NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM
REPORT 196

RECONDITIONING HEAVY-DUTY FREEWAYS IN URBAN AREAS

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL

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RESEARCH SPONSORED BY THE AMERICAN ASSOCIATION OF STATE HIGHWAY AND TRANSPORTATION OFFICIALS IN COOPERATION WITH THE FEDERAL HIGHWAY ADMINISTRATION

AREAS OF INTEREST:
- PAVEMENT DESIGN
- PAVEMENT PERFORMANCE
- BITUMINOUS MATERIALS AND MIXES
- CEMENT AND CONCRETE
- CONSTRUCTION
- GENERAL MATERIALS
- MAINTENANCE, GENERAL

TRANSPORTATION RESEARCH BOARD
NATIONAL RESEARCH COUNCIL
WASHINGTON, D.C. 1978

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

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FOREWORD

By Staff
Transportation
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This report is recommended to design, construction, materials, and maintenance engineers, and others who become involved in the reconditioning of heavily traveled urban freeway pavements. The research included an extensive literature search and discussions with contractors, material suppliers, equipment manufacturers, representatives of state highway and transportation agencies, and others having information that could help in solving the many problems involved in reconditioning busy urban freeways. The end product of the study is a series of detailed rehabilitation schemes for several specific structural conditions. All of the schemes are designed to permit completion of construction activity on ¼-mile single-lane segments in 48 hr or less without total closure of the freeway to traffic and without significantly raising the elevation of the pavement surface.

As the original life cycles of the substantial mileage of freeway pavements constructed during the unprecedented roadbuilding surge of the 1960's draw to a close, highway and transportation agencies are faced with an equally unprecedented task of pavement rehabilitation. Nowhere are the problems of rehabilitation more severe and more trying than on urban freeways that must be kept open to traffic during the construction process. The interference of construction operations with the flow of traffic on already crowded pavements, the interference of traffic with construction operations, the ever-present need for concern about the safety of both the highway users and workers in such situations of conflict, and the loss of the most viable of rehabilitation alternatives, the overlay, because of overhead clearance requirements in the typical urban environment, present problems of imposing magnitude.

In the study reported herein, the researchers made a careful analysis of the capabilities of existing construction equipment and normally available construction materials, and concluded that, through some unconventional application of both equipment and materials, a significant improvement and speedup of urban freeway rehabilitation operations is within reach. Some well-defined guidelines are offered for several specific rehabilitation situations. Agencies that have situations similar to those for which solutions are proposed may want to test the proposed solutions in limited areas to evaluate their effectiveness.

Several revolutionary concepts for urban freeway rehabilitation that were identified during the course of the investigation are also briefly described in the report. The researchers concluded that major expenditures for development would be required before any of the concepts could be brought to an application stage.

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The research reported herein was performed under NCHRP Project 14-4 by the Texas Transportation Institute, Texas A&M University. The Texas A&M Research Foundation was the contractor for the study.

Dr. William B. Ledbetter, Professor of Civil Engineering, and Dr. Alvin H. Meyer, Associate Professor of Civil Engineering, Texas A&M University, were the co-principal investigators. The other authors of this report are Dr. Andrew H. Layman, Professor of Civil Engineering, and Dr. Donald Saylak, Associate Professor of Civil Engineering, Texas A&M University.

The work was done under the general supervision of Dr. Meyer with assistance from Dr. Ledbetter. In addition to Drs. Layman and Saylak, assistance was received from Debbie Fisher, Research Assistant, and Sidney Greer and Edward Ellis, Technicians.

The following served as advisory consultants to the project: Bruce Cloud, Executive Vice President, H. B. Zachary Company, San Antonio, Tex.; Ralph W. Coho, Jr., Vice President, Irl Daffin Associates, Inc., Lancaster, Pa.; Charles Foster, Director, Engineering and Research, National Asphalt Paving Association, Riverdale, Md.; Professor B. M. Gallaway, Research Engineer, Texas Transportation Institute, Texas A&M University, College Station, Tex.; Harold Halm, Executive Director, American Concrete Paving Association, Oak Brook, Ill.; Alan Kus, Vice President, Brighton-Krug Construction Company, Chicago, Ill.; R. W. Beaty, General Marketing Manager, Asphalt Construction Products, Barber-Greene Company, Aurora, Ill.

Thanks also are extended to the many equipment manufacturers, contractors, engineers, and government agencies who responded to inquiries and provided the project with the detailed information necessary for completion.

RECONDITIONING HEAVY-DUTY FREEWAYS IN URBAN AREAS

SUMMARY

A significant portion of urban freeway pavement is more than 10 years old and many of the designed traffic volumes and loads have been exceeded. In the next few years an important mileage of these pavements will require structural rehabilitation to remain in operation.

The objective of this project was to develop new technology by which all or part of the pavement structure on a heavily traveled urban freeway could be rapidly reconstituted or replaced (or both) so that the finished product would have a design service life equal to, if not greater than, that of the original pavement. Included in the objective was the restoration of the riding quality and nonskid characteristics of the pavement structure.

The major constraints in developing solutions to this rehabilitation problem included a time constraint that the rehabilitation must be completed in less than 48 hr, a facilities constraint which involved closing only one lane of the travel-way during most of the construction, and a quantity constraint in that significant lengths of travel-way were to be rehabilitated. An additional constraint was that vertical clearances at overpass structures could not be reduced, therefore overlays of any type could not be used.

At the initiation of this project a comprehensive literature search was conducted, which resulted in the preparation of an annotated bibliography containing more than 500 references. Following the literature search, a number of sites were visited and discussions were held with contractors, suppliers, manufacturers, researchers, and highway transportation agency representatives. At each site the problem was presented and discussed for possible solutions. Using the information gathered from the literature and the site visits, a rehabilitation strategy was developed to focus on the various aspects of the problem.

It was obvious from the beginning of the study that a universally acceptable solution to the problem presented would require the development of totally new and innovative technology. However, in view of the applied nature of the National Cooperative Highway Research Program, effort was concentrated on the development of practical advances within the realm of current engineering technology. No single, best, unique solution was found to the problem; rather a number of solutions were developed which appear promising for certain preexisting conditions. In this strategy three pavement structure conditions were considered: (1) all layers are structurally unsound; (2) the surface layer is structurally unsound whereas the sublayers are structurally sound; and (3) surface layer and all sublayers are structurally sound.

The major findings of the study are:

1. Because of the vastly different situations existing throughout the United States, no single solution to the problem could be developed. Therefore, a number of realistic solutions were developed to cover the expected range of situations which might be encountered.

2. The solutions involving the least amount of new technology were those in

which an adjacent lane could be closed during the rehabilitation of a significant length of a freeway lane.

3. Because of the time constraint (48 hours), improvement of the subgrade, or natural soil, could not be accomplished and had to be left "as is." Thus, the rehabilitated pavement structural layers had to be designed to prevent overstressing the unimproved subgrade.

4. The construction management techniques of precedence diagraming and analysis bar charting indicated the critical aspects of each rehabilitation strategy and provided the information necessary to schedule rehabilitation within the time constraint.

5. For the "worst probable" case [a 10 in. (0.25 m) portland cement concrete pavement exceeding 14 in. (0.35 m) of sublayers, all of which are deteriorated], solutions were developed for removing all materials in a quarter mile long (400 m) lane and replacing such material with materials equal to or better than the original pavement structure—in 43 hr from the time the barriers are erected. Using new and innovative technology the potential exists to reduce this time to 35 hr.

6. When only the surface layer is structurally unsound, solutions were developed whereby a quarter mile long (400 m) lane could be rehabilitated in 26 hr.

7. When all pavement layers are structurally sound (requiring only the restoration of riding quality and/or skid resistance), solutions were developed whereby a quarter mile long (400 m) lane could be rehabilitated in from 4 to 24 hr, depending on existing conditions.

8. Promising innovative materials/systems to accomplish rapid rehabilitation of significant portions of an urban freeway include:

(a) Deep-lift asphaltic concrete.

(b) Rapid-hardening, high early-strength concrete.

(c) Sulphur systems for pavements.

(d) Precast portland cement concrete panels.

9. Futuristic systems offer potential for rehabilitating significant sections of a freeway lane by closing only one lane of traffic throughout the rehabilitation periods.

10. One futuristic system has the potential for keeping all lanes open during rehabilitation.

11. Development of the futuristic systems will require the expenditure of large sums of money, and thus are unlikely to be undertaken without significant Federal support.

12. A large number of special materials/systems techniques exists that offer potential for rapid spot rehabilitation.

The solutions developed in this report call for close control of materials and close construction scheduling of the rehabilitation. To test the validity of the solutions, one or more solutions should be tried on a full-scale rehabilitation project in which a quarter mile long (400 m) lane is selected, a rehabilitation strategy is selected within the specified time limit, and a construction technique is employed to accomplish the rehabilitation. Materials selection should be made in cooperation with the materials supplier and the constructor. Construction management techniques, to include precedence diagraming and analysis bar charting, should be employed to insure proper execution of the rehabilitation.

In addition to the aforementioned materials/systems for replacement, several other popular and seemingly promising techniques were considered and rejected. These materials/systems are discussed in Appendix C and include polymer concretes, polymer modified concretes, polymer impregnated concretes, fiber concretes, epoxy injections, ultrasonic vibrations, and fabric layers.

CHAPTER ONE

INTRODUCTION AND RESEARCH APPROACH

INTRODUCTION

In 1972 the Federal-Aid Primary Highway System in urban areas included 35,350 miles (56,900 km) of the nations highways (*1*). In this urban system there are some 23,300 miles (37,500 km) which have 4 or more lanes of traffic and of those, 17,000 miles (27,400 km) are divided highways. Of these divided highways, 1,800 miles (2,900 km) have partial access control and 9,700 miles (15,600 km) have full access control. Approximately 62 percent of the urban highways carry more than 10,000 vehicles per day (vpd) and more than 3,800 miles (6,100 km) of these highways carry more than 40,000 vpd.

The Interstate Highway System comprises about 8,800 miles (14,200 km) of this urban system. Of these, 5,400 miles (8,700 km) are portland cement concrete and 3,400 miles (5,500 km) are asphaltic concrete surfaces. Some 5,400 miles (8,700 km) of this system carries more than 20,000 vpd of traffic.

A significant portion of the urban freeway pavement is more than 10 years old, and many of the design traffic volumes and loads have been exceeded. In the next few years, a substantial mileage of these pavements will require structural rehabilitation to remain in operation. In several areas, especially where volume capacities have been increased by adding traffic lanes, the entire pavement will not need rehabilitation. Other pavements may exhibit distress only in relatively short segments along their lengths, due to loss of subgrade support by erosion, pumping, saturation, or other causes.

The types and causes of distress must be identified before realistic decisions regarding rehabilitation can be made. Three distress modes (fracture, distortion, disintegration) and the associated mechanisms have been suggested (*2*). These distress modes would lead to a need for rehabilitation to improve one or more of the following (*2*):

1. Level of service.
2. Riding quality.
3. Safety.
4. Structural adequacy.
5. Surface condition.
6. Cost of maintenance.

Structural adequacy is probably the most difficult to improve because removal and replacement of part of the existing pavement or expensive in-place subgrade treatment or relatively thick structural overlays may be required. Often, these types of rehabilitation treatments are difficult to perform on limited portions of the roadway width.

Defining the problem of needs for pavement rehabilitation can be an infinite task; however, there are several rational approaches to evaluating pavements. One such systematic approach has been proposed by Hudson and Finn (*3*). In this approach, a number of inputs are used to predict the need for rehabilitation, and rational methods for establishing priorities are presented.

The probable causes of distress which require removal and replacement can be categorized in four general areas:

1. Inadequate design.
   (a) Estimate of traffic and/or percent of trucks too low.
   (b) Change in allowable loads using roadway.
   (c) Excessively severe environment.
   (d) Other.
2. Inadequate materials.
3. Inadequate construction procedures.
   (a) Improper consolidation.
   (b) Inadequate quality control.
   (c) Other.
4. Inadequate subgrade support.
   (a) Inadequate drainage.
   (b) Inadequate maintenance.
   (c) Swelling soils.
   (d) Other.

The first of these areas, inadequate design, shortens the expected life of the pavement. For example an 8-mi (13 km) portion of the Dan Ryan Expressway in Chicago, Illinois, was removed and replaced after being in service for only 9 years (*4*). One of the principal factors responsible for its early failure was that it carried greater than 35 percent more traffic than could have been anticipated from the information available when the pavement was designed. The distress due to inadequate design does not "per se" require removal and replacement. In many cases, overlay procedures are adequate for rehabilitation to extend the life of the pavement. However, when inadequate design is coupled with other modes of pavement distress, removal and replacement may then be required.

The remaining three of these general areas tend to be localized and seldom, if ever, exist for the entire length of a given roadway section.

An example of area 2, inadequate materials, might be improper reinforcement causing inadequate performance of the pavement structure which usually results in localized failures.

In area 3, inadequate construction procedures, a number of possibilities exist, such as periodic improper batch quantities causing lean mixes with inadequate strength. This could be true for either asphaltic concrete (AC) or portland cement concrete (PCC) pavements. Sometimes starting and stopping the paver can result in a weak spot. In either condition the weak spots will eventually result in localized failures.

Area 4 is probably the type of distress that will most often exist if the pavement rehabilitation is to require removal and replacement. Inadequate drainage has been identified as a cause of pavement distress in almost all parts of the country. California, Illinois, and Florida, among others, have reported this type of distress (*4, 5, 6*). As reported by Ring (*7*), inadequate drainage can result in significantly weakened subgrades and/or untreated (and in some cases treated) base courses. Ring points out that overlays per se will not solve problems caused by poor drainage.

When the pavement distress is sufficiently localized, a number of patching repair techniques have been used with adequate performance. Some of these can be performed in less than the 6-hr time frame usually existing for off-peak hours on urban freeways. These include precast slabs such as used in California, Florida, and Michigan (*8, 9, 10*), or special concrete patching materials such as those evaluated by Pike and Baker (*11*).

All cases of rehabilitation require advance planning and cooperation among the several agencies involved in the management of an urban system. As pointed out in *NCHRP Synthesis 25* (*12*), every available means should be exercised to insure the safety of the workmen and the driving public. In addition, the problems of noise and dust must be controlled and the disposal of waste products (such as old pavement) must be considered. Even under the best of conditions, urban highway rehabilitation is expensive in terms of both direct costs and user costs. The longer a rehabilitated segment remains serviceable, the lower will be the cost per unit time. This forces consideration of high quality materials, unusual subgrade treatments, and even ultra conservatism in strength design to extend the service life of the rehabilitation.

This project sought to synthesize the current technology available to accomplish rapid removal and replacement of distressed segments of urban highways. In addition, some futuristic systems were considered.

OBJECTIVE

The objective of this project was to develop new technology by which all or part of the pavement structure on a heavily traveled urban freeway could be reconstituted or replaced (or both) so that the finished product would have a design service life equal to, if not greater than, that of the original pavement. It includes restoration of riding quality and nonskid characteristics of the pavement structure.

CONSTRAINTS

In proposing a solution to any problem some constraints must be recognized or assumed. The following constraints are imposed in order to present a feasible case for consideration:

1. The facility has a minimum of three lanes in each direction or two lanes with a shoulder capable of carrying traffic through a construction site at reduced speed.
2. The rehabilitation must be completed in less than 48 hr, closing only one lane during peak-traffic periods and closing an adjacent lane during off-peak traffic periods. For the purpose of this study the peak traffic periods are defined as 6 to 9 AM and 3 to 8 PM.
3. Significant lengths of a single-lane are to be rehabilitated; that is, deteriorated sections exceeding 200 ft (60 m) in length.
4. The original pavement structure has deteriorated to the point where partial to complete removal for depths up to 24 in. (0.6 m) are required.
5. The natural subgrade has a California bearing ratio (CBR) of 2 to 5, or a modulus of subgrade reaction (k) of less than 100.
6. No weather delays will be encountered.
7. Vertical clearances at overpass structures cannot be reduced, so that overlays of any type cannot be used.
8. Traffic barriers to include visual screens are to be erected to prevent motorists in adjacent lanes from becoming distracted by the construction.
9. The rehabilitation systems developed will be flexible and "open-ended" to permit rapid adaptation to particular conditions on a given project.

RESEARCH APPROACH

Literature Search

At the initiation of this project a four-phase literature search was conducted. First a Highway Research Information Service (HRIS) search was made using selected key words (see Appendix A). This search generated 720 abstracts for review.

A second search using the same key words was made through the National Technical Information Service (NTIS). An additional 1,322 abstracts were secured for review. From the articles used in the bibliography it was observed that only about 15 percent overlap of articles existed between the HRIS and NTIS searches.

The research librarian for the Texas Transportation Institute (TTI) then made a search using similar key words for references not identified by either of the previous searches. This search produced 180 more reference abstracts for review. Additionally, each of the researchers involved in the project secured references for his particular area of concern. An annotated bibliography containing more than 500 references was prepared from these searches.

Site Visits

A number of sites were visited and discussions were held with contractors, suppliers, manufacturers, researchers, and agency representatives. Locations included Alabama, California, Florida, Georgia, Illinois, Massachusetts, Michigan, Mississippi, Missouri, Ohio, Oklahoma, Texas, Utah, and Washington. These are more fully described in Appendix B. At each site the problem was presented and discussed for possible solutions.

General Approach

Based on the established constraints, the information gathered from the literature search and site work was analyzed in the context of meeting the objective of the project. A rehabilitation strategy was developed, as shown in Figure 1 to focus on the various aspects of the problem. In this strategy three pavement structure conditions were considered: (1) all layers are structurally unsound; (2) the surface layer is structurally unsound, whereas the sublayers are structurally sound; and (3) the surface layer and all sublayers are structurally sound.

Because of the construction time constraint (48 hr) the researchers concluded that improvement of the underlying subgrade could not be accomplished rapidly enough to allow time to rehabilitate the pavement structure layer (see Appendix F for details). Thus, the subgrade was considered "as is" and the upper layers of the pavement structure were designed to prevent overstressing the subgrade materials.

Early in the study a basic premise established was that the research should concentrate on solutions within the realm of current engineering technology. Although it was recognized that the problem almost certainly demanded revolutionary developments for a completely satisfying solution, practical considerations of project funding, time, and the applied nature of the National Cooperative Highway Research Program provided persuasive reasons for not engaging in research that would be highly dependent on the uncertainties of future development before practical application of the results could be achieved.

The research led to a number of solutions which appear to be promising for certain preexisting conditions.

These solutions use various candidate materials/systems, some of which are new and innovative (see Appendix C for details). These systems were first analyzed for structural adequacy in either an elastic-layered analysis system or a finite-element system (Appendix D). Those solutions found structurally sound were then presented to a panel of researchers from the Texas Transportation Institute. Subsequently, this panel's suggestions were incorporated into the proposed solutions and presented to two panels; one being primarily rigid- (portland cement concrete) systems oriented and the other flexible- (bituminous concrete) systems oriented. The panels consisted of representatives from paving associations, equipment manufacturers, and contractors. Using their input, the proposed solutions were further refined and the most promising approaches to the problem were selected. These promising approaches are presented in Chapter Two of this report. Other materials/systems which may be applicable in some cases are described in Appendix C.

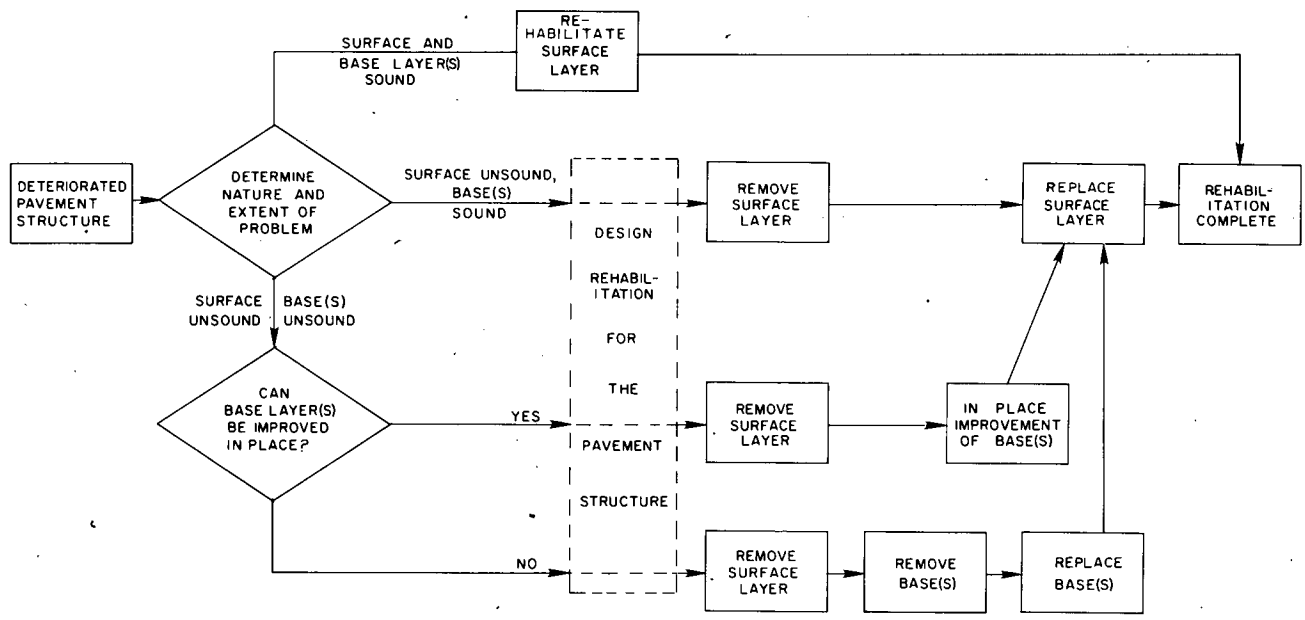

Figure 1. Rehabilitation strategy.

