500 ft/sec.

The test location was then moved to an area where a new portion of the deck had been constructed and asphalt-surfaced about one month previously, and not yet subjected to traffic loads. Three separate refraction surveys at this location indicated an asphalt velocity of about 9,500 ft/sec, an asphalt thickness of 2 to 2¼ in. (which accorded with the known construction), and concrete velocities of about 13,000 ft/sec.

Since the results of the preliminary tests were favorable and correlated with available visual information (indicating the feasibility of the method), a systematic survey of a large portion of the main span deck system was conducted. The survey included refraction determinations at a total of 65 points on the westbound lanes over the entire suspended span and portions of the east and west cantilever spans. The outer westbound lane in this section was scheduled for partial removal and widening. It was desired to check on the condition of the center westbound lane so that any necessary repairs could be accomplished at the same time.

The 65 point determinations, using laboratory-type equipment, required the equivalent of two working days for two men. Of the total number of determinations, 45 were made in the center traffic lane, and of these, severe concrete deterioration was evidenced at 7 points, and slight surface weathering was indicated at 3 points. The remaining 35 points gave data indicative of good or excellent concrete.

Twenty determinations were made at points in the outside lane (which was scheduled for partial removal) adjacent to the main steel trusses. Of these, 12 points gave indication of severe deterioration of concrete, in three cases for the full depth of the slab. In 9 of these cases, the overlying asphalt appeared in satisfactory condition to visual inspection. Five other points indicated moderate or slight weathering to partial slab depth, and only 3 points indicated sound concrete for the full depth of the slab. Of the 20 points, asphalt appeared good at 13 locations, fair at 6 locations, and poor at only 1 location. The inferior condition of the outer lane concrete compared to that of the center lane is assumed to result from the proximity of the outer lane slab joints to the curb, the sidewalk, and the constantly moving steel trusses. The most severe deterioration was observed at points closely adjacent to heavy supporting steel members.

CORRELATIVE EXAMINATIONS

Shortly after completion of the field evaluation studies, the asphalt surfacing and portions of the concrete slabs were removed from the outer portion of the westbound lane to permit construction of a new and wider roadway. Visual inspection at the locations of the 20 refraction test points in this lane gave good correlation with predicted conditions. At points where the refraction method predicted severe deterioration of the concrete, the slab was found to be essentially destroyed. At points where surface weathering was predicted, the slab was found to be pitted, with cement bond broken between aggregate particles, and the asphalt/cement interface broken and containing voids and loose aggregate. At points where good concrete was predicted, the weathering process had not occurred.

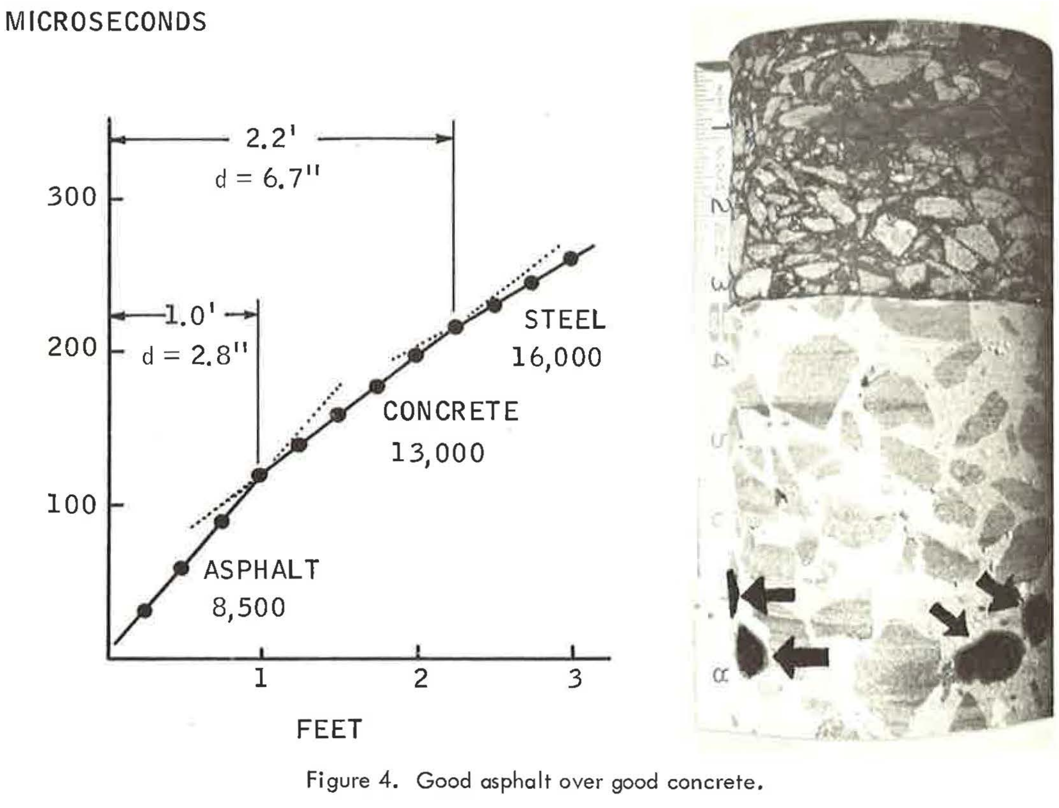

Figure 4. Good asphalt over good concrete.

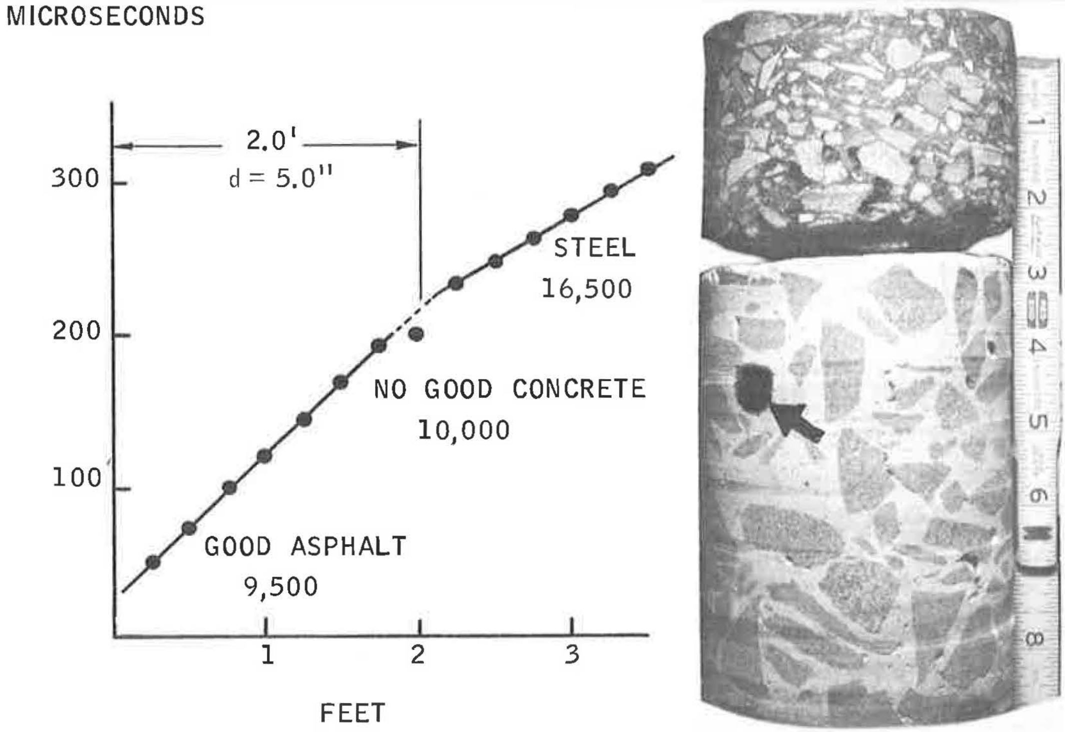

Figure 5. Some concrete deterioration beginning.