CHAPTER TWO

FINDINGS

GENERAL

In this chapter solutions are presented for three rehabilitation problems enumerated in Chapter One (see Fig. 1); to wit:

1. All layers are structurally unsound.
2. The surface layer is structurally unsound whereas the sublayers are structurally sound.
3. The surface layer and all sublayers are structurally sound.

Solutions to these problems involve two aspects—construction techniques and materials. To meet the project objective, innovative concepts had to be employed in both aspects, and solutions had to be formulated in sufficient detail to demonstrate feasibility. Accordingly, innovative construction techniques are described using (whenever possible) currently available equipment with proven performance records. This equipment is often mentioned by name—for illustrative and documentation purposes only. Such mention should not be construed to imply endorsement of one brand of equipment over another. Some innovative material concepts are also described by name. Again, this should not be construed to imply endorsement of a particular product, but rather as an illustration of a type of material suitable for a particular situation. Sufficient detail is given to demonstrate feasibility. However, specific sequencing and details for routing of equipment and materials flow are not included because each situation is different and may require special routing considerations which cannot be enumerated in this report.

In addition to presenting innovative solutions to the three problems, a section is included discussing futuristic systems for solution to the problem. These futuristic systems are conceptual, and indicate the types of materials/systems that might be employed—should sufficient need develop.

Of the two pavement systems—flexible and rigid—the rigid system requires more effort to remove, because breaking portland cement concrete is an expensive, time- and energy-consuming operation. Solutions are proposed for rehabilitating a rigid system within the constraints enumerated in Chapter One. These solutions can then be used, with only minor modification, for flexible pavement systems.

All three problems have in common the requirement of at least a one-lane closure (the one being repaired) to effect the rehabilitation. The methods and procedures for lane closure are extremely important as well as time-consuming. A number of reports exist on this subject and the reader is referred to references *12, 13,* and *14* for detailed information. For completeness of the solution, one system for lane closure(s) is presented in this report. This system is *not* to be considered as the optimum one, but rather as a system which offers some advantages under the stringent time constraint imposed for the rehabilitation. Following the section on lane-closure system, solutions to the three problems are presented.

LANE-CLOSURE SYSTEM

Lane closure on a high-volume roadway is hazardous at all times and costly. Quoting from *NCHRP Synthesis 25* (*12*):

> The development of traffic handling plans must be given as much comprehensive professional attention as is required for the physical repairs themselves. Agencies must be prepared in some instances to spend as many dollars on the traffic handling requirements of the project as on its basic construction features.

The problems of lane closure(s) are intensified when work is accelerated, constricted, and proceeding on a 24-hr day basis (all of which are required to meet the objective of this project). One notable problem is the so-called "motorist gawking syndrone" (*12, 14*) where motorists become distracted by the construction activity and thus are more accident prone than when their entire attention is focused on driving their vehicle.

The proposed closure system employs standard warning signs, flashing lights, overhead illumination for nighttime construction, traffic cones, and reduced speed limits through the construction site. In addition, a "vision barrier screen," as shown in Figure 2, is proposed. This screen, made by attaching standard chain-link fence units to a 12 by 12 in. (0.30 by 0.30 m) timber base, can be quickly deployed and removed. It offers an effective, stable screen in winds gusting to 50 mph (80 kmh). This screen is a modification of a barrier system presently being used in Massachusetts (see Appendix B) and reported to give excellent results.

PROPOSED SOLUTIONS FOR ALL LAYERS STRUCTURALLY UNSOUND

General

Additional assumptions made for this particular problem are:

1. The pavement to be rehabilitated has a minimum of three 12-ft (3.7-m) wide lanes in each direction and consists of a 10-in. (0.25-m) thick unreinforced concrete pavement resting on a 6-in. (0.15-m) granular base and an 8-in. (0.20-m) lime-stabilized subgrade.
2. The vertical alignment of the rehabilitated surface will conform to the average elevation of the adjacent lane.

3. A ¼-mile (400-m) section of outside lane will be rehabilitated.

The solutions to this problem involve the following 15 activities:

CODE	ACTIVITY
A	Barrier Erection
B	Breaking Concrete Pavement
C	Loading and Removing Concrete Pavement
D	Handling Subbase Layers
E	Transport/Mix Base 1
F	Compact Base 1
G	Cure Base 1
H	Transport/Mix Base 2
I	Compact Base 2
J	Cure Base 2
K	Transport/Mix Surface Layer
L	Place/Finish Surface Layer
M	Cure Surface Layer
N	Stripe Surface
O	Remove Barriers

These activities are shown in modified-precedence diagram form in Figure 3. The sequence of events is shown in the diagram, as well as the duration of each activity and the time each activity is scheduled to start and finish. Also shown are the periods of delay where the rehabilitation operation is restricted to a single lane.

To better visualize the time sequence and time control, the activities are shown in analysis bar chart form in Figure 4. The activity is scheduled to start at 6 PM, with the barrier erection to precede the start of operations. If all the activities proceed as scheduled, the ¼-mile (400 m) section will be completed and opened to traffic 43 hr later. The 6 PM starting time was selected for several reasons. First, it allows the constructor to complete the removal operation during the night hours when traffic is at its lowest volume (such removal operations are not seriously curtailed by darkness). Secondly, the sequence provides for the majority of the replacement work to be accomplished during the daylight hours when such light is necessary for efficient operations. And thirdly, this particular sequencing plan provides for a 43-hr lane closure in which the construction could be continued to an adjacent roadway section by moving the barrier system down the road. This would permit a series of ¼-mile (400 m) sections to be closed, rehabilitated, and opened to traffic in 48-hr time increments.

In the following paragraphs each activity is described giving the materials/systems proposed for use and the construction technologies required to complete the activity within the time allotted.

Code A. Barrier Erection

To provide an effective visual screen between the moving traffic and the construction, the barrier shown in Figure 2 is proposed. This unit weighs approximately 500 lb (230 kg) and can be moved into place using a light crane or small hoist, such as the knuckleboom crane, at the rate of one every 2 min. Assuming the barrier is to be placed on the traffic side of the construction, closing the outer two lanes for a distance of 1,320 ft (400 m) plus 300 ft (90 m) on each end for transition, equipment maneuvering, and the like, a total barrier length of 1,920 ft (580 m) is required, which means that 240 sections will be needed. Using two hoists operating an average of 50 min. each hour, the barrier can be off-loaded and erected in 5 hr. In addition to the barrier system, conventional signing, including sequential illuminated arrows and warning lights, should be employed in accordance with established procedures. During periods of peak traffic, the barrier system can be placed across one lane in less than 1 hr, leaving only the outside lane closed to traffic.

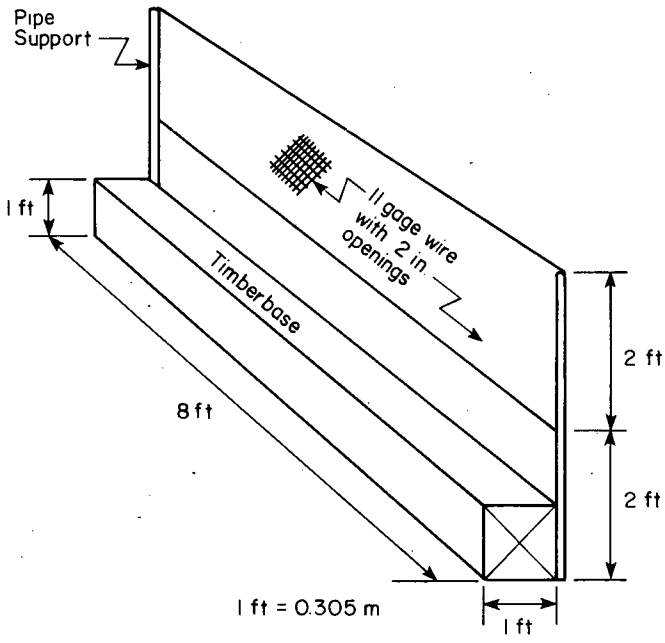

Figure 2. Proposed vision barrier screen.

Code B. Breaking Concrete Pavement

This activity involves the breaking of some 1,760 sq yd (1,470 m²) of concrete pavement, 10 in. (0.25 m) thick, in 3 hr during a peak-traffic period. To do this, the pavement must be broken at a rate of close to 600 sq yd (500 m²) per hour without producing "flying" chips of concrete. The equipment proposed for this operation is a large air hammer on a self-contained rubber-tired chassis. The CMI H-16 super hammer shown in Figure 5 is capable of punching a hole through and partially breaking the pavement every 20 sec without creating any flying rock problems. Allowing 10 sec for movement between holes, and punching holes on 2 ft by 3 ft (0.05 by 0.08 m) centers, this unit can break some 150 sq yd (125 m²) of concrete per hr. To meet the production schedule, four of these units are proposed. In addition, saw cuts might be necessary at each end of the section, and occasionally along a longitudinal edge. Such sawing can be accom-

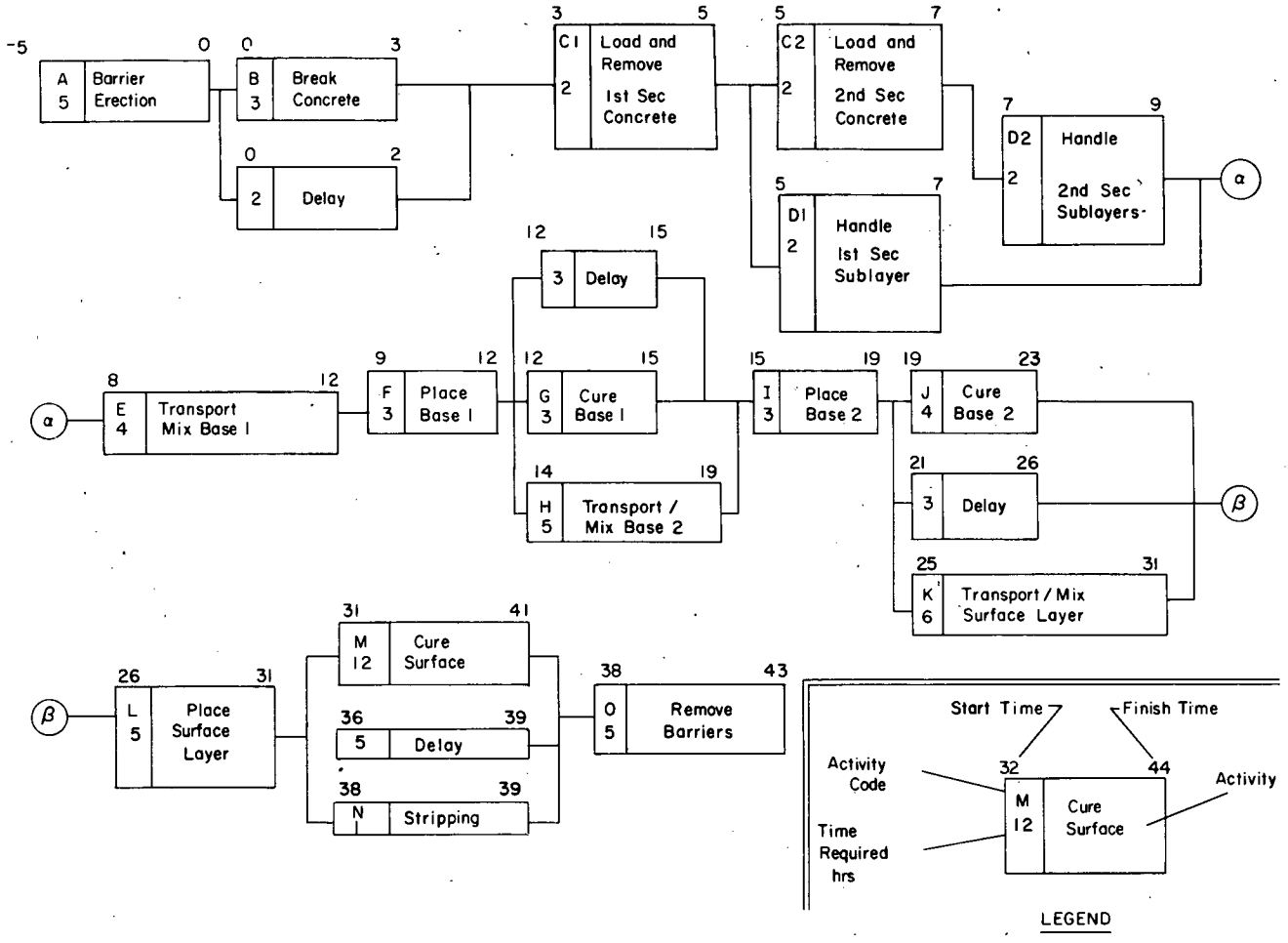

Figure 3. Modified precedence diagram for highway rehabilitation—all layers structurally unsound.

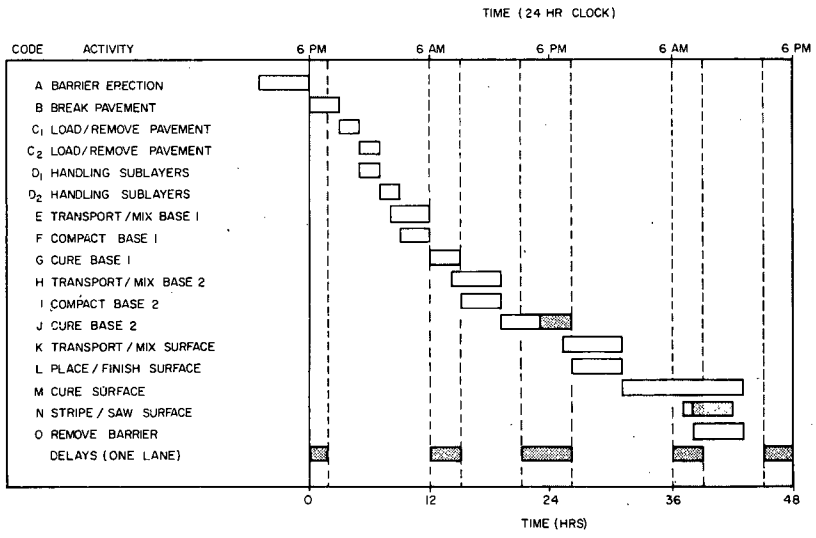

Figure 4. Analysis bar chart for highway rehabilitation—all layers structurally unsound.

plished by a portable concrete saw, such as the Clipper, Model 655, shown in Figure 6, which can average 30 lineal ft (9 m) per hr of full depth cut.

Code C. Loading and Removing Concrete Pavement

The broken concrete will still appear to be joined together after the breaking operation, but such equipment as the Gradall G-880 shown in Figure 7 can easily complete the break-up of the concrete and load it onto dump trucks. For this operation, approximately 500 cu yd (380 m^3) of consolidated concrete or 800 cu yd (610 m^3) of loose concrete need to be loaded and removed in 4 hr. One or two G-880 Gradall units, with articulated booms and pavement removal buckets, can meet this production rate.

Code D. Handling Subbase Layers

In this activity, the sublayers are assumed to be unsound and therefore require rehabilitation. Treatment may consist of either in-place rehabilitation or removal and replacement; both types of treatment are given in Table 1. Removal and replacement is, by far, the more time-consuming and complex and will be discussed first.

Operations for Removal and Replacement

Code D. Removal Operation.—In comparison with the concrete pavement, the excavation and removal of the sublayers (in this case a granular base and a lime stabilized subgrade) will be a relatively simple task, but a rather large quantity of material must be moved. Assuming the 14-in. (0.36-m) thick base and subbase are to be removed in 4 hr, some 700 cu yd (540 m^3) bank measure or 900 cu yd (690 m^3) loose measure of material must be moved. Here the Gradall G-880 (Fig. 7), with its versatility, would appear to be ideal for the job. These second units would work closely with the first units loading both broken concrete and the subbase layers.

Codes E, F, G, H, I, J. Replacement Materials/Systems for the Sublayers.—Acceptable candidate materials/systems for the sublayers are given in Table 2. Each system has advantages and disadvantages. Asphaltic concrete and portland cement concrete bases are both cost effective, and they both work, given enough time to emplace and cool (or cure). Seven hours are allotted for this activity (Fig. 4). When the removal activities are completed, the site will consist of a "trench" 24 in. (0.6 m) deep by 12 ft (3.7 m) wide by 1,320 ft (400 m) long. The bottom of the trench will be somewhat rough, and is assumed to consist of a weak material (CBR of 2 to 3 or a k less than 100). In order to conserve time, no fine grading or compaction of the subgrade is proposed. The placement of each of the various candidate materials is described in the following paragraphs.

1. Hot-Mixed Asphaltic Concrete Base.—The method proposed here is to transport the mixture in insulated dump trucks (if necessary) and dump into a side delivery spreader (commonly referred to as a "road widener") that will spread the mixture into the hole for subsequent compaction to a thickness of approximately 8 in. (0.2 m) as shown in Figure 8. Following the placement and leveling operation, the mix will be compacted with pneumatic and steel wheel rollers until the desired compaction is achieved. This lift will require 750 tons (680,400 kg) of asphaltic concrete in 4 hr, or less than 200 tph (181,400 kg per hr). The three main problems are: (1) delivering this quantity

Figure 5. CMI H–16 super hammer.

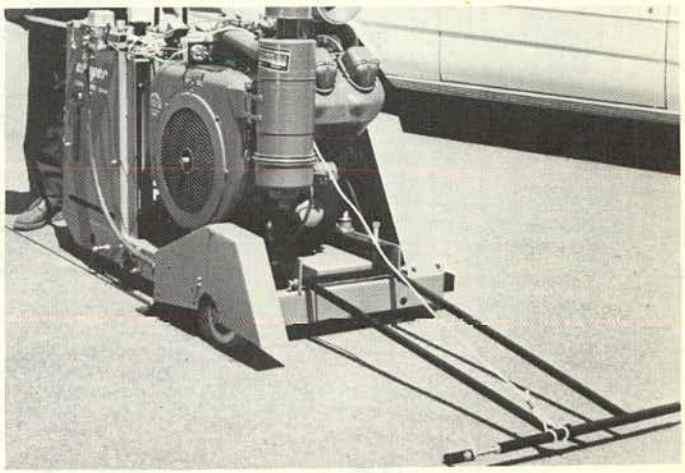

Figure 6. Clipper concrete saw, Model 655.

Figure 7. Gradall model G–880.

TABLE 1

REHABILITATION TREATMENTS OF SUBLAYERS

CODE	(FIGS. 2 AND 3) ACTIVITY	OPERATIONS FOR REMOVAL AND REPLACEMENT	OPERATIONS FOR IN-PLACE REHABILITATION
D_1	Handling sublayers	Remove granular base	Manipulation of granular base
D_2	Handling sublayers	Remove stabilized subgrade	None
E	Transport/mix base 1	Transport/mix base 1	None
F	Compact base 1	Compact base 1	None
G	Cure base 1	Cure base 1	None
H	Transport/mix base 2	Transport/mix base 2	Addition of stabilizer
I	Compact base 2	Compact base 2	Mix/compact base
J	Cure base 2	Cure base 2	Cure base

TABLE 2

ACCEPTABLE CANDIDATE MATERIALS/SYSTEMS FOR SUBLAYERS

MATERIAL/SYSTEM [a]	REMARKS
1. Hot-Mixed Asphaltic Concrete Base	Easily constructed, cost effective,[b] no new technology required, needs time to cool.
2. Lean Portland Cement Concrete Base	Easily constructed, cost effective,[b] no new technology required, needs time to cure.
3. Lean Rapid-Hardening, High Early-Strength Cement Concrete Base	Difficult to construct, needs innovative technology, expensive, fast.
4. Sulphur-Aggregate-Asphalt Systems	Need innovative technology, fast, needs time to cool, cost effective.[b]
5. Polyurethane Foam	Very fast, very expensive, easily constructed.
6. Combination Systems	Combinations of 1 through 5.