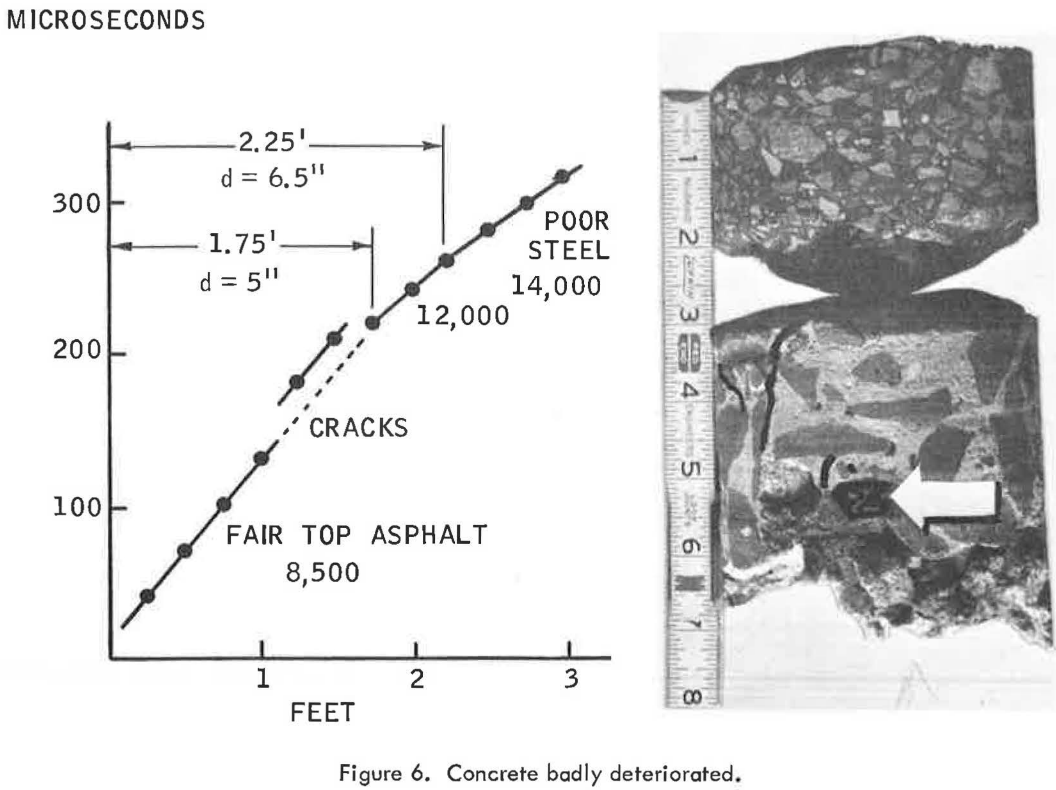

Figure 6. Concrete badly deteriorated.

A number of core samples were taken in the center westbound lane, at the location of some of the 45 refraction test points made in that lane. These core samples were photographed and examined in the laboratory. Again, the results gave good correlation with the predictions of the refraction tests. Typical core samples are shown in Figures 4, 5 and 6, together with their corresponding travel-time graphs and the predictions based thereon.

In summary, evaluation of the test method and correlative examinations indicate that the microseismic refraction method is suitable for the routine monitoring of concrete base-slab conditions over large portions of asphalt-overlaid structures and for determining asphalt thickness without disturbance of the material. Economical use of the method for routine testing presupposes the existence of suitable high-speed, direct-reading timing instruments that permit rapid procurement of field data.