[a] See Appendix C for additional details.
[b] Anticipated costs do not appear to be excessive in terms of anticipated benefits.

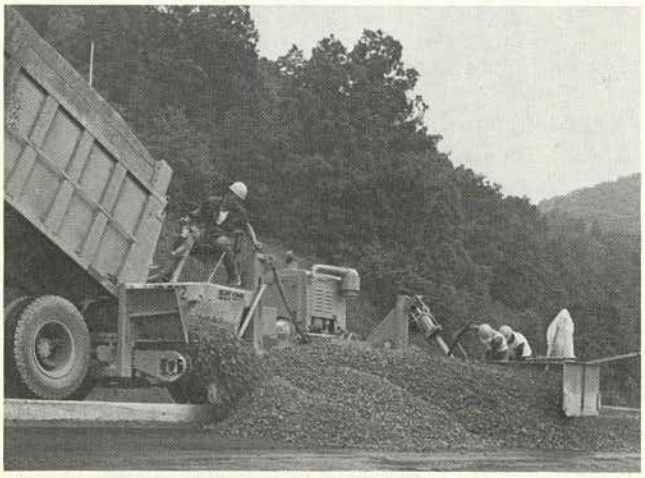

Figure 8. Blaw-Knox RW–195 road widener.

of material for the job through heavy urban traffic; (2) achieving adequate compaction of such a deep lift; and (3) having enough time to adequately cool the lift. The first problem can only be overcome by firm commitments from the supplier and careful scheduling. The second and third problems can be overcome by careful temperature control of the mix from batching through delivery in insulated trucks and final laydown. If the mix is too hot, then deep compaction becomes very difficult and time-consuming (see Appendix C). Thus, lower than usual mix temperatures are recommended.

A 3-hr cure time is programmed to allow the layer to cool sufficiently for placement of the next layer. Although this time may not be needed, it is deemed necessary to allow time for the heat to dissipate (when necessary).

The next sublayer will be placed and initially compacted with an asphalt paver to a compacted thickness of 6 in. (0.15 m). Such lift thicknesses have been successfully constructed, and standard pavers are capable of placing the 560 tons (508,000 kg) required in this activity within the 3 hr allotted, as the paver will need to advance at a rate of less than 8 ft per min. (0.04 m/s) to complete the 1,320 ft (400 m) long section. Following the placement of this sublayer an additional 3-hr cure time is again provided for cool-down.

2. Lean Portland Cement Concrete Base.—Lean concrete (low-cement factor concrete) has been shown to be cost effective * and structurally adequate (*15*). In this case, a 14-in. (0.36-m) thick lift of concrete must be placed in 7 hr [some 700 cu yds (535 m^3) of concrete]. This can be accomplished using two 8-cu yd (6-m^3) concrete ready-mix trucks dumping simultaneously into the trench. Each truck can easily maneuver and discharge in 5 min., supplying 16 cu yd (12 m^3) of concrete every 5 min. The entire 700 cu yd (535 m^3) can be supplied in 6 hr, which is within the 7 hr allotted. Then, using a horizontal screw and a row of internal vibrators mounted on a self-propelled unit, the concrete can be quickly spread and consolidated as it is supplied. As this is a sublayer, no additional screeding or finishing is proposed.

Referring to Figure 4, a 7-hr cure period for base 2 is

* Anticipated costs do not appear to be excessive in terms of anticipated benefits.

provided in the time schedule. Although this is not needed before placing a surface layer on the lean concrete, 5 of the 7 hr are during a peak-traffic period when construction is restricted to one-lane operation. Thus, where such restrictions exist, the rehabilitation time cannot be further compressed.

3. Lean, Rapid-Hardening, High Early-Strength Concrete Base.—From the time schedule (Fig. 4) it would appear that the use of rapid-hardening, high early-strength, concrete bases would not be justified because of the delay caused by peak-traffic periods. However, in those cases where two-lane closures can be permitted continuously throughout the construction period, rapid-hardening, high early-strength, concretes do offer a time savings (see Appendix C). Should this approach be selected, ready-mix trucks should not be employed, as the rapid-hardening cements have too short an initial set time (see Appendix C). About the only equipment suitable for mixing this type of concrete is a continuous, auger-mixer system such as the Concrete-Mobile, shown in Figure 9, manufactured by Irl Daffin Associates, Inc. A specification for this system has been adopted by ASTM (ASTM C-685) and a 10-cu yd (8-m^3) unit can mix and discharge 1 cu yd (0.8 m^3) of concrete every 2 min. Using four of these units at any given time, 700 cu yd (535 m^3) of concrete can be placed in less than 7 hr. These units could be spaced in pairs (one pair somewhat behind the other) and each pair would be followed by a horizontal screw-auger/gang vibrator unit to quickly consolidate the concrete before it sets.

4. Sulphur-Asphalt-Aggregate Systems.—These systems offer exciting potentials for savings in energy, cost, and time. They are still new and experimental, but results to date indicate considerable promise (see Appendix C). From the construction viewpoint, such systems can be mixed and placed using essentially conventional hot-mix equipment. The thickness of the layers that can be effectively placed is dependent on both cooling rate and sulphur content. At sulphur contents above 35 volume-percent of the binder, nonuniform cooling and flow of hot liquid sulphur by gravity could produce a nonhomogeneous mix with sulphur-rich layers at the low points in the pavement. For these reasons, mixes with sulphur contents above 35 volume-percent should be placed in no greater than 3-in. (0.08-m) lifts. High-sulphur content mixes (up to 50 volume-percent) are recommended primarily for use on poorly-graded, high void mineral aggregate (VMA) sands which have the added advantage of not requiring compaction. Given the constricted area imposed, elimination of compaction equipment is extremely advantageous. Sulphur contents lower than 35 volume-percent are recommended for dense-graded aggregate mixes that can be placed in thicker lifts. These mixes do, however, require conventional compaction to achieve desirable air void contents. The sulphur-asphalt-aggregate systems would be mixed in a conventional pugmill, transported to the job in conventional trucks, and discharged into a modified hot-mix, lay-down machine. The major modification applied to this machine is the addition of equipment to unload the screeds. Once through the lay-down machine, the mixture would be allowed to cool until it solidified; then another layer could be placed. For sulphur-asphalt systems using dense-graded quality aggregates, it is recommended that an 8-in. (0.20-m) layer be placed followed by a 6-in. (0.15-m) layer, much the same as the hot-mix asphaltic concrete base.

5. Polyurethane Foam System.—This system, although very expensive, offers two important advantages: (1) it can be employed very rapidly and (2) it can be placed on a very weak subgrade to provide a working table for subsequent operations. For the assumptions stated in Chapter One, polyurethane foam would not be used. However, if the subgrade were very weak and could not support construction traffic, then a thin layer of polyurethane foam—placed pneumatically as a liquid and mixed at the nozzle with a foaming agent—could be placed and allowed to "foam up" to a predetermined thickness (see Appendix C). Using a 6 to 1 ratio of foamed volume to liquid quantity, the resultant foam would have a density of approximately 12 pcf (190 kb/m^3), a modulus of elasticity of about 13,000 psi (89,600 kPa), and a tensile strength of about 360 psi (2,500 kPa). It can be mixed and placed by portable units and cured in less than 90 min. Once in place, subsequent layers could be placed as described in previous sections.

6. Combination Systems.—For particular situations, combination systems could be advantageously employed. For example, a system consisting of a thin layer of polyurethane foam, followed by an asphaltic concrete base, followed by a lean portland cement concrete base might be used where a contractor had the various types of equipment available and could effectively use it.

Operations for In-Place Rehabilitations

Even though this case involves sublayers that are structurally unsound, in many cases the sublayers will be of such quality that they can be rehabilitated in place. Under such conditions, the operations involved (given in Table 1) include manipulation of the granular base, addition of a stabilizer, and curing of the base. Each of these activities is discussed in the following paragraphs.

Code D. Manipulation of Granular Base.—Manipulation, as used here, means loosening the base and changing its moisture content. Referring to Figure 4, approximately 7 hr can be spent for this activity without exceeding the time limits. With only 400 cu yds (300 m^3) of base involved, no innovative technology is needed to loosen and remix the base. Rotary disc mixers, such as the Pulver mixer, can be employed to accomplish the task. If additional water is needed, it can be easily provided; however, if the base is too wet, some innovative technology may be needed. For these cases, a heater-planer with ripping teeth attached is proposed. This piece of equipment can dry the aggregate to a degree sufficient for the addition of a stabilizing agent (see Appendix E). The unit would only have to travel about 4 ft per min (0.02 m/s) to complete the operation in less than 7 hr.

Code H, I. Addition of Stabilizer.—Acceptable stabilizers include asphalt, emulsions, lime, portland cement,

and sulphur—the first three of which are widely used and accepted. It is sometimes difficult to obtain desired uniformity with field mixing of emulsions and aggregates. It is, however, both cost and energy effective, and the resulting layer has improved strength and durability once the emulsion "breaks."

Lime stabilization is inexpensive and easy to construct. The strength improvement to the base normally is much less than with the other stabilizing agents, and relatively long time periods are required for the lime reactions to be completed (*16*). In those cases where only limited improvements (within the time constraints) in base quality are required, lime stabilization should be strongly considered. Where the base is quite wet, quicklime (CaO) might well serve to both dry up the base and stabilize it.

Stabilization of granular materials with portland cement is an accepted practice (*16*). It is cost effective and can be completed fairly rapidly. The cement can be placed on the surface, water added, and then mixed with the base using a rotary disc mixer. Acceptable uniformity of mixing can easily be achieved. Once mixed, the base is compacted with standard compaction equipment and allowed to cure. Although this curing takes time, the granular mass can usually take construction traffic immediately without damage so rapid hardening cements would not generally be needed. Where significant strength gains are required in the base layer, portland cement stabilization offers some significant advantages.

Sulphur is a new, promising and innovative stabilizer for granular materials (see Appendix C). One innovative idea, if the base is in fairly good shape and relatively porous (such as that used in the so-called free-draining subbases), is to pour molten sulphur on the surface of such a base that has been heated and dried. The molten sulphur should infiltrate the base until the sulphur cools. In most conditions, the base will be wet and relatively cool. If molten sulphur is poured on such a surface it will immediately solidify and prevent infiltration of any additional sulphur that might be added. To prevent this, the porous base must first be heated and dried. This heat can be applied with the traveling heater-scarifier, shown in Figure 10, which should drive off the water. Probably relatively long heating times will be required to heat the aggregate sufficiently. Once heated, the molten sulphur, added quickly, should infiltrate the base approximately 3 in. (0.03 m). If the base is dense and relatively nonporous, then it needs to be loosened in place before heating and the addition of any sulphur. Using a rotary disc, the material can be rapidly loosened, then heated with the heater-scarifier. Following immediately, the molten sulphur can be applied, allowed to impregnate the base, and cooled (although relatively long cool-down times may be necessary). The stabilized base, though not as strong as a portland cement stabilized base, might be made significantly stronger than the original granular base, and the treatment should be quite cost effective. New technology is needed to verify this idea and evaluate its potential.

One other innovative approach would be to use cement slurries—made with rapid-hardening, high-strength cements—in the same manner as proposed for the sulphur in that the slurry would infiltrate the base and rapidly stabilize it in place. The base would need to be porous enough and the voids dry enough to accept the slurry. The U.S. Air Force is reported to be pursuing this idea for runway repair.

Code J. Curing of the Base.—In this operation, enough time must be provided for the stabilized base to cure (or cool) sufficiently to withstand the loads imposed upon it during placement of the surface layer. For asphalt, lime, or portland cement stabilizing agents, the aggregate interlock will usually be sufficient to prevent damage to the stabilization process. Sulphur, however, must crystallize before any loads are imposed, or the structure may become fractured. Thus, sulphur-impregnated systems should be allowed to cool to at least 200 F (93 C) before subsequent layers are placed. Within the time limits imposed for this activity, sufficient cool-down time is provided for the sulphur system, should it be selected.

Code K, L, M, N. Rehabilitation Materials/Systems for the Surface Layer

Once the sublayers have been rehabilitated, either in-place or by removal and replacement, then the last major activity is rehabilitation of the surface layer. Referring to Figure 4, 5 hr are allocated for the placing and finishing operation, and 12 additional hr are allotted for curing (or cooling) this surface layer. Promising candidate materials/systems are listed in Table 3 and discussed in the following paragraphs.

Hot-Mixed Asphaltic Concrete.—Hot-mixed asphaltic concrete (HMAC) surfaces can be placed and compacted in much the same manner as HMAC bases except that closer vertical tolerances must be maintained. For the assumed condition, a 10-in. (0.25-m) thick surface layer is needed. Standard practice calls for this surface layer to be placed in three lifts of 5½ in. (0.14 m), 3 in. (0.08 m), and 1½ in. (0.04 m) (*17*). One asphalt paver can place all three lifts in the 17-hr placement cool-down time allotted, even though there is a 3-hr period during which the construction operation is restricted to a single lane. The only problems envisioned would be the steady supply of HMAC through heavy urban traffic, and the provision of enough time for the mix to cool sufficiently to support traffic (see Appendix C). By overcoming these problems, this material/system offers one solution to the problem.

Portland Cement Concrete, Cast-In-Place.—Use of portland cement concrete, cast-in-place, offers many advantages; namely, it is the same material that was removed, which makes the rehabilitation aesthetically acceptable, it requires no new technology, and it is cost effective. The one major disadvantage is the cure time necessary before the section can be opened to traffic. In order to cut this cure time to 12 hr, Type III cement with a strength accelerating admixture (see Appendix C) is proposed. Transportation of the concrete can be achieved in concrete ready-mix trucks, but the water and admixture should not be added until the truck reaches the construction site. This precaution will provide maximum time for placement and finishing of the concrete. An 8-cu yd (6-m^3) mixer can

Figure 9. Concrete mobile.

Figure 10. Jim Jackson heater scarifier.

maneuver and discharge its load in less than 5 min. Using two mixers discharging more-or-less simultaneously, some 16 cu yds (12 m^3) of concrete can be discharged every 5 min. Because approximately 500 cu yds (380 m^3) of concrete are needed, ready-mix trucks can ideally deliver and discharge this quantity in 2.6 hr—much less than the 5 hr allotted. Using a slip-form paver, such as the CMI Super Paver shown in Figure 11, this machine would have to advance at a rate of 4.4 ft per min (0.02 m/s) to complete the paving in 5 hr. These machines can routinely pave at speeds of up to 20 ft per min (0.10 m/s), hence, ample time is allotted for this activity. Following the paver would be the finishing and curing machine. Then, as soon as the concrete hardened sufficiently (around 12 hr in moderate climate), the joints could be sawed and the surface striped.

Portland Cement Concrete Panels, Precast.—The use of precast elements reduces the cure time required, and have been successfully employed for small patches (see Appendix C). However, to the authors' knowledge such

TABLE 3

PROMISING MATERIALS/SYSTEMS FOR THE SURFACE LAYER [a]

MATERIALS/SYSTEMS	REMARKS
1. Hot-Mixed Asphaltic Concrete	Easily constructed, cost effective, no new technology required, needs time to cool.
2. Portland Cement Concrete, Cast-In-Place	Easily constructed, cost effective, no new technology required, needs time to cure.
3. Portland Cement Concrete Panels, Precast	Difficult to construct, relatively expensive, no new technology required, no cure time needed.
4. Rapid-Hardening, High Early-Strength Concrete, Cast-In-Place	Difficult to construct, expensive, new technology required, cure time is reduced.

[a] See Appendix C for additional details.

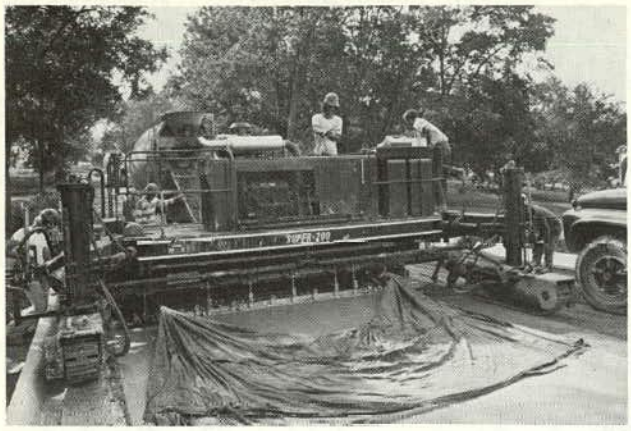

Figure 11. CMI slip-form paver.

elements have not been used in an extensive replacement system such as the 1,320-ft (400-m) system proposed here. Innovative technology is needed to more rapidly place and set the panels in the grout bed to the correct grade. The system proposed here is to provide sublayers to within 7 in. (0.18 m) of the surface. This would then be followed by pump placement of a 1-in. (0.03-m) thick portland cement grout bed on which the concrete panels would be placed. These panels, 21 ft by 12 ft by 0.5 ft ($6.4 \times 3.7 \times 0.15$ m) in size, constructed with reinforced, structural lightweight concrete, would each weigh approximately 15,000 lb (6,800 kg). Three such panels could be carried on a flatbed truck, and a total of 63 slabs would be required for the section. Using a 35-ton (32,000-kg) capacity crane, each panel could be secured, lifted, and placed in approximately 10 min., assuming a tolerance of ¾ in. (0.02) on both ends and one side. For a 10-min placement time per panel, the entire 1,320-ft (400-m) section can be placed in less than 11 hr, which is 8 hr less than the 19-hr placement and cure time allotted. Immediately following placement of each panel, a crew would check alignment, correct as necessary, and place a flexible sealing compound around each panel. This material/system can cut approximately 8 hr off the rehabilitation time—provided no delays are encountered. The major disadvantages are the relatively high probability that delays will occur, the need for an expensive joint-sealing compound around each panel, the lack of load transfer between each panel (which must be accounted for in the design), and the fact that the joint spacing in adjacent lanes will not match the joint spacing in the rehabilitated sections.

Rapid-Hardening, High Early-Strength Concrete, Cast-in-Place—This material/system offers the major advantage of reducing the required cure time by 8 hr, as adequate strengths are achieved in 4 hr in moderate climates (see Appendix C). The problems and construction techniques have been discussed previously in the section on replacement materials/systems for sublayers. As soon as the surface is strong enough to support the equipment (in approximately 2 hr), transverse joints can be sawed and the surface striped.

Code O. Remove Barrier

This final activity can be performed in the same manner as the barrier erection, and 5 hr are again allotted for the operation. The barrier system can be removed completely or moved down the roadway to another section to be rehabilitated, depending on the situation.

PROPOSED SOLUTIONS FOR SURFACE LAYER STRUCTURALLY UNSOUND—SUBLAYERS STRUCTURALLY SOUND

General

Additional assumptions made for this particular problem are:

1. The pavement to be rehabilitated consists of either 8 in. (0.20 m) of hot-mixed asphaltic concrete or a 10-in. (0.25-m) thick unreinforced concrete pavement, both resting on a structurally sound sublayer.
2. The vertical alignment of the rehabilitated surface will conform to the adjacent lane.
3. A ¼-mile (400-m) section of outside lane will be rehabilitated.

The solution to this problem involves the following eight activities (the coding follows that given in Fig. 4):

CODE	ACTIVITY
A	Barrier Erection
B	Breaking Pavement
C	Loading and Removing Pavement
K	Transport/Mix Surface Layer
L	Place/Finish Surface Layer
M	Cure Surface Layer
N	Stripe Surface
O	Remove Barriers

These eight activities are shown in analysis bar chart form in Figure 12. The activities are scheduled to begin at 8 PM with the erection of the barrier system. This time was selected because, generally, the peak-traffic period will have ended, and the construction activity can commence with minimum interruption to traffic flow. The ¼-mile (800-m) section is scheduled to be completed and reopened to traffic in about 26 hr.

In the following paragraphs, each activity is described giving the materials/systems proposed for use and the construction technologies required to complete the activity within the time allotted. Where the activities for this problem coincide with the activities of the previous problem (all layers structurally unsound), the reader is referred to the preceding applicable discussion.

Code A. Barrier Erection

(See previous discussion.)