FIELD EQUIPMENT FOR MICROSEISMIC REFRACTION TESTING

After successful demonstration of the feasibility of the microseismic refraction method for asphalt and concrete testing, development work began on compact, self-

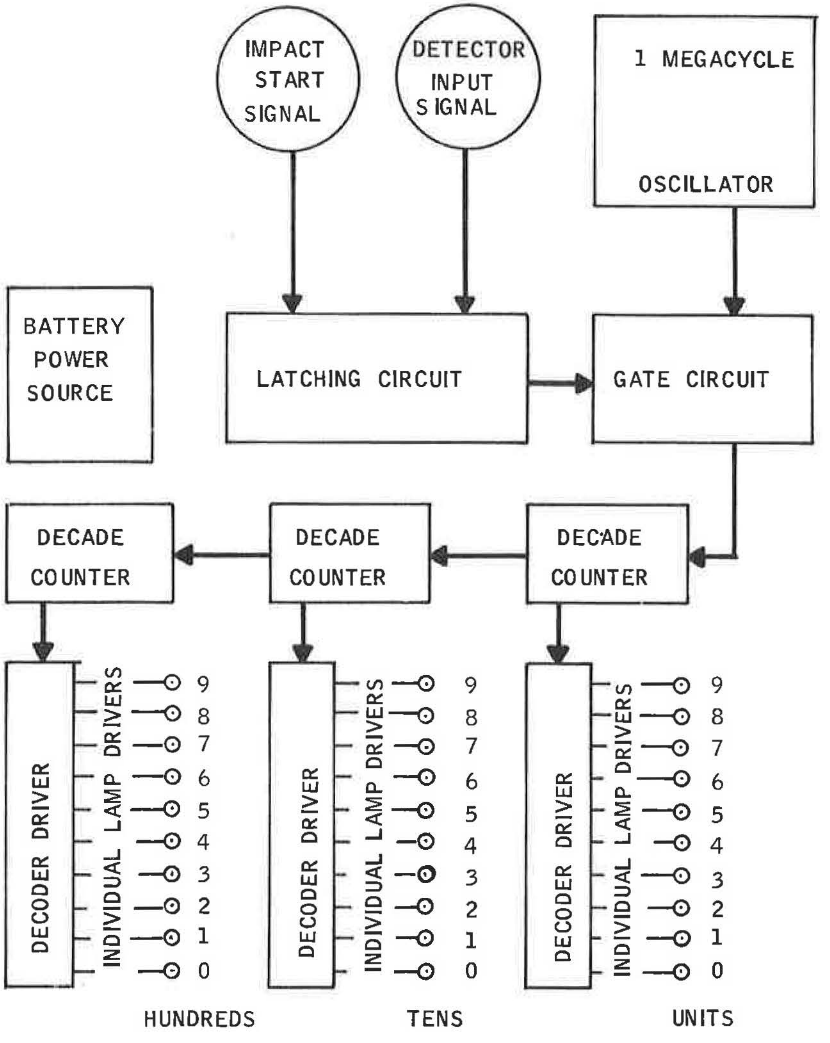

Figure 7. Functional components of direct-reading portable microsecond timer.