Code B. Breaking Pavement

This activity involves the breaking of 1,760 sq yd (1,470 m²) of pavement in 3 hr. To do this the pavement must be broken at a rate of close to 600 sq yd per hr (500 m²). For portland cement concrete pavement, removal methodology is given in the previous section under "Proposed Solutions for all Layers Structurally Unsound." For hot-mixed asphaltic concrete pavement, removal methodology is similar, yet somewhat simplified as this type pavement is fairly easily broken. An excellent machine for breaking pavement is the Gradall, Model 880 (Fig. 7) which can tear and pick up the pavement without the need for prior break up. At each end of the section a saw cut might be necessary, and a portable concrete saw, such as the Clipper, Model 655 (Fig. 6), is recommended for this operation.

Code C. Loading and Removing Pavement

For concrete or bituminous pavements, the Gradall, Model 880, is recommended to pick up the material and load it onto trucks for removal. Machines of this size are needed because only 4 hr are allocated to remove some 500 cu yd (400 m³) of consolidated concrete or some 400 cu yd (300 m³) of consolidated hot-mixed asphaltic concrete.

Code K, L, M, N. Rehabilitation Materials/Systems for the Surface Layer

These activities have been fully discussed previously, and, therefore, will not be reiterated here. However, it should be pointed out that should a rapid-hardening, high early-strength concrete pavement (cast-in-place or precast portland cement concrete panels) be used, the total rehabilitation time can be cut approximately 8 hr. This results in a total lane closure of 18 hr for rehabilitating a ¼-mile (400-m) section of the outside lane of a highway.

Code O. Remove Barrier

(See previous discussion.)

PROPOSED SOLUTIONS—ALL LAYERS STRUCTURALLY SOUND

When the pavement structure is sound and only the surface requires treatment to restore riding quality or skid resistance, the usual practice is to overlay the pavement. If overhead restrictions prevent the use of an overlay, then partial-depth rehabilitation must be considered.

Additional assumptions made for this particular problem are:

1. The pavement surface is either wavy with the waves and sags not exceeding 2 in. (0.05 m) above or below the average pavement height, or the surface is deteriorated (spalled, ravelled, rutted, etc.). In either case, all pavement layers are structurally sound.
2. A ¼-mile (400-m) section will be rehabilitated.

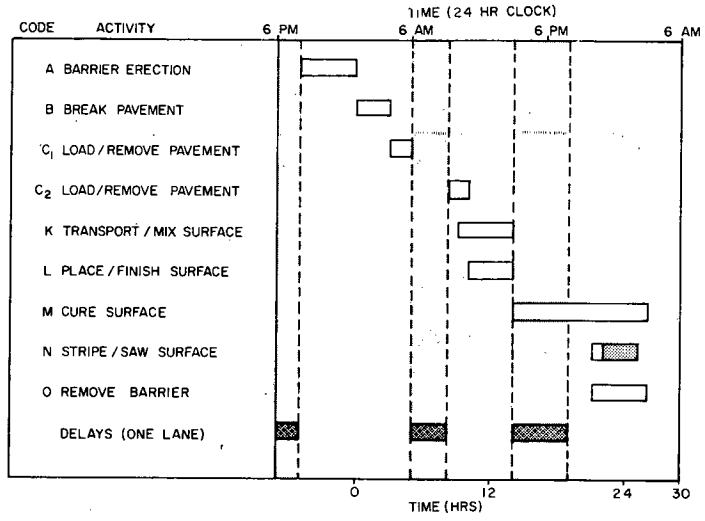

Figure 12. Analysis bar chart for highway rehabilitation—surface layer structurally unsound, sublayers structurally sound.

Portland Cement Concrete

For portland cement concrete surfaces, the solution involves the following eight activities (the coding follows that given in Fig. 4):

CODE	ACTIVITY
A	Barrier Erection
B	Break Pavement (Grind)
C	Load/Remove Pavement
K	Transport/Mix Surface
L	Place/Finish Surface
M	Cure Surface
N	Stripe/Saw Surface
O	Remove Barrier

These activities are portrayed in analysis bar chart form in Figure 13. The section is scheduled to be completed and reopened to traffic 24 hr after barrier erection.

In the following paragraphs, each activity is described giving the material/systems proposed for use and the construction technologies required to complete the activity within the time allotted. Where the activities coincide with activities described earlier, the reader is referred to the previous discussion.

Code A. Barrier Erection

(See previous discussion.)

Code B. Break Pavement (Grind)

This activity involves grinding 1,760 sq yd (1,470 m²) of the pavement surface to an average depth of 2 in. (0.05 m) below grade line. Grinding can be accomplished in approximately 6 hr by using four Galion RP 30 road planers shown in Figure 14. As before, a portable concrete saw may be required for vertical cuts at each end of the section.

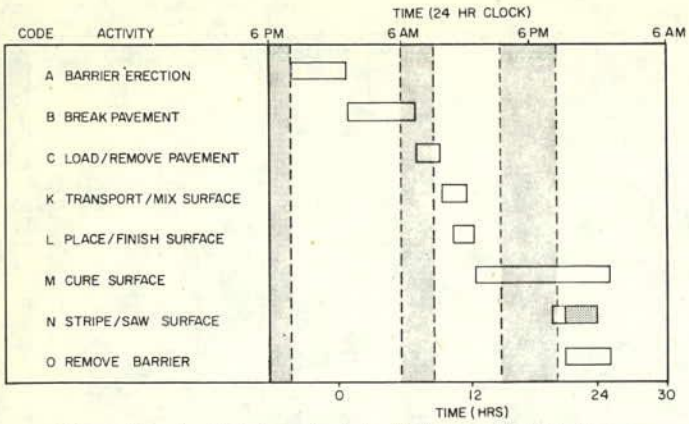

Figure 13. Analysis bar chart for highway rehabilitation—portland cement concrete surface deteriorated.

Figure 14. Galion road planer RP–30.

Code C. Loading and Removing Concrete

For removal of the broken material, any small front-end loader will suffice. Removal of about 100 cu yd (75 m^3) of loose material is required. With only a 2-in. (0.05-m) drop-off, haul trucks will be able to enter or leave the system at any point along the section and, hence, this activity can be restricted to one lane. After loading, a power sweeper could be used to remove the remaining fine material; 2 hr are allocated to this operation.

Codet K. Transport/Mix Surface Layer

An acceptable surface layer has been shown to be steel-fiber-reinforced concrete using a Type III cement with an accelerator (see Appendix C). It should be noted that 2 in. (5 mm) is considered the minimum thickness for this material and, in general, unbound overlays have better performance. Approximately 100 cu yd (75 m^3) will be required. The concrete can be hauled in transit mix trucks and placed on grade directly in front of a paver as described previously. Also, rapid-hardening, high early-strength cement could be used to reduce curing time (see earlier discussion).

Code L, M, N, O. Rehabilitation Materials/Systems for Surface Layer

These activities are fully discussed in an earlier section and will not be reiterated here.

Asphaltic Concrete Surfaces

For asphaltic concrete surfaces, the solutions involve the following five activities (the coding follows that given in Fig. 4):

CODE	ACTIVITY
A	Barrier Erection
L	Place/Finish Surface
M	Cure Surface
N	Stripe/Saw Surface
O	Remove Barrier

These five activities are shown in analysis bar chart form in Figure 15. The section is scheduled to be completed and reopened to traffic 14 hr after barrier erection. In the following paragraphs each activity is described as before.

Code A. Barrier Erection

(See previous discussion.)

Code L. Place/Finish Surface Layer

Because the pavement depth is 2 in. (0.05 m), a repaver is proposed to reprocess the existing surface material, add appropriate juvenators, and add aggregates or hot mix where required. The Cutler Repaver shown in Figure 16 has been shown to be adequate (see Appendix B). It was selected because it is a self-contained unit and, for the 1,320-ft (400-m) section, it can carry the necessary materials. After placement, a steel wheel roller can be used for compaction.

Code M. Cure Surface Layer

For the 2-in. (0.05-m) lift, less than 1 hr is required for adequate cool-down.

Code N and O. Stripe Surface and Remove Barriers

It should be noted that this process lends itself to a continuous operation. That is, with proper planning, significantly more than 1,320 ft (400 m) can be treated during the 14-hr period. This is accomplished by moving barriers to the end of the section (or supplying additional barriers) as long as the work section is not too far removed from the first warning signs and supply trucks can gain periodic access to the repaver.

Restoration of Skid Resistance Only

In many cases the pavement surface will have worn down and become slick, but, otherwise will be satisfactory; for such cases, restoration of skid resistance is all that is required.

Many systems have been successfully used to restore skid resistance to asphaltic concrete pavements (*18*). Most of these are "seal-coat" types of operations which add less than ½ in. (0.01 m) to the thickness of the pavement. In a majority of the situations this could be tolerated by overhead obstructions. For high traffic volumes, consideration should be given to the use of hard, durable, nonpolishing aggregates. Such expensive aggregates as bauxite (which would have to be imported) may be justified in some cases if their use will provide a much longer service than locally available igneous and noncarbonate rocks and rock blends containing such rocks as trap rock and hard sandstones. These processes are well-documented and cost effective and can be performed at off-peak hours with a minimum of traffic delay.

For portland cement concrete pavements, grooving is the most successful and cost effective method of reducing wet-weather skidding accidents. Also, the British report good results from a seal-coat type of process using epoxy to bind bauxite aggregates to a portland cement concrete surface (*19*). However, such processes are expensive and do not seem to be significantly more effective than grooving. The grooves are generally sawed patterns ⅛ to ¼ in. (3 to 6 mm) deep, and are usually produced by diamond bit blades (*20*). A British firm (Errut Limited, London) has manufactured a groover which uses a flailing process to produce the grooves. In this process carbide teeth groove the pavement by impact rather than by cutting.

A diversity of opinion exists as to whether the grooves should be longitudinal or transverse. Both will reduce wet-weather skidding accidents; which is more effective is, as yet, unverified. Transverse grooves require more time to produce and generally will be more expensive. California contracts for 1973 show grooving to cost $0.46 per sq yd.

FUTURISTIC SYSTEMS

All the previously described materials/systems to remove and replace significant portions of the pavement structure have one major drawback—the requirement that two lanes be closed during off-peak periods. Such lane closing is necessary even though innovative concepts have been employed. To restrict the closure to only one lane throughout the rehabilitation period would require the introduction of systems not presently available or tried.

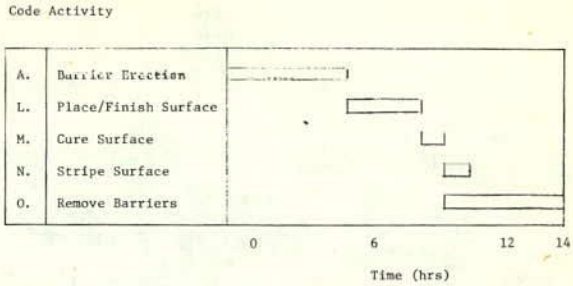

Figure 15. Analysis bar chart for highway rehabilitation—asphalt concrete surface deteriorated.

Hence, these systems are termed "futuristic" in that major, expensive, technological developments are necessary to put them into practice; three such systems are discussed in the following sections.

Horizontal Overhead Tunnel Approach (HOTA)

This system requires the removal of deteriorated sections of pavement and sublayers in a tunnel-type fashion and then the transportation of the material to haul units either ahead of or behind the excavation. Mining shovels, such as the O&K RH 25 H.D. Shovel shown in Figure 17, could remove concrete pavement previously broken by a CMI H-16 Super Hammer (Fig. 5). This shovel could then load the broken pavement into dump trucks stationed ahead of the shovel as shown in Figure 18. This shovel is 10.8 ft (3.3 m) wide, so it would probably have to be slightly modified to fit into a 12-ft (3.7-m) wide trench without hitting the sides.

Following removal of the pavement and sublayers (if necessary), replacement would be accomplished by pumping appropriate materials through large diameter pipes into the trench. Pumps, such as the Pecco Putzmeister Model PMP 160 for concrete shown in Figure 19, could be located on overpass structures or adjacent to the travelway. The materials would then be transported by flexible pipe over the traffic (if necessary) to the rehabilitation section. This pump has a theoretical rated output of 160 cu yds per hr (130 m^3) with a practical output of 110 cu yds per hr (85 m^3) through a 9-in. (0.2-m) line. The replacement materials in such a line would weigh up to 70 lb per lineal ft (105 kg per m). This would require a powered hoist to move the line as the materials are placed. Initially,

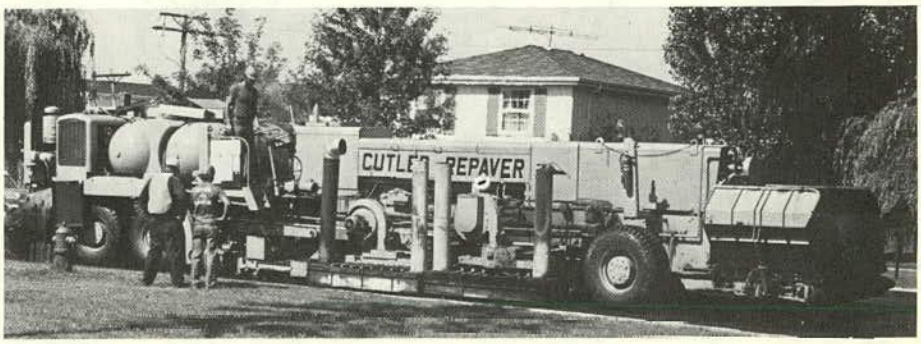

Figure 16. Cutler repaver.

Figure 17. Mining shovel, O&K RH 25 H.D.

the entire line would be laid out and then sections would be removed as replacement proceeded. Materials specifically suited for this type of delivery include the lean concrete base and the portland cement concrete pavement. By placing the pump off the travelway, potential bottleneck conditions resulting from material deliveries are reduced, and rehabilitation can proceed with only a one-lane closure. Conceptually, a ¼-mile (400-m) section of pavement could be completely removed and replaced in 44 hr as shown by the analysis bar chart in Figure 20. The cost of such an approach would depend on equipment availability, inasmuch as this equipment is not commonly used by most highway contractors. New technology needed includes:

1. Modified mining shovel.
2. Means to rapidly assemble and disassemble pipe for the pump.
3. Pumping lean concrete.

Moving Barrier Support Systems (MBSS)

This system requires the development of a moving barrier system inside of which the necessary rehabilitation operations are carried out. Such a system is schematically shown in Figure 21 and Figure 22, and is an amplification of a system first conceived by a group of graduate civil engineering students at Texas A&M University. The system consists of a moving barrier support frame 150 ft (45 m) long, attached to the chassis of an H-16 Super Hammer. This frame has the following three functions: (1) to provide a visual screen between the rehabilitation work and the adjacent traffic; (2) to support the two conveyor systems; and (3) to support the loading clamshell. This frame would come to the job in five, 30-ft (9-m), sections and, after assembled, would move in a

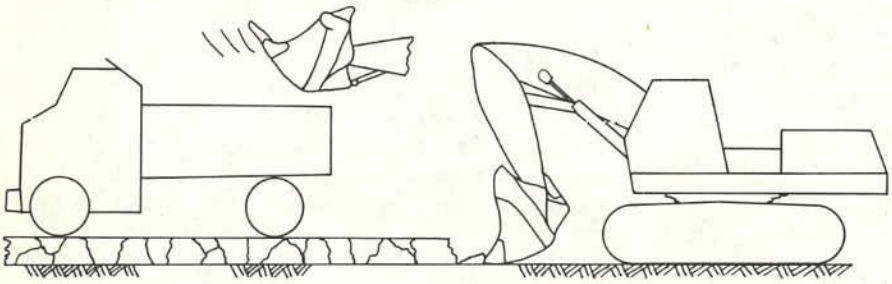

Figure 18. Loading arrangement using a mining shovel.

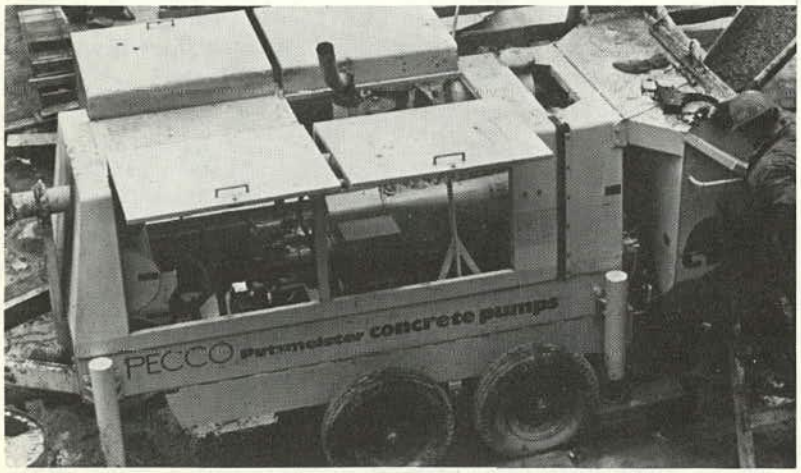

Figure 19. Pecco Putzmeister concrete pump, Model PMP 160.

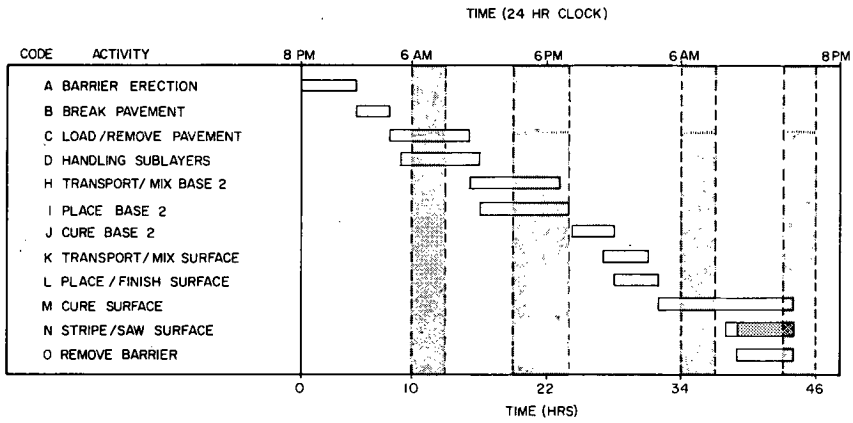

Figure 20. Analysis bar chart for highway rehabilitation, HOTA materials/systems.

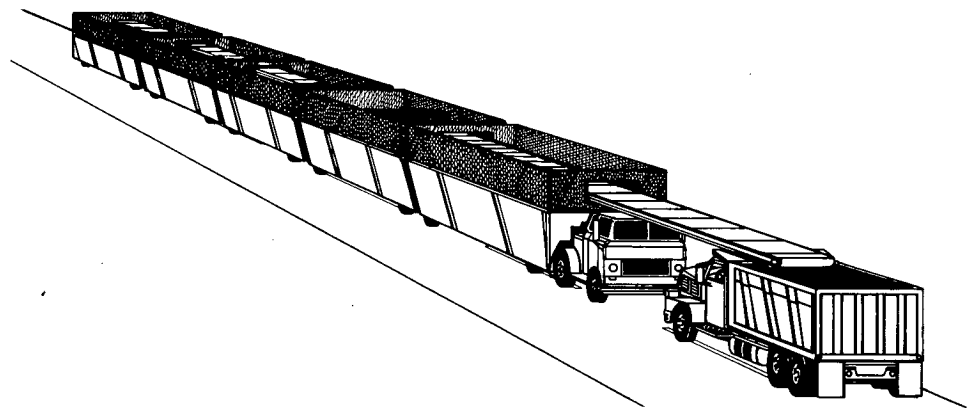

Figure 21. Moving barrier support system (MBSS).

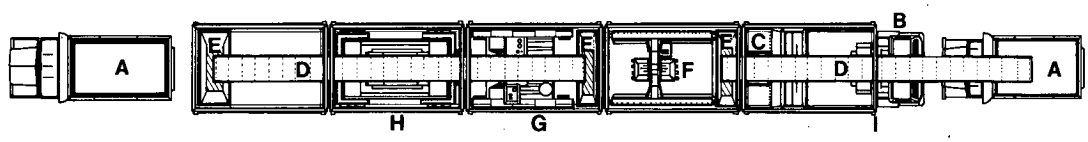

Figure 22. Plan view of MBSS showing components.

Legend:

- A. DUMP TRUCK
- B. HYDRA HAMMER
- C. CONCRETE FLAYER - NEW TECHNOLOGY REQUIRED
- D. CONVEYOR
- E. HOPPER
- F. LOADING CLAMSHELL
- G. PAVER
- H. COMPACTOR / FINISHER
- I. MBSS FRAME - NEW TECHNOLOGY REQUIRED

direction counter to the flow of traffic. At the front of the system, a conveyor would transport removed material to waiting dump trucks for disposal. At the rear of the system, dump trucks would be feeding in replacement materials.

Such an approach could be used to remove and replace pavement surface layers of either portland cement concrete or asphaltic concrete. The system could also be used to replace both the surface layer and an underlying layer, provided the replacement material could be placed in a single lift. Conceptually, such a system could move at a rate of approximately 1.8 ft per min (0.009 m/s), which is the rate of advance of the super hammer. The super hammer would punch holes in the concrete pavement and initially fracture the concrete. Following the hammer would be a "concrete flayer" machine. This machine, not presently available, would need to have the capability of breaking the pavement down from 2-ft (0.6-m) sized pieces to 6-in. (0.15-m) sized pieces. New technology is needed to develop such a machine that will accomplish this task efficiently and rapidly, and at the same time be durable. The term "flayer" is used to denote one method that might be used—impact hammers on a rotating shaft "flaying" the concrete surface. Following the flayer machine, a frame-mounted clamshell would pick up the broken concrete pieces (and sublayers if required) and

deposit them into a hopper. The clamshell, hopper, and conveyor subsystems are connected to the MBSS frame in order that the forward motion of the frame will not cause a clamshell load to miss the hopper. The clamshell will move back and forth on tracks and from side to side. Thus, almost all deteriorated pavement material can be readily removed. The small amount of material remaining can be left without harming the replacement material or the pavement structure.

Following the clamshell is an automatic, grade-leveling, slip-form paving machine. Concrete is supplied to the paver by conveyor belt from hopper E which is supplied by dump trucks or ready-mix trucks. What is envisioned here is similar to that shown in Figure 11. The size of the machine would need to be reduced to make it fit into the 12-ft (3.7-m) width. It should be able to handle all the materials proposed for use in the rehabilitated pavement.

Following the paver is a final compaction machine (for asphaltic concrete) or a finishing/curing machine (for rapid-hardening, high early-strength concrete).

At the rear of the MBSS is a receiving hopper and conveyor subsystem for the receipt of replacement materials and the conveyance of these materials to the paving machine.

Such a system as the MBSS would be approximately 150 ft (45 m) in length. It must be capable of maintaining proper alignment as it moves along either tangent sections or curved sections.

To facilitate hauling unit egress and ingress, the MBSS will move in a direction counter to the flow of traffic. As the construction operations are all enclosed in the system, traffic handling ahead of and behind the MBSS could be accomplished by using mobile sequential arrow units.

One major drawback to this concept is that if the machine moves at approximately 1.5 ft per min (0.008 m/s), the new pavement will be exposed to the stress of haul units bringing in new material at an age of approximately 1 hr. If asphaltic concrete is used, then a tandem wheel roller, following the MBSS, could be employed to smooth out any irregularities that were introduced in the pavement surface by the haul units. On the other hand, if a rigid type of pavement were required, rapid-hardening, high early-strength concrete, mixed through a continuous type of mixer, such as the Concrete Mobile (Fig. 9), would appear to be the only potential material to use.

New technology needed for the moving barrier support includes:

1. A frame subsystem that will support the conveyors and moveable clamshell.
2. A frame subsystem that will track properly as it moves along the roadway.
3. A concrete "flayer" machine that will efficiently break down concrete pavement from 2-ft (0.6-m) sized pieces to 6-in. (0.15-m) sized pieces.
4. A modified slip-form paver, capable of placing hot-mix asphaltic concrete (HMAC) and rapid-hardening, high early-strength concrete materials.

Mobile Roadway Repair Unit

This concept consists of a self-propelled reconditioning factory which, by use of a bridge over the repair site, provides pavement rehabilitation with minimal disturbance to through traffic. This concept was developed by Professor Rollin C. Dix of the Illinois Institute of Technology, and has been patented (U.S. Patent 3.811, 147 titled "Mobile Roadway Repair Unit") (*21*). The vehicle is essentially a 4¼-ft (1.4-m) high by 160-ft (50-m) long bridge which is moved into place in four sections and remains stationary while work is proceeding. Following completion of the work, the unit will move down the roadway at a speed of approximately 2 mph (0.9 m/s). Traffic movement over the bridge can continue whether the unit is stationary or moving. The artist's conception of the unit is shown in Figure 23.

The concept offers several advantages. One, no lanes are completely closed at any time (except when assembling the system and resupplying the system with paving material). A second advantage is that repair can proceed without interruption or undue hazard using conventional material/systems to effect the rehabilitation. A third advantage is the mobility of the system and enclosure of all equipment, operators, and material within the unit. Concerning the first advantage, any system that can keep all lanes open is certainly a desirable goal, especially when considering the very high traffic volumes in many urban areas. The second advantage means that the most cost effective materials can be used even if they require the unit to remain stationary an extra few hours for these materials to attain sufficient strength to be subjected to traffic. The third advantage means that minimum traffic disturbance will result.

The mobile roadway repair unit concept has two major disadvantages. First, the cost involved in developing it to the point where it can be put into commercial use, and second, the restrictive working height [4½ ft (1.4 m) for workmen in the unit (this height might be raised to 5½ ft (1.7 m) without decreasing traffic flow]. The first disadvantage (which is the same for any new concept) can be overcome only by the desire of some agency or industry to underwrite the development cost. The second disadvantage only means that work inside the unit will be uncomfortable, and hence, labor cost will be higher than normal.

Discussion of Futuristic Systems

The development of new and advanced materials/systems is an expensive, time-consuming process. The research team discussed concepts, such as the three proposed previously, with numerous contractors, equipment manufacturers, and highway engineers. One contractor interviewed discussed his concept of a system to continuously remove, upgrade, and replace asphaltic concrete pavement (*22*). In his opinion, the development cost for such a system could easily escalate to $1,000,000. An equipment manufacturer indicated that his company seriously considered developing rapid pavement rehabilitation systems, using its knowledge of mining equipment (*23*). However,

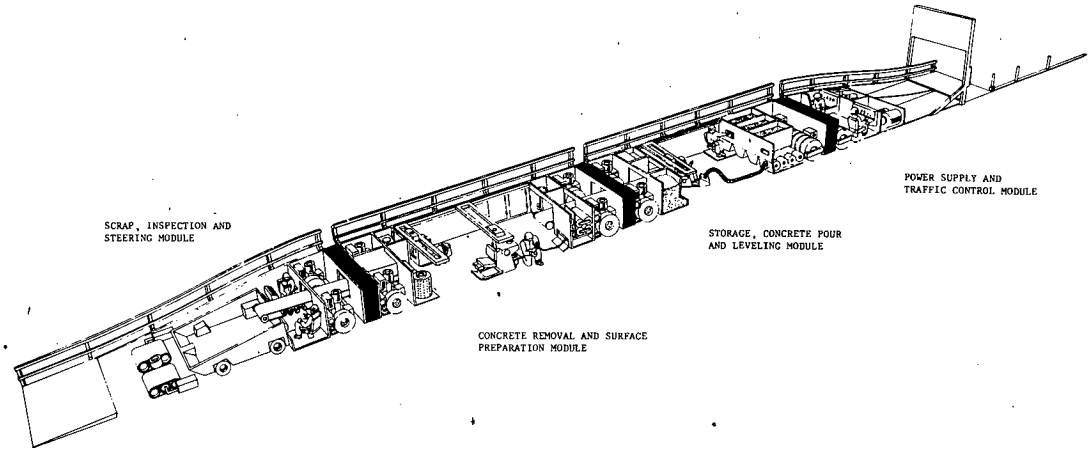

Figure 23. Artist's conception of the mobile roadway repair unit (20).

after study, the company decided the market was too limited to justify a major development expenditure. Another contractor stated that no contractor could afford to own such specialized and expensive equipment (even if it did exist) unless he had a guaranteed market for its use (*24*).

Thus, the researchers are forced to conclude that the possibilities for development of advanced technology materials/systems are somewhat slim at this time—if private industry has to bear the whole cost of development. Before such systems can be developed, it would appear that partial financing from public agencies with specific needs would be required, and, once developed, the system would be made available to qualified contractors for use.

CHAPTER THREE

APPRAISAL AND APPLICATIONS OF FINDINGS

Any appraisal of the findings of Chapter Two must begin with the objectives of the problem and the accompanying constraints imposed. To rehabilitate a significant portion of pavement structure in less than 48 hr requires new and innovative technology—new technology in terms of materials to be used and in terms of construction systems to be employed. The 48-hr time limitation was the first constraint faced by the researchers. A second constraint was the requirement for a one-lane closure only, during peak-traffic periods. And a third major constraint was the elimination of overlays for pavement rehabilitation. These three constraints forced the researchers to critically examine all aspects of pavement rehabilitation. Current state-of-the-art rehabilitation technology involves significant closure times (1 to 2 weeks are normal) and multiple lane closures (at least two), if the pavement structure is to be removed in any quantity, or a relatively thick overlay for rapid, single-lane, repair. Also, present technology includes rapid spot-patching using new and innovative techniques, as discussed elsewhere in this report.

Examination of the literature and detailed investigation of current rehabilitation procedures throughout the United States revealed that no one was employing techniques that would meet the objectives. The natural question was "were the objectives unrealistic?" Fortunately, the answer was "no!" Using new and innovative technology, solutions are presented that will meet the objectives of the research study; however, no single, unique solution emerged. Because of the vastly different situations existing throughout the United States, a specific solution had to be developed for certain conditions, and a different solution had to be developed for a different set of conditions.

Essentially, solutions are presented for four different conditions:

- All layers structurally unsound.
- Surface layer structurally unsound.
- All layers structurally sound.
- Futuristic systems.

Within each solution, several materials/systems are

presented and discussed. Each materials/system presented is recommended for a particular case and, as such, may not apply for other than the stated case.

Such an approach seems consistent with the reality that vastly differing situations will be encountered in different sections of the country.

The solutions presented are not meant to be inclusive of all possible solutions to the stated problem. As the research progressed, ideas and concepts were developed through systematic appraisals of the problem by various members of the research team working with the literature and by knowledgeable people contacted on this project. Many ideas, materials, and construction systems were discussed, and quite a few were discarded. For example, in the many materials or additives or modifications to existing materials reviewed, most dealt with improving strength which, in a full-depth repair, is not generally needed. The ones that dealt with rapid-strength gains were, in general, too fast for large-scale work but worked well for patches—some as large as 100 sq yd (80 m^2). However, with replacements on the order of 1,000 sq yd (800 m^2), an agency could ill afford the handwork usually associated with the rapid-hardening materials used in patches. Another example involved equipment that had performed well in certain applications but had never been restricted to a one-lane width.

Each process, material, equipment, or procedure was reviewed by the research team. The available literature was reviewed and people who used a particular system were interviewed. Where possible, an onsite evaluation, considering project objectives, was made. Most of these ideas are presented in the appendixes and, as might be expected, more were found that did not have application to this problem than were found that had application.

The solutions offered in Chapter Two were reviewed by the research team, and knowledgeable consultants were drawn from highway transportation departments, manufacturers, and highway contractors. The proposed solutions appear to be technically feasible and conceptually sound for the applicable conditions imposed.

Appraisal of the cost effectiveness of the proposed solutions is a difficult task. Obviously, under the constraints imposed by the problem to be solved, any solution will be costly to the agency paying the bill for the rehabilitation. On the other hand, reducing the out-of-service time for the rehabilitation of a major freeway carrying in excess of 100,000 vehicles per day will significantly reduce the delays, annoyance, automobile fuel consumption, accident potential, and thus cost, to the users of the facility. For each situation encountered, the following factors should be determined:

1. Estimated cost to the owner for various rehabilitation strategies.
2. Estimated extra fuel and delay costs to the users for various rehabilitation strategies.
3. Available funds for the rehabilitation.
4. Estimated costs to the users if the rehabilitation is postponed.
5. Estimated future costs for rehabilitation if postponed.
6. Estimated level of traffic service required (the public simply may not permit the facility to be closed down for any appreciable length of time).

These factors should be analyzed for their interactive effect and a solution generated that offers the greatest over-all cost and energy effectiveness to both the owner and the users—within the available funds for the rehabilitation. In Chapter Two, all the solutions presented, except the futuristic systems, contained materials/systems that were judged to be cost effective. The basis for this judgment was largely logic, stemming from the fact that currently existing materials were used with innovative, but plausible construction techniques; or new and innovative materials were used with currently existing construction techniques. Occasionally, both new materials and new construction technology were proposed. In these cases cost effectiveness cannot be assessed without field trials.

The cost effectiveness of the futuristic systems cannot be assessed at this time because of the large costs of developing such systems to the point where they can be successfully employed. Costs of one million dollars, or more, might be needed to develop any one of the systems. Complicating the cost problem is the reality that, in order to recover the developmental costs, the system would have to be transported throughout the country and made available to contractors bidding on specific jobs. The logistics and administration of such an undertaking would be difficult, to say the least. Therefore, the researchers are of the opinion that the futuristic systems proposed in Chapter Two will probably not be developed within the private sector of the economy; such development would have to come from public funds.

CHAPTER FOUR

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

To rapidly rehabilitate heavy-duty freeways in urban areas requires innovative technology in two areas—materials/systems to be used and construction procedures to be followed. Based on the results of this study the following conclusions are made:

1. Because of the vastly different situations existing throughout the United States, no single solution to the problem is likely to be developed.
2. Solutions involving the least amount of new technology are those in which an adjacent lane can be closed during the rehabilitation of a significant length of a freeway lane.
3. Because of the time constraint (48 hr), improvement of the subgrade, or natural soil, cannot be accomplished by known technology and must be left "as is." Thus, the rehabilitated pavement structure layers must be designed to prevent overstressing the unimproved subgrade.
4. The construction management techniques of precedence diagramming and analysis bar charting will indicate the critical aspects of any rehabilitation strategy and provide the information necessary to schedule the rehabilitation within the time constraint.
5. The capability exists for the removal of ¼-mi (400 m), single-lane sections of the heaviest of unsound pavements and sublayers in existence today and replacement with equal or better pavement structures in 48 hr. Lighter structures can be removed and replaced in even less time. Using new and innovative technology, the potential exists for reducing the 43-hr time period to 35 hr.
6. When only the surface layer is structurally unsound, solutions are available whereby a ¼-mi (400 m) long lane could be rehabilitated in 26 hr.
7. When all pavement layers are structurally sound (requiring only the restoration of riding quality and/or skid resistance), solutions are available whereby a ¼-mi long (400 m) lane could be rehabilitated in from 14 to 24 hr, depending on existing conditions.
8. Promising innovative materials/systems to accomplish rapid rehabilitation of significant portions of an urban freeway include:
   (a) Deep-life asphaltic concrete.
   (b) Rapid-hardening, high early-strength concrete.
   (c) Sulphur systems for pavements.
   (d) Precast portland cement concrete panels.
9. Futuristic systems are discussed which offer the potential for rehabilitation of significant sections of a freeway lane by closing only one lane of traffic throughout the rehabilitation period.
10. One possible futuristic system has the potential for keeping all lanes open during rehabilitation.
11. Development of futuristic systems to the state of readiness for practical application is likely to require the expenditure of large sums of money, and thus are not likely to be undertaken without significant public financial support.
12. A large number of special materials/systems techniques exist that offer potential for very rapid spot rehabilitation.

RECOMMENDATIONS

The solutions developed in this report call for close control of materials and close construction scheduling of the rehabilitation. To test the validity of the solutions, one or more solutions should be tried on a full-scale rehabilitation project in which a ¼-mi long (400 m) lane is selected, a rehabilitation strategy is designed to accomplish the rehabilitation within the specified time limit, and a construction technique is employed to accomplish the rehabilitation. Materials selection should be made in cooperation with the materials supplier and the constructor. Construction management techniques, which include precedence diagramming and analysis bar charting, should be employed to insure proper execution of the rehabilitation. A suggested research plan is shown in Figure 24 for the field demonstration of a rapid rehabilitation strategy.

Preconstruction Evaluation

Once the administration has made the decision that rehabilitation must be done on an urban freeway, certain procedures are necessary to insure that time and money are not wasted. Any motorist can determine with reasonable certainty whether a pavement is in need of repair. However, more definitive evaluations are required if resources are to be best utilized. Such an evaluation should include the following items:

1. The pavement design (both thickness and geometric) and the construction records should be examined to identify potential problem areas, such as sag curves and low areas, where drainage may be inadequate and base and subgrade problems may be anticipated.
2. Traffic histories should be examined; such histories may give probable cause for pavement failure and better predictive traffic to promote adequate strength design for the rehabilitation.
3. Maintenance records should be examined for areas that might affect removal of the existing structure such as overlays, base repairs, or extra-depth patches with dissimilar materials.
4. The actual physical condition of the existing pavement should be determined. A number of states are now keeping condition survey records which may supply much

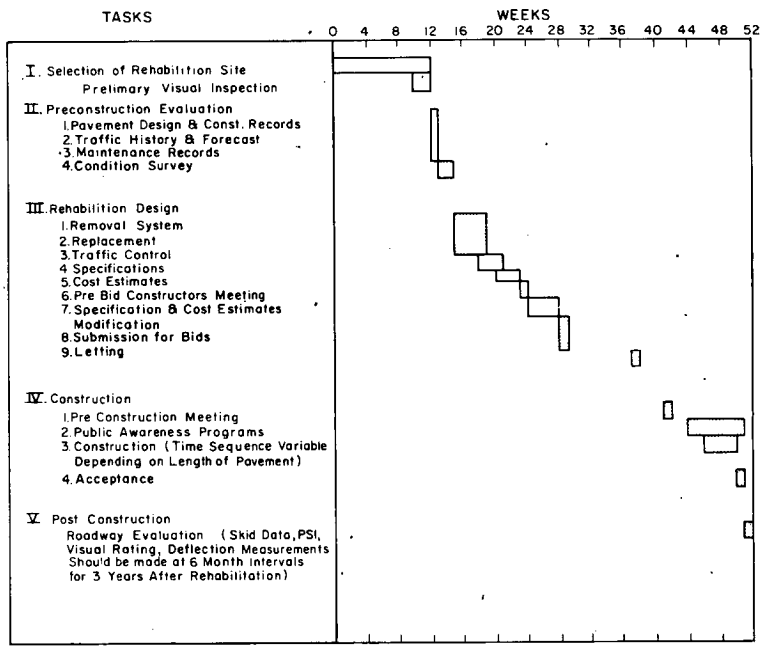

Figure 24. Suggested research plan for field demonstration of a rapid rehabilitation strategy.

of the needed information. Data on the pavements physical condition should be acquired both for the design of the rehabilitation and the forecasting of future rehabilitation needs. In some areas, cores will be required to determine the condition of the base and subgrade.

5. Underground utilities should be located and identified.

Design of Rehabilitation

After the existing pavement is thoroughly evaluated, a rehabilitation design can be formulated. First, it must be determined whether or not a significant portion of the pavement must be removed and replaced. For those cases where an overlay, spot patch, or a combination of spot patches followed by an overlay will not be satisfactory, major reconstruction should be considered.

For areas of poor drainage, consideration should be given to installation of a drainage system to improve conditions. Such systems can be installed alongside the pavement with minimal traffic interference. These systems should be installed before removal of the pavement structure.

In the areas where the subgrade is unstable and conditions favor stabilization by injection with lime or a proprietary chemical, consideration should be given to horizontal injection from the side of the pavement (see Appendix F). This is a recent construction innovation which can be accomplished with minimal traffic interference.

In those areas of poor drainage and/or stable subgrades where no in-place improvement can be made, consideration should be given to adding extra thicknesses of pavement to compensate for poor drainage conditions. Other alternatives for subgrade treatments are discussed in Appendix F.

Removal and replacement systems are described in detail in Chapter Two. The following are general suggestions that may have merit for certain conditions.

In consideration of a removal process, most cases will require that the surface layer or layers be removed and hauled from the site. If the subgrade is adequate, then consideration should be given to in-place treatment of the base courses. This is especially true if the subbase is an unstabilized granular base. Such bases could be loosened with a ripper and then mixed in place with a stabilizer in a pulver mixer. With a minimum of compaction, a stabilized base can be produced. If the subbase is an asphalt-stabilized base or an asphalt-concrete base, consideration should be given to reprocessing in place (Appendix C).

The replacement design, especially the selection of materials, will depend to a large extent on the thickness of the pavement to be replaced and the type of surface on adjacent lanes. If the adjacent lanes are considered adequate for the traffic, then a stress analysis may not be necessary inasmuch as the replacement will be at least equal and, in most cases, structurally superior to the existing pavement.

The actual removal and replacement process should not be completely designed until a specific contractor is selected, inasmuch as it is highly unlikely that many contractors will have the exact equipment array described in Chapter Two. However, this is not considered to be an insurmountable problem. Most systems could be reasonably selected by ranges of values and performance times.