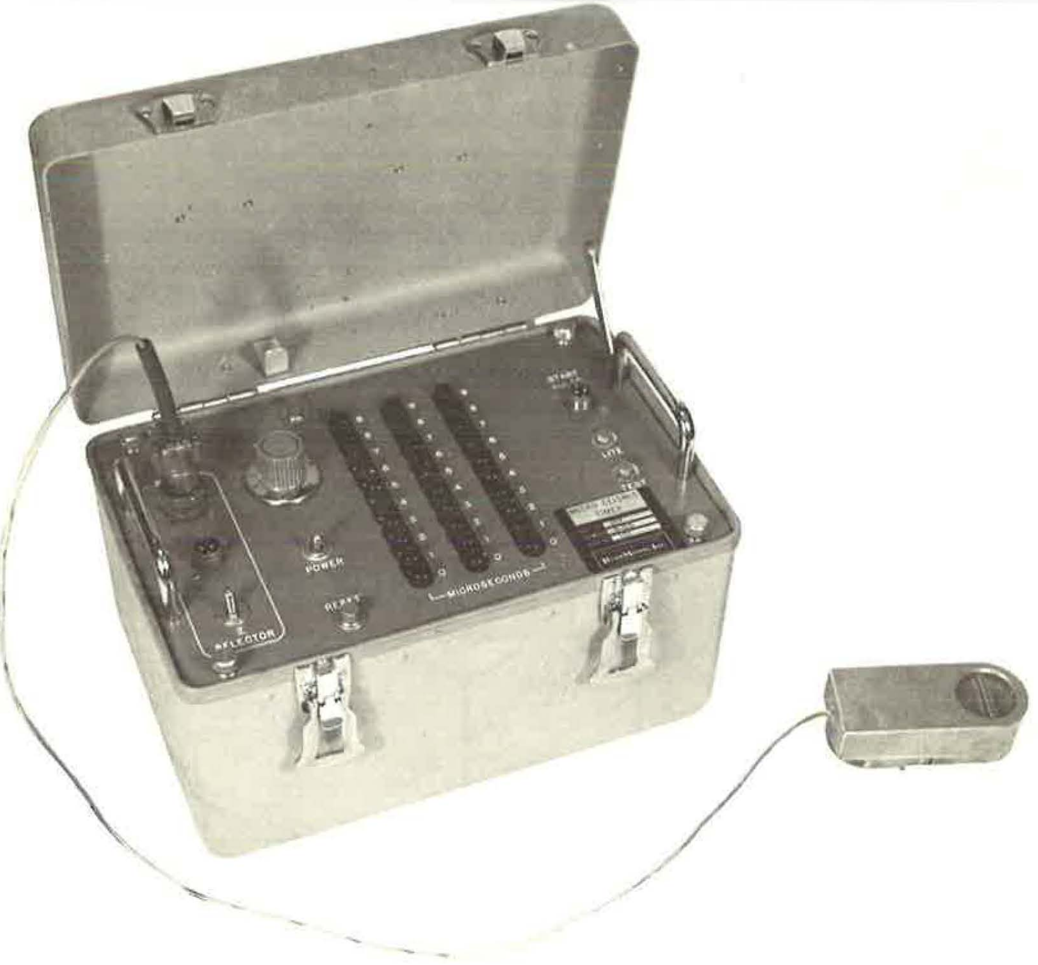

Figure 8. Microseismic timer and pickup.

powered, direct-reading instruments and accessories suitable for regular use by engineering department and maintenance department inspectors. A microsecond seismic timer and the necessary accessories have been developed, and field application studies are in progress.

The timer replacing the portable oscilloscope for use in direct measurement of travel times is a self-powered, direct digital reading instrument similar to the millisecond and tenth-millisecond timers used in shallow seismic exploration. Its time-counting operation is initiated by receipt of a start signal from the hammer impact. The counter is controlled by a stable oscillator. Time is measured by a series of three decade counters with microsecond switching capability displayed on appropriate decades of incandescent lights. A suitable amplifier and gate circuit receive the wave-arrival signal from the transducer and operate to stop the time count. The elapsed time in microseconds is displayed by the lamps that remain lit on the instrument panel, from 0 to 999 μsec. The timer can be reset instantly to 0, ready for a repeat reading. For elapsed times greater than 999 μsec, the count recycles. A simplified block diagram of the instrument's functional components is shown in Figure 7. The instrument is shown in Figures 8 and 9.

The transducer is a modified crystal-type pickup, having the required frequency response characteristics. The impact and triggering device is an improved version of