Construction Control

As pointed out in *NCHRP Synthesis 25* (*12*), any construction in urban areas requires a sizeable effort in coordination and cooperation between several agencies. Public awareness of the project will tend to maximize acceptance of traffic delays. Coordination of operations with police can maximize traffic safety during construction. Strict adherence to predetermined schedules by both the agencies and the contractor will minimize delays and thus minimize contractor occupancy of the roadway.

In terms of quality control and acceptance procedures, this type of rapid rehabilitation construction lends itself to end-product specification and acceptance, which should be used whenever possible. The construction sequence should be closely controlled, monitored, and documented for evaluation.

Post Construction Evaluation

After construction is completed, certain data should be gathered periodically to evaluate the project. As a minimum, skid data, pavement serviceability, and a visual rating should be performed initially and at 6-month intervals for at least 3 years. Deflection data are desirable and should be obtained. Depending on the state agency's policies, the aforementioned data could be obtained as a part of a regular inventory of state highways.

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APPENDIX A

LITERATURE SEARCH

The total abstracts described in the literature search were reviewed and a number were eliminated from further consideration as not being germane to the project. A second review was made and salient articles were secured for reference in the final report, and additional articles were eliminated from consideration.

The remaining abstracts assembled by topic areas are given in Table A-2. Where possible, the abstracts within each topic area were arranged in alphabetical order by author. More than 500 abstracts were assembled in an annotated bibliography as a separate document from this report. The annotated bibliography is available from NCHRP staff.

TABLE A–1

KEY WORDS LIST

I. PAVEMENT	II. RECONDITIONING	
pavements	creep rupture strength	metal particle composites
highways	resilience	composite structures
asphalts	skid resistance	compressive strength
concretes	reconditioning	compression tests
portland cements	renovating	compressibility
concrete pavements	fracture strength	epoxy resins
freeways	surface finishing	epoxy coatings
roadways	reinforcement (structures)	crack propagation
roads	binders (materials)	reinforcing steels
aggregates	stresses	sulphur
aggregate seal coats	tensile strength	glass fibers
reinforced concrete	tensile impact strength	cement additives
cements	impact tests	admixtures
bituminous cements	impact strength	glass particle composites
bituminous concretes	stress relieving	weatherproofing
bituminous coatings	flexural strength	density measurement
flexible pavements	stress relaxation tests	fiber reinforcing
paving	durability	waterproofing
	fiber composites	structural reliability

TABLE A–2

TOPIC AREA LIST

General	Recycled Pavements
Pavement Repair Techniques	Pavement Surface Treatments
Precast Panel Construction	Portland Cement Concrete Pavements
Fiber Reinforced Concrete	Asphalt Cement Concrete Pavements
Polymer-Strengthened Concretes	Subbases
Sulphur Repair Techniques	Pavement Design
Rapid-Setting Cements	Pavement Evaluation
Structural Foams	

APPENDIX B

SITE VISITS

A number of state and federal agencies were visited and the objectives of this project were discussed. At each agency, rapid repair procedures and techniques in use by the agency were reviewed in light of how such techniques might be applied to this project. Where possible, field locations were visited and contractors were interviewed. An attempt was made to identify the causes of pavement failures at each agency, the solution used, and the agency's assessment of the potential of the method as a means of rapid rehabilitation. Agencies were visited in:

Boston, Mass.
Calif.
Tulsa, Okla.
Dallas, Tex.
Salt Lake City, Utah
St. Paul, Minn.
Olympia, Wash.
Chicago, Ill.
Naval Civil Engineering Laboratories, Port Hueneme, Calif.
Lansing, Mich.
Southeastern States

Summaries of these site visits are given in the following paragraphs.

RAPID REPAIR TECHNIQUES ON THE FITZGERALD EXPRESSWAY VIADUCT IN BOSTON, MASSACHUSETTS

Owner: The Department of Public Works
Office of Transportation and Construction
100 Nashua Street
Boston, Massachusetts 02114

Contractor: Builders Iron Works, Inc., and
J. L. Caputo Construction Co., Inc.
33 Park Street
Somerville, Massachusetts 02143

Project Description:

The project consisted of excavation of bituminous concrete patches and sections of disintegrated portland cement concrete, followed by replacement with new portland cement concrete patches on a six lane viaduct (bridge) approximately 0.55 miles long (2900 ft or 900 m) in the summer of 1974. 5500 sy (4600 m^2) of deteriorated concrete was estimated by the owner for removal, and 550 cy (460 m^2) of concrete patching was estimated to be placed. An overall view of the project is shown in Figure B1. Peak hourly traffic averaged 1540 in each lane with an average daily traffic count for the facility of 135,000. Views of the surface deterioration are shown in Figure B2 and Figure B3. The total contract bid was $825,725 which includes $140,000 for traffic control.

Rehabilitation Procedures:

The following unique procedures were utilized to expedite the

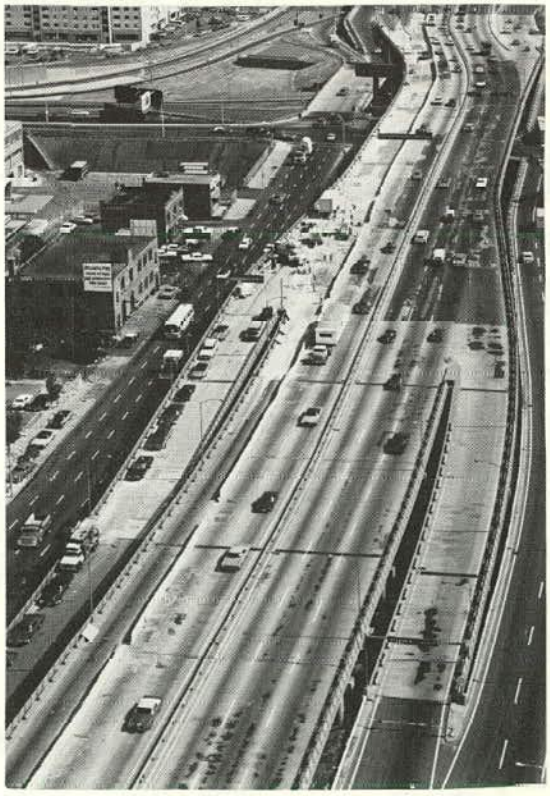

Figure B2. Boston Project, View of Deteriorated Bridge Deck.

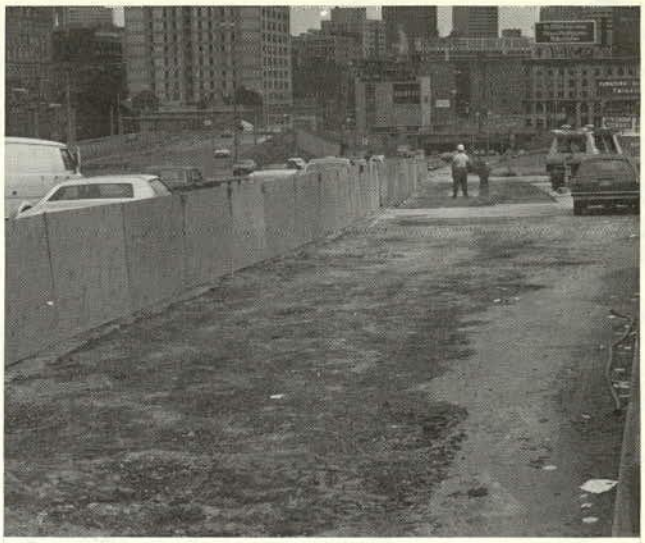

Figure B3. Boston Project, View of Deteriorated Bridge Deck.

rehabilitation program:

1. The contractor was required to work 24 hours per day, seven days per week.

2. The contractor could close only one lane at a time, and was required to have a 12 in. by 12 in. (300 mm by 300 mm) wooden beam wheel guard and a 4 ft by 8 ft (1.2 m by 2.4 m) plywood facing between his work and the traffic (Figure B4 and Figure B5).

3. Patching was accomplished by removing the unsound concrete by use of jack hammers to a level just below the top layer of reinforcement (Figure B6).

4. Work proceeded at a rapid pace with each 2900 ft long lane being closed, repqired, and reopened in approximately two weeks (depending upon the extent of patching required).

5. The contractor was required to have an extensive traffic control program including warning signs, cones, flashing arrows, flags, and police control. Advance publicity was programmed to inform the traveling public on what to expect as the project commenced. It required two days to set up and remove the traffic control system for each lane closure.

The work went very smoothly and rapidly. Mr. L. A. Defranzo, District Eight Structures Maintenance Engineer for the Massachusetts Department of Public Works reports:

> 1. The contract will be completed approximately four (4) days in advance of the completion date. An over-run of approximately twenty-eight (28) per-

Figure B4. Boston Project, View of Traffic Barrier.

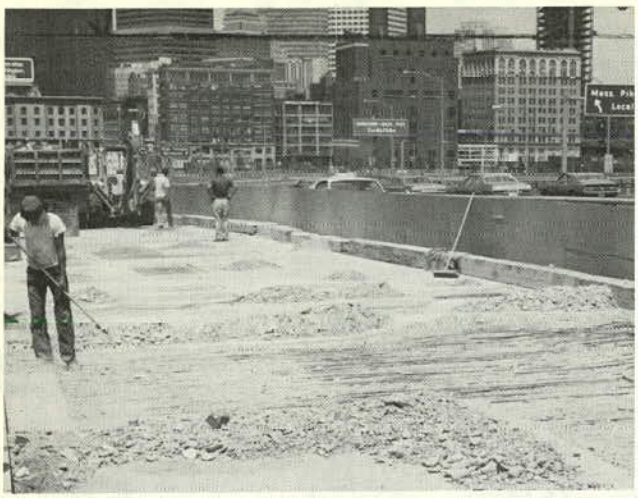

Figure B5. Boston Project, View of Traffic Barrier and Deck Removal Operations.

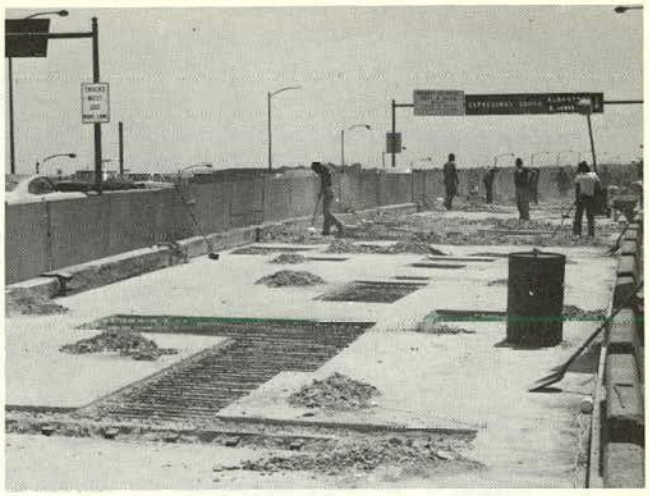

Figure B6. Boston Project, View of Deck Removal Operations.

Figure B7. Pavement Surface.

Figure B8. Cutler Repaver.

cent in the deck excavation and concrete delayed completion of the project well in advance of the completion date. This is accredited to the cooperation of the Contractor with regards to his work force remaining constant while work progressed.

2. Use of police details to control the flow of traffic on the viaduct and surface roads resulted in a minimum delay to the traveling public. The contractor benefited from the use of police details by allowing minimum delays in moving equipment on the viaduct structure.
3. On a project of this size the use of patching compounds would have been economically unsound. Any time saved by use of these compounds would have been minimal because of the length of the project.
4. The method of replacing those sections of deck found unsound, either by visual inspection or by sounding, leaves the question of how long the remainder of the deck will last. Taking the problem of deck repairs from a maintenance standpoint it is only a temporary measure to insure a rideable roadway for the traveling public.

The major problems with the job seemed to be the occasional delay of ready mixed concrete due to traffic tieups and weather. Otherwise the job went about as rapidly as possible under the conditions imposed by the owner. About the only procedures which could have been employed to speed the job any faster were:

1. Allow the contractor to remove the full thickness of the concrete in the deck at each deteriorated site rather than only 2 to 4 in. (50 to 100 mm). By so doing, larger jack hammers could have been used and thus it is estimated that approximately 15 percent less time would have been required. However, this would have been much more expensive.

2. Use a more rapid hardening concrete in the last few patches in each lane, as they had to keep the lane closed for 72 hrs beyond the last pour in order to provide sufficient cure time for the concrete. This could save an estimated 10 percent in the time each lane was closed. However, these more rapid hardening concretes would have to be shown to be sufficiently strong and durable for the intended use.

3. Combining 1 and 2 above, it would appear possible to remove and repair patches in one lane of a 2000 ft (600 m) long section of concrete highway or bridge in approximately 10 days, which would include 2 days of traffic control setup and removal and 8 days for fairly extensive repair work using readily accepted techniques and materials. Reducing the length of repair in each lane closure would reduce the lane closure time, but not the two days required to set up the traffic control system as it would be almost independent of the length of lane closure.

FREEWAY REPAIRS IN CALIFORNIA

Two of the most vivid examples of the problems confronting the highway engineer in rapidly rehabilitating major freeways are in the Los Angeles and San Francisco areas of California. Here traffic moves at volumes in the neighborhood of 200,000 vehicles per day past a given point, with peak volumes in excess of 15,000 vehicles per hour.

A detailed examination of the expressways in these two areas revealed that the major rehabilitation efforts seem to be concerned with portland cement concrete (PCC). This is true first, because virtually all of the major, high-volume facilities are PCC and secondly, because flexible pavements have (to date) usually been rehabilitated by relatively conventional means. The major pavement problems requiring rehabilitation are:

1) Joints at structures
2) Joint filler materials
3) Edge curling of PCC slabs and slab rocking
4) Salt, chain and studded tire damage to PCC pavements in ice and snow areas
5) Settlement at bridge ends
6) Structural cracking of PCC slabs in the outer lanes which carry the majority of the truck traffic. It has been observed that some of the cement treated base courses (CTBC) with relatively low cement contents (3 to 5 percent) lose their slab action after a number of years and degenerate into a "granular" mass of individual larger particles and concretions of smaller

particles. It should be noted that there is no steel in these PCC pavements.

7) Vertical faulting of the unreinforced pavement slabs. These pavements were constructed over unreinforced CTB, and the magnitude of vertical faulting exhibited has made necessary an extensive grinding program which is followed, ultimately, by complete slab replacement.

It should be emphasized that only a small portion of the expressway system is in need of repair, with freeway failures no more than one or two lanes wide and generally hundreds of feet (or less) in length. But, these repairs are extremely difficult to execute due to the traffic. How can rehabilitation be accomplished without serious disruption to traffic? The answers lie in speed and closure of as few lanes as possible for any given period of time.

Compounding the problem of rapid rehabilitation are several factors, including:

1) Some years before the creation of OSHA, a series of traffic induced fatalities among the Maintenance workers of the Los Angeles District produced a concerted demand from the unions for better protection of the workers. The California Department of Industrial Safety required the District to establish a Lane Closure Policy whereby "When work is performed on a freeway or ramp, any traffic lane of the freeway of the ramp whose nearest edge is within 6 ft (2 m) of the work shall be closed and a traffic control system shall be installed."
2) The vast majority of detours required to permit freeway

rehabilitation are confined within the freeway R.O.W.

3) Rerouting freeway traffic at night demands coordination with utility and transportation companies, fire, police and county jurisdiction. Otherwise freeway traffic may unknowingly be routed over streets that are blocked by utility work.

4) Requirements enforced by EPA, restrictions resulting from the Porter-Cologne Water Quality Control Act, Fish and Game Commission Controls and the County Air Control Board all combine to make the maintenance and reconstruction of freeways increasingly difficult. Construction operations particularly affected include: pavement grooving, night construction from the standpoint of noise, production of hot mix - emissions, placement of hot mix - blue smoke.

California has incorporated a number of innovative techniques in order to cope with rehabilitation problems.

1) Freeway reconstruction plans include traffic control procedures and sequencing of construction operations. The contractor is controlled with regard to when he can work, how many hours he can occupy, and what type of traffic control devices and what method traffic control he must employ.

2) All lane closures are logged with the Communications Division and information on closure locations and durations is transmitted to all interests affected.

3) The resident engineer on a freeway reconstruction project notifies all local interests affected by a construction closure. Signs are posted in advance and the public is also

alerted through the radio, T.V. and newspaper facilities.

4) Shoulders and flush-paved medians are sometimes converted to extra passenger car lanes by proper striping and signing and shifting of the center median guard rail

5) For the PCC pavement over 4 in. (100 mm) CTBC, the reconstruction is usually as a 12 in. (300 mm) PCC pavement to achieve greater speed of construction.

6) On the highest traffic volume freeways, emergency repairs are made to PCC pavements at night by removing the PCC pavements and underlying CTBC to the subgrade. The repair is made by filling the excavated area with a 564 to 658 lbs per cu yd (192 to 213 kg/m ^{3}) Type II cement concrete to which 2 percent $CaCl_2$ is added on-site. The patch is opened to traffic at 6 A.M. the following morning at an age of 8 hours.

7) When the outer truck lanes of a rural PCC freeway become too rough, truck traffic moves toward the inner lanes. To forestall this unwanted shift of heavy axle loads into the passenger car lanes, truck lanes have been rehabilitated by a combination of:

   a) Grinding of step-faulted slabs

   b) Mudjacking individual slabs

   c) Overlaying with 4 to 6 in. (100 to 150 mm) of hot mix asphaltic concrete where vertical clearances permit.

ASPHALT PAVEMENT REHABILITATION, TULSA, OKLAHOMA

Project Description: A major arterial street in Tulsa, Oklahoma, was cracked and had lost riding quality (Figure B7).

Rehabilitation Procedure: May, 1974. A Cutler Repaver was used to heat and scarify a surface layer approximately 3/4 in. (20 mm) in thickness. Additional hot-mix asphaltic concrete was added to the removed material, mixed with an auger-type mixer and laid down in a single pass. The layer was then compacted in a conventional manner (Figures B8 through B12).

The machine was able to progress at an average speed of about 30 fpm (9 m/min) on this project. The cost of the Repaver was about 50¢/sy (35¢/m^2) and the additional material added about 50¢/sy (35¢/m^2) for a total cost of about $1/sy (85¢/m^2) in place (1974 prices).

Analysis: The Cutler Repaver performed well and the rehabilitation was satisfactory with a minimum of traffic delay.

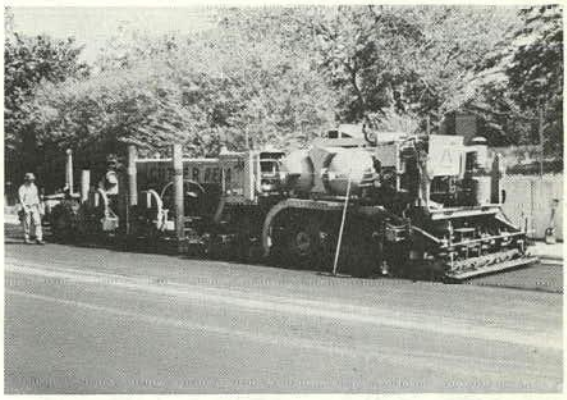

Figure B9. Heater Portion.

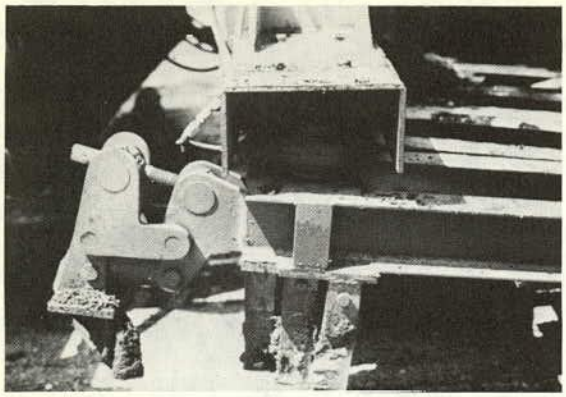

Figure B10. Scarifier Teeth.

Figure B11. Additional Hot-Mix.

Figure B12. Compactors.

PORTLAND CEMENT CONCRETE PAVEMENT REPAIRS - DALLAS, TEXAS, AREA

Project Description: Full depth patching of Portland Cement Concrete (PCC) pavement. Generally the sections are less than 20 ft (6 m) in a single lane.

Rehabilitation Procedure: Saw cut partial depth around area to be replaced. Unsound material was broken up with jack hammers and removed with back hoe or gradall. SDHPT District 18 (Dallas Area) has used among others

1. Custon Crete - (previously known as Fast Fix by Western Company).

2. Duracal.

With both materials they use an epoxy around the patch to promote bonding. Generally the cement content is 7 to 8 sk/cy (5 to 6 sk/m^3) and costs about $52 per cy ($40/m^3). They view the repair as temporary and expect it to last 5 years or less. The patches can be accomplished and opened to traffic in less than six hours.