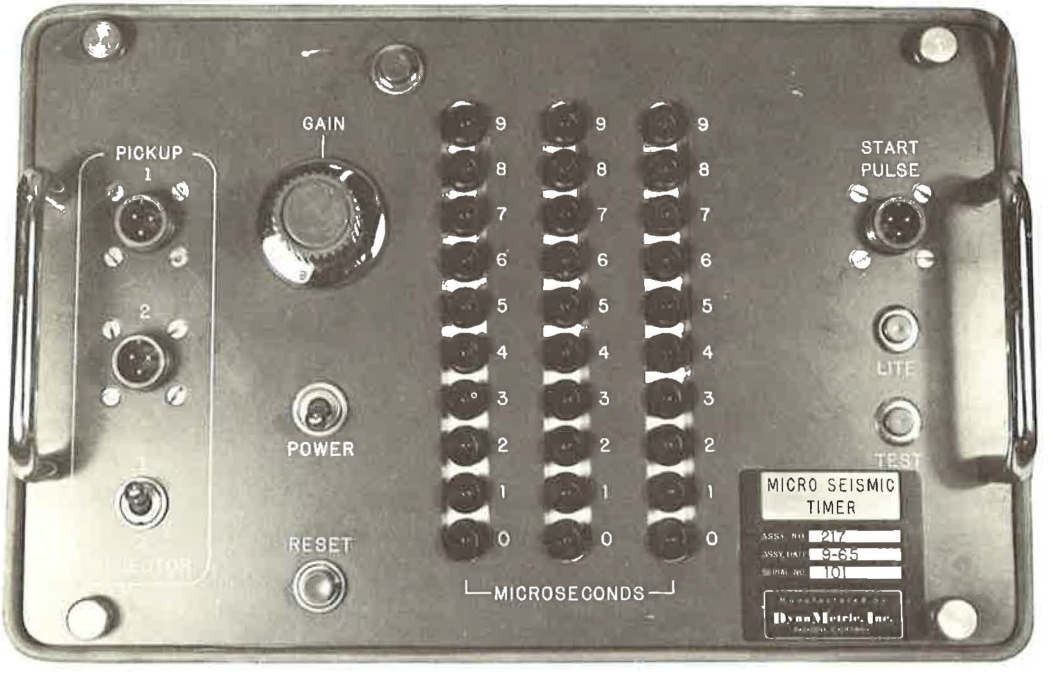

Figure 9. Microseismic timer control panel.

the hammer-coupler contactor used in evaluation tests, designed to withstand continuous hard field use. An accessory has also been developed for rapid layout of the survey lines. The instrument is powered by rechargeable batteries, and the total equipment kit is suitable either for hand-carrying or for transportation and operation from a small trailer with operator's seat mounted close to the pavement surface, for rapid coverage of long stretches of roadway.

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Appendix

THEORY OF MICROSEISMIC REFRACTION

This brief discussion of the refraction method is presented for those not familiar with seismic techniques, and will aid in understanding the method described in this paper.

Consider the surface of a broad expanse of material such as asphalt or concrete, as shown in Figure 10. If, at point A, a shock is imparted to the material, by hammer impact or other means, an elastic wave will travel out through the material in all di-

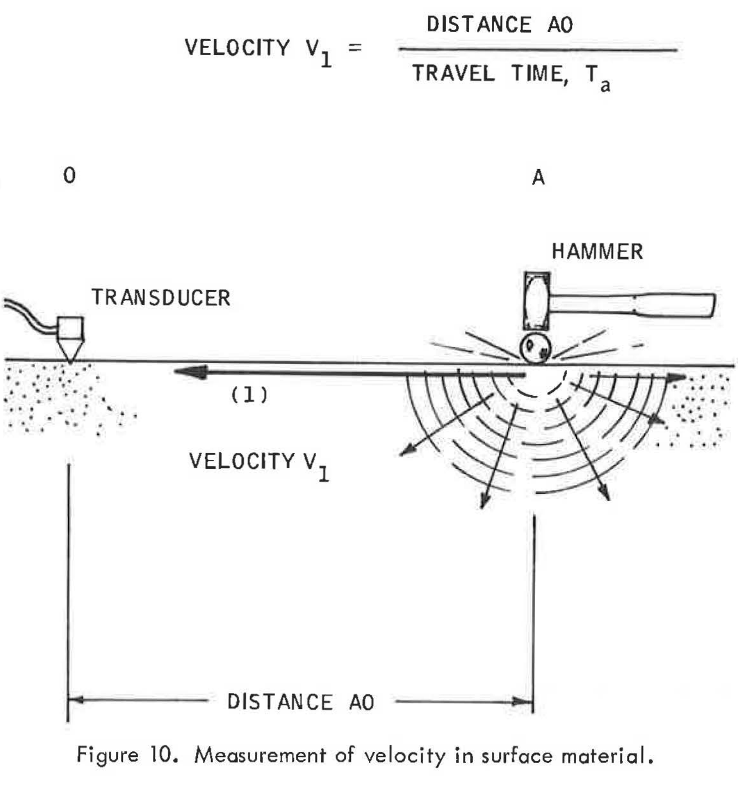

Figure 10. Measurement of velocity in surface material.

rections. If the material is homogeneous and free from boundary conditions, the wave front will be of spherical shape.

Elastic energy travels through elastic materials in a number of modes. The most important modes are the compressional mode (P-wave), in which elastic deformations take place in a direction parallel to the line of propagation; and the transverse, or shear, mode (S-wave), in which elastic deformations occur in directions normal to the line of propagation. Of these two modes, the compressional mode energy travels at the higher velocity, and so represents the leading surface of the expanding spherical wave front with which the refraction method is concerned.

It is helpful to visualize the expanding wave front as composed of an infinite number of rays, similar to the light rays emitted by a point source.

If there is placed at point O (Fig. 10) a transducer capable of detecting the arrival of the disturbance, it is possible to measure the time required for the wave front to move from point A to point O, along the path of ray (1). If the distance, AO, is known, the velocity, V_1 of the compression wave in the material can thus be determined.

Next, consider the condition encountered in a refraction survey (Fig. 11) in which the surface material is underlain at some depth, d, by another material capable of transmitting the compression wave at a velocity, V_2, higher than that of the surface material. Note that this condition is essential to the refraction method—underlying materials with velocity lower than that of surface materials cannot be detected.

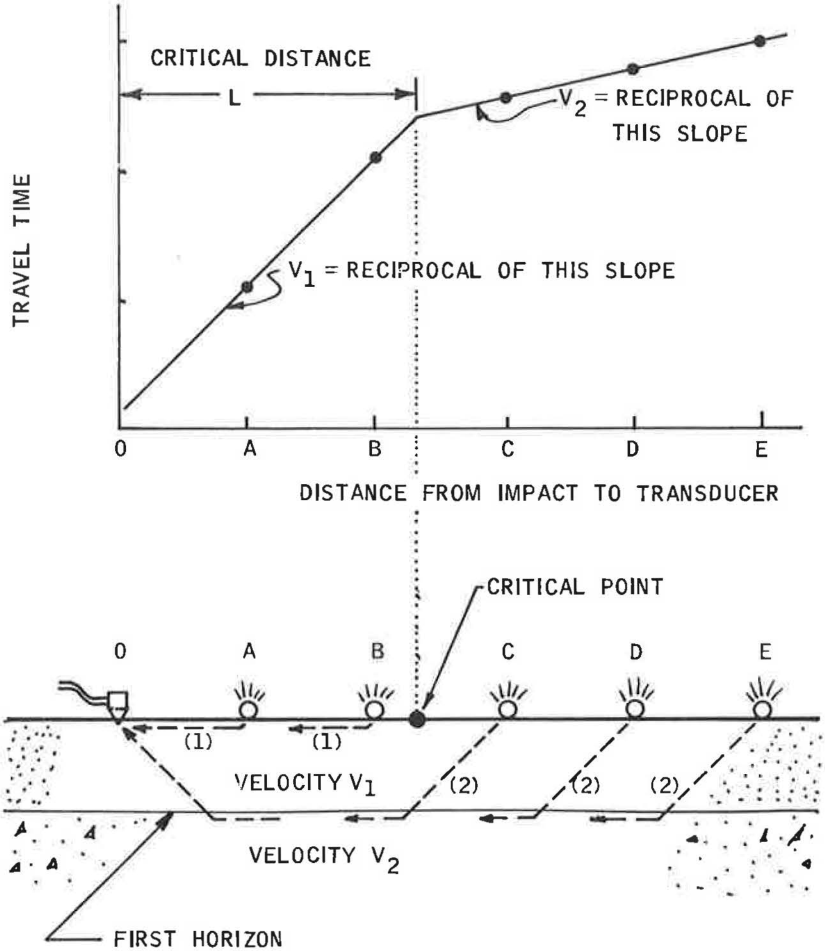

Figure 11. Refracted wave path, travel-time graph.