Analysis: These products seem to lack the durability for extensive long term rehabilitation.

PAVEMENT REPAIRS - SALT LAKE CITY, UTAH, AREA

Projects Description:

1. Pot hole repairs, primarily on bridge decks. Distress due in part to the use of deicing salts.

2. Overlay to restore riding quality.

3. Overlay on bridge.

Rehabilitation Procedure:

1. The first step was to clean and dry each pot hole and remove

all unsound concrete. Patches were made using several epoxy admixtures (see photograph). After only a short time cracks usually developed at the edge of the patch indicating bonding problems. The engineer inspecting the jobs stated that the additional cost of the new materials did not seem justified on the basis of performance to date.

2. An asphalt concrete overlay using a product called "Crumbme" rubber (a ground latex furnished by Atlas Rubber Co., Los Angeles, California) was placed to restore riding quality (see photograph). The overlay required the use of a special asphalt. The combination of asphalt and rubber was intended to reduce reflection cracking. The overlay was relatively new (less than 6 months old) and exhibited no cracking.

3. A latex modified PC concrete overlay 1 in. thick (25 mm) was placed on a bridge deck to seal it and restore riding quality. The overlay cost approximately $19 per sy ($16/m^2) and developed crazing cracks in less than six months.

Analysis: None of these materials/systems related directly to the problem of sizable full-depth replacements.

STEEL AND GLASS-FIBER REINFORCED PCC OVERLAY ON

TH51 IN RAMSEY COUNTY, MINNESOTA

Owner: State of Minnesota
Department of Highways
St. Paul, Minnesota 55155

Contractor: Arcon Construction Company, Inc.

Project description: Experimental sections of plain, steel-fiber reinforced and glass fiber reinforced, each three in. (76 mm) thick were placed on the four lane facility. The work was completed in June, 1974. Two traffic lanes were closed during construction.

Rehabilitation Procedures: The existing surfaces were first cleaned either with a power broom or sandblasting, then flushed with water. Epoxy or cement slurry was used in those areas requiring special bonding. The overlays were batched, placed, consolidated and finished using normal procedures.

The plain concrete was 649 lb of cement per cy (226 kg/m^3) batch using 3/4 in. (19 mm) maximum sized aggregates. Both fiber mixes were of 800 lb of cement per cy (277 kg/m^3) with 3/4 in. (19 mm) maximum sized aggregates, and contained 1.2 percent fibers by volume. Higher than normal water to cement ratios had to be used with the glass fibers in order to maintain workability, and the glass-fiber strength was 15 to 40 percent lower than the plain or steel fiber strength at all ages.

Analysis of Project: The steel fiber concrete worked essentially the same as other reported steel fiber overlays (Iowa, Michigan, Illinois). A. E. Pulk, Resident Engineer with the Minnesota Department of Highways and in charge of the Project, stated the biggest problem with the glass fibers seemed to be that occasionally the fibers would ball up and create a pop-out in the concrete (Figures B13, B14, B15). Part of the glass-fiber section was constructed during a rain. Mr. Pulk thought that this was the primary cause for the poor performance of the surface. He stated that this would not be a good field test for the glass-fiber reinforced concrete.

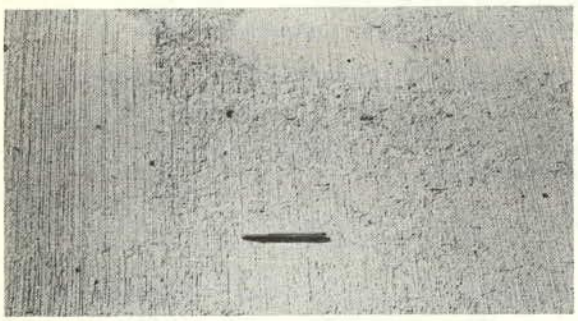

Figure B14. Surface Deterioration.

Figure B15. Pop Out Due to Balled Up Fibers.

Figure B13. View of Fiber Glass Concrete Overlay.

PAVEMENT REPAIRS - OLYMPIA, WASHINGTON, AREA

Project Description:

1. Portland Cement Concrete (PCC)
2. Asphalt Cement Concrete (ACC)

Rehabilitation Procedure:

1. Remove unsound concrete and replace with one of the following materials/systems.

   a. Quick Set-N. This is a shrinkage compensating cement. It has a high purchase cost but performance has been reasonable. This agency would not use it for large repairs.

   b. High Alumina Cement. Has been used and opened to traffic in 24 hrs. This agency does not see much advantage of this cement over Type III cement with an accelerator.

2. a. A Cuttler Repaver was used about 1972 to resurface an experimental section of concrete pavement. Performance has been satisfactory though the process has not been used on other projects.

   b. Petromat has been used to prevent reflection cracking in an asphalt concrete resurfacing project. Performance to date has been satisfactory.

Analysis:

The above materials are evaluated in Appendix C of this report. Washington has tried the technique of closing a major structure (bridge) over a week end for resurfacing. By using the news media extensively to secure public cooperation the project was deemed successful.

REPAIR OF CONTINUOUSLY REINFORCED CONCRETE

PAVEMENT ON THE DAN RYAN EXPRESSWAY, CHICAGO, ILLINOIS

Owner: Illinois Division of Highways
Chicago, Illinois
Thomas T. Morimoto, Project Group Engineer

Contractor: Brighton-Krug Construction Company
3533 Archer Avenue
Chicago, Illinois 60609

Project I Description:

The approximately 8-mile (13 km) project consisted of removing 200,000 sq yds (167,000 m^2) of 8 in. (200 mm) (CRCP) continuously reinforced concrete pavement on 6 in. (150 mm) of granular base and replacing them with 10 in (250 mm) of CRCP on 4 in. (100 mm) of asphalt stabilized subbase. The drainage system was renovated and new underdrains installed in critical areas. The ADT was 240,000 VPD on the 8 to 10 lane expressway. The total contract was for $5 million with a $5,000 per day bonus - penalty clause. The project was completed 31 days early.

Three factors were identified as possible causes for the early failure of the pavement.

1. Design traffic estimates were about 35 percent low and all trucks were confined to the two outside lanes.

2. A smooth bar, fabric-type, reinforcement was used which was thought to be inappropriate in terms of bond development.

3. The roadbed was constructed below the original water table

and the drainage system which was installed clogged and failed to provide adequate drainage. When the subbase and subgrade became saturated their strength was inadequate to carry the repeated loads.

Rehabilitation Procedure:

Work was scheduled for two 10 hr shifts per day on a continuous basis including Saturdays, Sundays, and Holidays. Traffic controls closed three traffic lanes in the right lane section for the full 8-mile (13 km) length during rehabilitation. Barricades and rubber cones were used as traffic control.

The pavement was removed with a D-9 Caterpillar tractor/dozer. The reinforcement was corroded and weakened and seldom required cutting with a torch. The pavement was replaced with standard slip form equipment.

An extensive public awareness campaign was used to solicit public cooperation during construction.

Analysis of the Project:

The use of the critical path method (CPM) by the highway engineers and the contractors coupled with the public cooperation allowed the 108 day contract to be completed 31 days early, resulting in a $155,000 bonus for the contractor. The project was returned to operation in September, 1971, and is providing excellent service.

Mr. Alan Kus, Vice President of the Brighton-Krug Construction Company, stated that, had the rehabilitation been restricted to one lane they would have used a Gradall for removal and he estimated about 50 to 67 percent reduction in productivity for that phase of the project. An additional 20 percent reduction in productivity would be experienced if the steel had to be cut with a torch.

Project II Description:

A second project was under construction in 1974. In this portion of the Expressway only short segments of the pavement were sufficiently distressed to require removal and replacement. Most of these were reported to be due to failure of the subbase in areas of poor drainage.

Rehabilitation Procedures:

Figures B16 through B21 illustrate the sequence of rehabilitation. The unsound concrete is identified and a partial depth saw cut is made. The pavement was then broken with the tractor mounted jack hammer and removed with a back hoe or the Gradall shown in the picture. The subbase was replaced with selected granular material and 7 sk PCC from a ready mix plant was used to replace the pavement surface. Lane closures were generally for a minimum of 10 days. In some areas only one lane was closed, but intermittent use of the adjacent lane facilitated removal and replacement.

Analysis of the Project:

The methods of removal seem applicable to the larger problem of continuous removal. A major effort in this project is the improvement of drainage which should prolong the service life of the pavement.

Figure B16. Saw Cut Partial Depth.

Figure B17. Tractor-Mounted Jack Hammer.

Figure B18. Grade All.

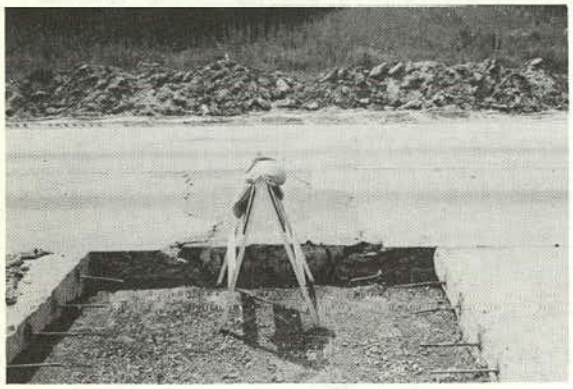

Figure B19. Subbase Replaced.

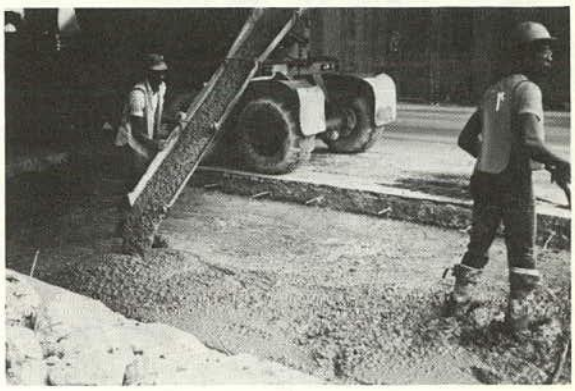

Figure B20. Ready-Mix Concrete.

Figure B21. Traffic.

NAVAL CIVIL ENGINEERING LABORATORIES (NCEL)

Projects:

1. Use of rigid foam systems for temporary or expedient runways for light aircraft.

2. Polymer strengthened concrete.

Procedure: (see Appendix C for more detailed information)

1. The NCEL has developed a pavement system using fiber glass filled with epoxy as a skin for a layer of 20 pcf (320 kg/m^3) polyurethane foam. They report difficulty in generating large quantities of the 20 pcf (320 kg/m^3) foam, but that lower densities of 12 pcf (190 kg/m^3) and less can be produced readily. They have used the system in several experimental tests on poor subgrades and as an expedient work table. The system is expensive but it will work.

2. The polymer strengthened concrete tests included an extensive list of polymers with various curing agents. They were able to develop some compressive strengths of 10,000 psi (69 MPa) in one day. However, most required the use of steam heat for 16 hours. Those tests performed without steam or heat offered little advantage over control specimens without the polymers. At best a 20 percent increase in strength was observed and in most cases studied, the air cured polymer concretes had lower strengths than control specimens.

Analysis:

The rigid foams offers potential as a subbase material.

PORTLAND CEMENT CONCRETE PAVEMENT REPAIRS, MICHIGAN

A conference was held in the office of the Research Engineer, Michigan Department of State Highways. He and the Physical Research Engineer discussed maintenance problems on Michigan expressways and presented a slide lecture on precast slab repairs conducted under their direction. Typical sections of CRC and PC pavements constructed in Michigan as well as specifications for precast slabs and slides and script of precast slab repairs have been made available to this research team. (See Appendix C.)

SITE VISITS TO THE SOUTHEASTERN STATES

Fibrous Concrete Waterways Experiment Station, Vicksburg, Mississippi:

A conference was held with the Chief, Materials Properties Branch, Concrete Laboratory, Waterways Experiment Station, Vicksburg, Mississippi. The subject of this conference was fibrous concrete and its suitability for the repair of distressed expressways. The work conducted on fibrous concrete at the Waterways Experiment Station was discussed. The research team was also provided a technical paper dealing with the development of fiber-reinforced concrete in North America. (See Appendix C.)

MUD JACKING OF PORTLAND CEMENT CONCRETE PAVEMENTS, GEORGIA

A conference was held in Atlanta, Georgia, with the Maintenance Engineer, Department of Transportation, State of Georgia. Information was provided on the mud-jacking program in Georgia as well as specifications and typical roadway sections employed in Atlanta expressways and Georgia Interstate highways. Time was spent in touring expressways in and around Atlanta under the guidance of the Executive Secretary, Georgia Asphalt Pavement Association, Inc. and timelapse films and color slides were taken of these expressways and of the bypass loops around Atlanta.

PORTLAND CEMENT CONCRETE PAVEMENT REPAIRS, FLORIDA

Conferences were held in Florida with the Tampa Maintenance District and with the Orlando Maintenance office. The subject of these conferences was the maintenance of IH 4 from Tampa to its intersection with IH 95 near Daytona Beach. Typical sections, mud-jacking information and a technical paper on the use of precast slabs for rigid pavement repair in the Tampa area were provided. A tour of IH 4 from Tampa to Lakeland, Florida was made during which several precast slab repairs were inspected and photographed. IH 4 was also inspected in the Orlando area as was the new Orlando toll road facility.

A conference was arranged with the Director of Planning and Development, Hillsborough County Aviation Authority, Tampa International Airport, and his assistant. Two fibrous concrete overlays on a taxiway at the Tampa airport were discussed and inspected.

The expressways and bridges of the Jacksonville, Florida area were inspected with the aid of the Maintenance Engineer of that district. Typical sections and valued information concerning the crack pouring schedule in the Jacksonville area were provided.

Types of Pavement Distress Observed

Pavement distress observed on high volume expressways and Interstate highways inspected to date varied to some degree with location

throughout the United States. Soil type, weather, traffic count and percentage of trucks were also factors which contributed in large measure to the total damage suffered by any given facility. Certain modes of incipient failure were present in all expressways examined thus far. Other types of failure were more localized in nature. With the exception of vertical faulting of rigid pavement slabs, failures were generally 50 ft (15 m) or less in length. An occasional distressed area, however, extended a hundred ft (30 m) or more. Failure modes identified at this point in the research program include but are not limited to the following:

1) Settlement and general evidence of subgrade instability in high fills, particularly at bridge ends and at other large structures - widespread.

2) Vertical faulting of rigid pavement slabs - widespread.

3) Edge pumping of rigid pavement slabs with associated slab cracking and joint spalling - localized.

4) Disintegration from de-icing salts at the joints and contraction cracks of unreinforced and continuously reinforced concrete pavements - localized.

5) Subgrade and shoulder subsidence leading to settlement and structural failure of both rigid and flexible pavements - localized.

6) Cracking and associated raveling in flexible pavements - localized and limited.

APPENDIX C

ASPHALTIC CONCRETE PAVEMENTS

The design and construction of bituminous base and surface courses is well documented (C1 - C15). During the past two years, significant changes have taken place in the industry (C16). New mixing equipment has been developed, new mix designs introduced, environmental and health standards enforced, and costs of materials, labor and equipment have increased. Additionally, new design procedures incorporating fatigue concepts have been introduced (C17). Even with all the changes asphaltic concrete remains one of the most economical paving materials.

Pavements can be placed in thick lifts to increase speed of placement. Single lifts in excess of 18 inches (460 mm) thick have been successfully placed and still achieved the desired density (C1). However, caution must be exercised when using thick lifts to allow sufficient curing time for heat to dissipate prior to load application. To treat this question, a heat-transfer model developed by Dempsey (C18) was used to produce Figure C1. The following assumptions were made to predict the time for a certain lift thickness to cool to 175°F (79°C) for any point in the layer.

1. Air temperature at a constant 90°F (32°C).
2. A 10 in. (250 mm) thick base at a constant 80°F (27°C).
3. A 130 in. (3300 mm) thick subbase at a constant 75°F (24°C).
4. Zero cloud cover

5. Zero wind velocity.
6. Solar radiation maximum.

Granted that these are extreme values seldom in combination in the field. However, it does demonstrate that for a 12 in. (300 mm) lift thickness placed at 220°F (104°C), as much as four hours may be required before other operations can begin. The New York State Thruway Authority has used full depth asphalt concrete patches for rigid pavements, and they report that some patches require as much as 24 hours for heat to escape (C19). Charles Foster (Director, Engineering and Research, National Asphalt Paving Association) states that this cure time can be reduced by balancing the lift thickness with the desired reduction in temperature, providing a finished product that can be immediately opened to traffic (C20). Examples of early traffic are given in references C1 and C18. In one recent application up to 625 cy (480 m^3) of asphalt concrete pavement were removed and replaced in one 9-hr shift (C20). This is the equivalent of 1400 lineal ft (425 m) of a 12 ft (3.7 m) wide by 12 in. (300 mm) deep pavement.

The asphaltic concretes have elastic moduli ranging from 40 ksi (275 MPa) up to 4000 psi (28 MPa) with 300 ksi (2,000 MPa) to 450 ksi (3,000 MPa) considered to be the general range. Santucci (reporting on the work Monismith, McLean, Epps, van Dijk and others), states that if tensile strains are less than 100 μ in/in, then a fatigue life, for 18 kip (8,000 kg) equivalent axle load in excess of 10^7 cycles, can be expected (C17, C21). Also, he reports that if the vertical subgrade strain is held below 250 μ in/in, a similar life can be expected without overstressing the subgrade. These design criteria

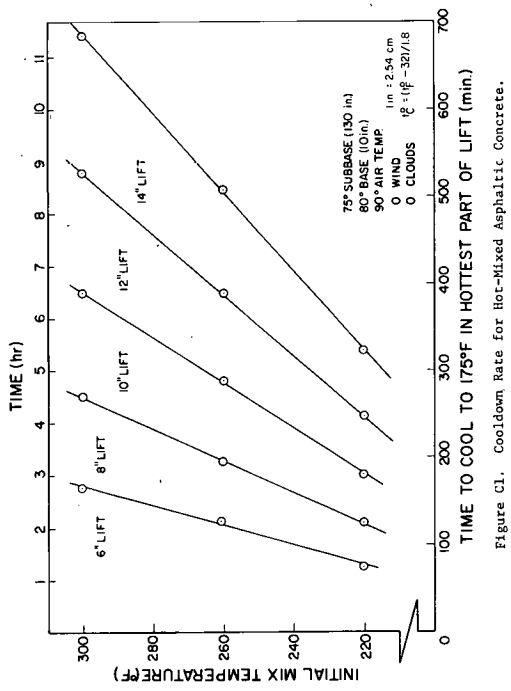

Figure C1. Cooldown Rate for Hot-Mixed Asphaltic Concrete.

lend themselves to computer analyses (C3). Some of these computer programs contain optimization techniques which include user delay costs as an input (C3).

In some circumstances, the existing material can be reprocessed to form base courses (C22, C23) (see Recycling Existing Pavements). Also, a recent study indicates certain so-called lower quality or marginal aggregates can be economically utilized in bituminous treated base courses (C15). This could reduce replacement costs under certain circumstances.

Care must be exercised when asphaltic concrete and portland cement concrete are interfaced in the pavement surface. For example, replacing a PCC slab with an HMAC surface or vice versa. Due to the dissimilarities in the physical properties of the two materials, there seems almost certain to be a "bump" at the interface of the two materials. Good construction practices (strike-off, consolidation, shrinkage compensation, etc.) must be used to minimize the "bump".

An asphaltic concrete pavement can be designed to satisfy most constraints. Its recommended use will be a function of the location and condition of the pavement to be rehabilitated.

RAPID HARDENING, HIGH EARLY STRENGTH CONCRETES FOR PAVEMENTS

Hydracal, a product of the U. S. Gypsum Company, was introduced in 1931 (C24). Interest has continued to be high in the development of cements which

harden very rapidly. The military, with need to rapidly repair bomb damaged runways, have investigated a number of processes. Notable among their works are those of Griffin (C25), Mosseir (C26), Lorman (C27), Setser (C28) and Pruitt (C29).

Developmental work in other countries has produced some important and successful results. In particular, the work in Japan (C30) and England (C31) demonstrate some of the successful work with rapid hardening cements.

Rapid hardening, high early strength, concretes have been made by (a) extensive grinding of portland cement to finenesses of up to 8000 sq. cm. per gm. (C32, C33), (b) using alumina cement - either by itself or with portland cement (C30, C34, C35), (c) using additives (accelerators) with portland cement (C36) and (d) using chemical compounds specifically formulated to produce rapid hardening concrete.

In recent years, a number of special compounds have been developed and introduced. These cements have been evaluated to some extent and have been successfully used in a number of cases. A partial listing of these cements is given in Table C1 together with reference where results have been published concerning their use.