Assume that an impact is first produced at point A, close to the transducer, O, and that impacts are then produced at points B, C, D, and so on, each successively more distant from O. The travel time from each impact station is measured and plotted on a graph similar to the one in Figure 11. The measured travel times are based on the first arrival of disturbing energy at the transducer. Energy from any impact station can now reach the transducer over more than one ray path. Since time measurements are based on first arrival, it is a permissible simplification to state that there are now two available paths. One is the direct path (1) of Figure 10, at velocity V_1. The other is the refracted path (2) of Figure 11, along which energy travels down to the underlying material at velocity V_1, through the underlying material at velocity V_2, and back to the surface at velocity V_1.

Obviously, if the impact point is close to the transducer, the direct path offers the shortest travel time. As impact distance increases, and if V_2 is appreciably higher than V_1, a point will be reached at which the refracted path (2) offers a travel time as short as the direct path (1). This point is called the critical point, and its distance from the transducer is called the critical distance. From all impact points beyond the critical point, the first energy to reach the transducer will have traveled over the refracted path (2), i.e., travel times measured from impact points at greater than critical distance will be less than those which would be observed were there no underlying material, hence no refracted path available.

A distinct change in the slope of the travel-time graph thus indicates the presence of underlying material (in this case, concrete) capable of transmitting elastic energy at a velocity higher than that of the surface material. The velocity, V_1, in the surface material is represented by the reciprocal of the slope of the first portion of the graph. The velocity, V_2, in the underlying material is represented by the reciprocal of the slope of the second portion of the graph.

The physics governing the refracted path follows the laws of optics in refractive media. The various rays entering the underlying material are refracted by varying amounts according to Snell's law, i.e., the relation of the sine of the angle of incidence to the sine of the angle of refraction is proportional to the ratio of the velocities in the two materials. One of these rays will therefore be refracted at the correct angle to travel along the surface of the underlying material. This ray, in turn, will continue to emit elastic energy, some of which will eventually be refracted at the correct angle to return to the surface at the transducer.

Calling the depth to the underlying material (first horizon) d, and using the previously given nomenclature, the formula for depth to first horizon can be derived by writing expressions for the travel time from the critical point in terms of the two available paths, and equating these two expressions (since travel times are equal from the critical point), then solving for d. The expressions are derived in all standard tests on exploration geophysics. The resulting depth formula is

$$d = \frac{L}{2}\sqrt{\frac{V_2 - V_1}{V_2 + V_1}}$$

where L is the critical distance as observed on the travel-time graph, and V_1 and V_2 are the velocities in the surface and underlying materials, taken from the slopes of the respective portions of the travel-time graph.

Similar expressions can be derived for depths to second, third, and successively deeper horizons, the mathematics in each case becoming somewhat more complicated. For concrete testing, it is doubtful that more than a second horizon (three materials) would ever be encountered. The second horizon (surface of a third material) would appear on the travel-time graph as a second critical point, or change in slope. This might be caused, for example, by medium quality concrete on the surface of a slab, underlain by high quality concrete, both being covered by asphalt. The expression for depth to a second horizon is

$$d_2 = d_1(1 - R) - \frac{L_2}{2}\sqrt{\frac{V_3 - V_2}{V_3 + V_2}}$$

where d_2 is the depth from surface to the second horizon, d_1 is the depth to the first horizon computed by the simpler formula, L_2 is the second critical distance from the transducer to the second change in slope of the travel-time graph, V_3 is the velocity in the third material, and as before, V_2 is the velocity in the second material. R is an algebraic simplification factor related to the ratios of V_3/V_2, and V_2/V_1, having the following values:

Values of R:

Values of V_3/V_2 \ Values of V_2/V_1	1.1	1.5	2	3	5	10
1.1	.39	.17	.12	.06	.03	.02
1.5	.56	.31	.21	.12	.07	.03
2	.60	.34	.24	.15	.08	.04
3	.63	.36	.26	.16	.09	.04
5	.64	.37	.26	.16	.09	.05
10	.64	.38	.27	.17	.11	.05