A recent work by Pike and Baker (C38) summarizes several of these special cements and evaluates each of them in terms of their suitability as a concrete patching material. Two of these cements (the regulated set cement and the high strength, quick setting concrete) show promise in terms of strength gain, resistance to freeze thaw, and low chloride penetration. Since the time this report (C38) was being

Table C1. Rapid Hardening, High Early Strength, Cements or Concretes

Brand Name	Manufacturer	Related References
High Strength, Quick Setting Concrete	Republic Steel Corp. P. O. Box 6778 Cleveland, Ohio	C37, C38
Regulated Set Cement (RSC)	Huron Cement Co. Southfield, Illinois	C39, C40, C38
VHE Cement	U. S. Gypsum Co. Chicago, Illinois	C41
Duracal CEment	U. S. Gypsum Co. Chicago, Illinois	C38
Fast Fix C-1 Fast Fix C-2	Custon-Crete Inc. 2624 Joe Field Road Dallas, Texas 75229	C42, C43, C44

developed, U. S. Gypsum has introduced what they term an improved product over their Duracal. They call the product VHE cement, and have produced the cement in pilot quantities. In one report, (C41), the product was shown to have promise. Thus, the VHE cement, along with the regulated set cement, was selected for further evaluation in this study.

Laboratory batches of concrete were made up in the laboratory, and tested for strength. In addition, one performance test - the freeze thaw test in accordance with ASTM C666-Part A - was performed. The mix designs and results are given in Table 2.

From the results, it can be seen that both cements provide high strength concrete in a very short time (4 hours). These strengths compare with the High Strength, Quick Setting Concrete (HSQSC) manufactured by Republic Steel (C38). With sufficient quantities of cement, all cements appear to be quite durable (in terms of freeze-thaw resistance, Table 2). The (HSQSC) has also been shown to be durable (C37, C38, C40), but all three of these cements set very fast (Table 2) (C38) and appear to be very temperature sensitive. This would seem to pose little problem in the use of these products for small patches, but when major sections of freeway (up to 1500 ft, (460 m) in length) are to be repaired (see criteria), the authors are of the opinion that none of these cements could be used with conventional paving equipment because of their short setting times. Two possible solutions exist. One would be to use unconventional equipment which could handle, mix, place and consolidate the concrete within the short time

Table C2. Rapid Hardening, High Early Strength, Concrete Mix Designs

	VHE 1-1	VHE 1-2	VHE 1-3	RSC 6	RSC 6R	3073 4 (Control)
Cement Type	VHE	VHE	VHE	Reg. Set	Reg. Set	Type III
Cement (lbs)	6.6	20.8	20.8	46.8	46.8	41.7
Water (lbs)	2.8	8.9	10.7	21.8	25.3	19.0
Fine Aggr. (lbs)	14.7	57.3	57.3	87.1	87.1	83.6
Coarse Aggr. (lbs)	19.4	73.3	73.3	166.9	166.9	148.3
Admix. Type[a]	None	None	Citric Acid	None	None	None
Admix. (CC)	--	--		--		
Air Type	Sika	Sika	Sika	Darax	Darax	Darax
Air (CC)	0.5	2	2	25	25	22
Cement Factor (lbs/cy)	658	564	564	589	589	563
Slump (in.)	2-3	4	1-2	2	5	2

(Continued)

Table C2. (Continued)

	VHE 1-1	VHE 1-2	VHE 1-3	RSC 6	RSC 6R	3073 4 (Control)
Mix Temp. (°F)	72	67	73	67	69	74
Time of Set[b]						
Initial (min)	20	32	24	22	23	--
Final (min)	<28	36	33	26	25	--
Flexural Strength of 3x3x16 in. Beams						
Age of 4 hrs (psi)	--	533	--	148	128	30
Age of 1 day (psi)	--	592	--	325	224	440
Compressive Strength of 2x4 in. cyl						
Age of 2 hrs (psi)	3560	--	--	--	--	--
Age of 4 hrs (psi)	4750	3650	3260	800	800	--
Age of 1 day (psi)	--	4290	3330	1300	870	1560

(Continued)

Table C2. (Continued)

	VHE 1-1	VHE 1-2	VHE 1-3	RSC 6	RSC 6R	3073 4 (Control)
Durability Factor[c]	.94	--	40	>100	>100	74

[a]Solution of 2 lbs citric acid crystals dissolved in 1 gal water.

[b]ASTM C403.

[c]ASTM C666 - Part A (Modified by commencing freeze-thaw testing at an age of 24 hrs).

1 lb = .454 kg

1 lb/cy = 6 kg/m^3

1 in. = 25.4 mm

$t^\circ_C = 9/5\ (t^\circ_F - 32)$

1 psi = 6.9 kPa

allowed before the concrete sets. One such piece of equipment which has been developed to the point that it is conventional (in the sense that it is available and proved) is the Concrete Mobile (Figure 9). This system carries, in separate bins, all the unmixed components of the concrete to the job site in a single truck. Then, utilizing a mixer which is an integral part of the truck, the components are mixed and conveyed by chute to the placement area. This system has placed the regulated set cement concrete successfully (C45). An ASTM Standard Specification (ASTM C685) has been developed covering this operation.

The other solution would be to retard the initial set of these cements. The manufacturer recommended retardation for both the regulated set and the VHE cements is citric acid. However, citric acid will retard the development of an entraining system, so its use must be closely controlled. From our laboratory results (Table C2), it appears that the use of citric acid did not significantly retard the initial and final sets of the VHE cement.

RIGID FOAMS

Rigid foams have been used in pavement systems for more than ten years. The early installations were made in an attempt to control frost action in pavements placed in Canada (C46). Polystyrene foam 2 in. (50 mm) and 3 1/4 in. (80 mm) thick were placed under a 12 in. (300 mm) granular base and a 4 in. (100 mm) bituminous surface. The foam worked well, incurring no noticeable structural damage during construction and for the first year in service (C46).

Similar tests were made in Michigan and Minnesota with confirming results (C47). In Iowa, a styrofoam layer was placed successfully under a 9 in. (230 mm) PCC pavement (C47). Again no structural damage to the foam was observed.

In 1967 the Naval Civil Engineering Laboratory demonstrated the feasibility of using polyurethane foam as a structural layer for rapidly constructed helicopter pads. Irwin, in 1970, suggested a method to rapidly construct a 50 ft (15 m) wide highway by first placing a 12 in (300 mm) thick layer of polyurethane foam, followed by a thin wearing course. Irwin stated that the system could be completed for use at a rate of 330 ft per hr (100 m per hour) (C48).

Polyurethane was used by the Waterways Experiment Station to demonstrate the feasibility of using foams as pavements on very low-strength soils (C49).

The rigid urethane foam can be mixed and placed on location with densities from 6 to 25 pcf (100 to 400 kg/m^3) (C50). The structural properties are very closely related to the density of the foam. For the 8 pcf (130 kg/m^3) foam, the compressive modulus is 4,800 psi (33,000 kPa) and the tensile strength is 230 psi (1.6 MPa) while the 20 pcf (320 kg/m^3) foam has a modulus of 30,000 psi (207 MPa) and a tensile strength of 630 psi (4.3 MPa). The material has a relatively low Poisson's ratio (.1 to .2) and a very low water absorption. The material can be mixed, placed and completely cured in less than 90 minutes.

PRECAST SLABS

Precast PCC slabs, as a method of rapid repair of highway pavements, have been used in Michigan (C51), California (C52) and Florida (C53). Michigan began in 1969 with an experimental precast slab repair system that, through 1973, has been used in the placement of more than 900 precast units (C51).

The typical Michigan procedure is as follows:

1. Traffic control is established.
2. A full-depth longitudinal saw cut is made (to prevent damage to adjacent lane).
3. Holes are drilled and lift pins installed in as large a slab as can be lifted to minimize damage to subbase.
4. The slab is then lifted out by a crane (if slab is broken up it is removed with a front loader or back hoe).
5. Final clean up is accomplished with hand tools.
6. Subbase is then leveled and new material added and compacted if required.
7. A one in. (25 mm) layer of high-slump, rich, sand-cement mortar is placed and leveled with a screed.
8. The precast slab is then lowered slowly into the hole and leveled by beam guides with adjacent slabs.
9. The joints are filled with mortar.
10. The pavement is then opened to traffic.

This whole procedure, except for the saw cuts which are usually performed separately, requires less than 1 1/2 hours for undoweled joints, and less than 2 1/2 hours for doweled joints. These slabs are

usually 15 to 20 ft (5 to 6 m) in length.

Michigan reports the cost in 1972-73 to vary from $39 to $75 per sq yd ($47 to $90 per m^2) in-place, as compared to more conventional cast-in-place repairs of $45 to $78 per sq yd ($32 to $54 per m^2). Using the Engineering News Record Cost Index, this projects to about $100 maximum per sq yd ($120 maximum per m^2) in December, 1974, for precast, and $87 maximum per sq yd ($104 per m^2) for cast-in-place.

The California procedure (C52) is similar with the following exceptions:

1. An Arrow Hydralhammer was used to break the concrete and it was removed by a rubber tired loader. Final clean up of the hole was accomplished with hand tools.

2. The precast slab was placed on a cement-rich mortar and held at grade by I beam guides for approximately one hour.

Total time required was about 5 1/2 hours and the cost in 1974 was about $80 per sq yd ($67 per m^2).

The Florida procedure (C53) had some other variations. Namely:

1. The saw cuts were made through the dowel bars and then the concrete was broken with a typical headache ball.

2. A plate compactor was then used to densify the subgrade.

3. A crane was used to position the precast slab.

4. The slab was then jacked to final elevation by pumping grout through preformed openings in the slab.

5. The joints were then sealed and the repaired section opened to traffic. Florida's typical precast slab is 12 ft (4 m) wide by 20 ft (6 m) long and weighs about 12 ton (11,000 kg). They have placed as many as four

precast units in one eight-hour work night at a 1972 cost of $35 per sq yd ($42 per m^2) (which projects to a December, 1975, price of about $47 per sq yd or $56 per m^2).

Florida used the "Manual of Uniform Traffic Control Devices for Streets and Highways" and reported that it required about 2 hours to set up signs and detour barricades.

RECYCLING EXISTING PAVEMENT MATERIALS

The reuse of road-bed materials for paving has gained increasing attention. Increased costs of new materials, availability of acceptable quality aggregates, and the consideration of the disposal of materials removed from the road-bed are cited as factors contributing to this increased attention (C22).

Many projects in recent years have reused existing road-bed materials. These range from stabilizing a previously unstabilized base course to reprocessing and replacing a surface course (C22). Much of the work has used asphalt as the binding agent. However, portland cement has been used in several projects including a recent project in the Redondo Beach Freeway near Los Angeles, California (C54). In this project, both asphaltic and portland cement concrete were crushed and processed together, and used as aggregates for a lean concrete (econocrete) base.

Most current recycling processes require that the material be broken in place, then hauled to a remote site for crushing, sizing, mixing, and finally returned to the location for conventional placing operations (C22, C54, C55). In some circumstances, in-place crushing and processing were accomplished (C56). However, in-place mixing

processes normally do not lend themselves to high quality surface courses whether the binder used is asphalt or portland cement. Hence, with in-place recycling, a quality surface course usually must be placed as an overlay in order to provide the kind of service usually required on high volume highways.

The contractors and agencies that have used recycled materials generally report satisfactory results. Though in many cases the long term effects have yet to be evaluated, the early results give no reason to suggest that recycled materials will not perform as well as new materials. Some research has indicated that for portland cement mixtures, some reduction in compressive strength may be expected (C55). However, adequate strengths can be easily obtained for most pavement designs.

The recycling of highway materials will continue to merit attention for economic considerations, if nothing else. Most recycling projects have reported reduced costs over new construction. This will be especially true if the cost of disposal of rubble is included in economic evaluations.

ULTRASONIC VIBRATION CONCRETE

A sonically assisted cement mixer was invented at Ohio State University (C57). The process uses a lead-zirconate-titanate piezoelectric crystal driven by electricity to produce high frequency sound waves which can be transmitted to various mechanical devices. One of these is an experimental auger type concrete mixer. The vibrations are transmitted to the housing of the mixer.

The researchers theorize that the vibrations reduce the internal

friction of the batch, thus promoting more complete mixing and coating of the particles. This type of mixing reduces the requirements for water and cement to produce a specified strength. The process reduces the required water content so that a very stiff concrete is produced which had to be extruded in final form. No attempts were made to modify the process to produce more conventional mixes with excess mortar for finishing processes. Likewise, no attempts were made to determine the effects of various cements or additives such as Type III, Reg Set, retarders, plasticizers, polymers or epoxies.

Ultrasonic vibration was used to produce a high density, high strength concrete using normal portland cement and normal aggregates (C57). Concretes with a compressive strength of 3300 psi (23 MPa) were produced from a mix using 330 lb of cement per cy (200 kg/m^3). This compares with 100 psi (.7 MPa) produced by control batches with the same cement content (C57).

The dry materials are introduced into an auger which mixes and moves the material to a wetting point and then to a discharge port. The housing is vibrated at both low and high frequencies. The research sonic mixer required 3000 volts of power to drive the vibration mechanism at 10,000 hertz.

In 1969 the Ohio State University Research Team made the decision that the feasibility of this concept had been demonstrated and that industry should provide the development work for practical application of the technique. To the writers' knowledge, no further effort has been made to develop the process. It would seem to offer some promise, provided the necessary developmental work were performed.

POLYMER CONCRETE

The types of concretes under this heading can be broadly categorized by the following:

1. Polymer-concrete (PC) which is a concrete with a polymer as the matrix or cementing agent. The polymeric material is polymerized after placement during curing.

2. Polymer portland cement concrete (PPCC) is a premixed material in which either a polymer or monomer is added to the fresh concrete mixture in either a powder or dispersed phase, subsequently cured, and, if needed, polymerized in place.

3. Polymer-impregnated concrete (PIC) which consists of a dry hardened portland cement concrete impregnated either partially or fully with a monomer which is then polymerized in situ.

Polymer-Concrete (PC)

A 1972 state-of-the-art publication (C58) lists the following advantages and disadvantages for resin concretes.

"a. The advantages are:

High strength (both compressive and tensile)
High static and dynamic resistance
High bonding strength
Moisture tolerance of bond
Impermeability
Durability
High early strength (adjustable time of set)
Impact resistance
Low shrinkage

Creep resistance
Cracking resistance
Adjustable flexibility
Adjustability of composition to suit particular needs
Resistance to wear, chemicals, abrasives, oils, salts, caustics, and frost penetration

b. The disadvantages are:

High cost of materials (resins)
High cost of handling (special equipment, etc.)
Short pot life (requires small batches)
Reduction in strength with temperature increase
Shrinkage in large pours
Difficult handling and placement (due to toxic effects and bonding characteristics)."

The authors found that epoxy resin concretes generally exhibit higher strength, better bonding, higher moisture tolerance and better chemical resistance than other resin concretes (C58). The polyester and ceramic resins cost less, can be cured at lower temperatures, possess similar strengths at low concentrations, but are less moisture tolerant and have lower adhesive and durability values. The silicone resins are similar to the polyester and ceramic resins except they seem to be more durable than the epoxies.

Another type of polymer-concrete uses portland cement as a mineral filler in the concrete system. No water is used in this concrete and the strengths are greater than other types of polymer concrete (C59). A typical mixture contains 11 wt percent polymer (50/50 methyl methalcrylate

and styreme) and 89 wt percent cement, sand and coarse aggregate with a proportioned ration of 1:2.14:2.72. It is mixed and placed with conventional concrete equipment. The best results were obtained with a cure time of 6 hours @ 70°C (158°F). The polymer-concrete developed compressive strengths in excess of 11,000 psi (76,000 kPa) and modulus of rupture strengths in excess of 3,500 psi (24,000 kPa).

Most of these systems were developed as rapid patches. Repair of the Major Deegan Expressway in New York City was performed in 1974 using polymer concrete (C60). A patch 3 x 10 ft (1 x 3m) x 15 in. (380 mm) deep was made. The removal of the deteriorated pavement was done during the day and the hole covered with steel plate to carry traffic during the night. On the day of placement the material was placed and opened to traffic in 3 1/2 hours.

There has been very little large-scale testing of these materials, hence, little is known of their characteristics under full-scale pavement conditions. Such properties as shrinkage or thermal cracking of large slabs, cure time for full depth slabs, curing under field conditions, and wear resistance to tire scrubbing action must be given consideration prior to use in major pavement facilities.

The major drawback for the polymer-concretes is cost. Estimates for materials alone range from $170 per cu yd ($130/m^3) to over $300 per cu yd ($230/m^3) (C61). These costs, plus the additional construction difficulties, would preclude the use of polymer-concrete in full-scale, full-depth pavements at this time. This does not imply that its use in short, partial-depth patches should not be examined as a viable alternative.

Polymer Portland Cement Concrete (PPCC)

In the early research with polymer portland cement concrete, the results were considered to be less than desirable (C62). This was especially true when compared to the results obtained with polymer-impregnated concrete. The monomers used as additives tended to coat the aggregates and thus inhibited bond development.

Research using latexes and epoxy resins were conducted by the Naval Civil Engineering Laboratory during 1973. The results were encouraging with compressive strengths in excess of 10,000 psi (69 MPa) developed in some cases (C63). However, those mixes showing the most favorable properties were those cured with steam or heat or a combination of these with ambient air. In all cases, those specimens cured in air alone exhibited less than a 20 percent increase over control (untreated specimens) and in many cases developed lower strengths.

Other research seems to confirm that if early strength is desired, then some mechanism other than moist curing or air dry curing must be employed (C64, C65).

An installation of a 1 in. (25 mm) thick latex modified portland cement concrete placed as a bridge deck overlay cost $19 per sq yd ($23/m^2) (C66). This would indicate a cost of $150 to $190 per sq yd ($180 to $230 per m^2) for dull depth pavement (10 in. or 25 mm thick).

The additional strength is not really required for most pavement designs, hence, the additional cost of materials and construction difficulties do not appear to favor its use in a full depth, rapid rehabilitation system at this time.

Polymer-Impregnated Concrete (PIC)

The idea in PIC is to fill the voids in a normal PC concrete with a monomer and then polymerize the monomer in-situ. This has been done with dramatic success such as 285 percent increases in compressive strengths and 250 percent increases in the modulus of rupture (C62). However, to obtain these properties requires that the specimen be oven dried, placed in a vacuum, submerged in the monomer and allowed to soak under positive pressure, and then drained while still heated (C67). In other words, the process would limit itself to precast units such as precast bridge or pavement panels (C68).

Partial impregnation does not require the vacuum-soak methods of full impregnation, but usually partial impregnation does not significantly increase the strength characteristics of the concrete (C69). Generally, the process does reduce absorption of moisture through the surface and improves freeze-thaw durability of the surface (C69, C70). This surface impregnation has been used on a new bridge deck in the Denver area (C68). It required several days of special treatment.

Since the properties that were improved most by partial impregnation with polymer (moisture absorption and freeze-thaw durability) are not usually considered the most serious problems with pavement slabs, its use in rapid repair of urban freeways would seem to be of limited value.

FIBER REINFORCED CONCRETE FOR PAVEMENTS

General. Fiber reinforced concrete, defined as "concrete made of hydraulic cements containing fine or fine and coarse aggregate and discontinuous

discrete fibers" (C71), have been the subject of extensive research in recent years. The state-of-the-art has been clearly and concisely summarized by ACI Committee 544, with 101 references (C71) and by the Concrete Society in England with 80 references (C72).

Portland cement concrete, when used as a pavement material, is generally regarded as having one significant disadvantage - a low tensile strength in comparison to its compressive strength. This low tensile capacity can be significantly increased by the addition of discrete, randomly-ordered fibers which act as crack arrestors and signigicantly increase the ultimate tensile load-carrying capacity of the concrete. When the fiber's volumes are kept reasonably low - say less than around 4 percent by volume - the concrete's proportional limit does not seem to be significantly increased, but its ultimate flexural strength can be more than doubled (C73). There seems to be no quantitative expression yet for the prediction of the strength of the fiber-reinforced concrete as a function of the strength of the plain concrete, strength of the fibers, fiber volume, fiber geometry, etc. However, experimental data do indicate the significant improvement of ulitmate strength in either flexure or indirect tension, which is very important for a pavement material.

Fibers have been produced from steel, plastic, glass, and natural materials in various shapes and sizes (C71). Of these, steel and glass have been found to be the most useful and cost effective. In the following paragraphs each of these fibers will be discussed in terms of the current state-of-the-art.

Steel Fibers. Typical aspect ratio (fiber length divided by equivalent fiber diameter) range from 30 to 150 for lengths 0.25 to 3 in. (6.35 mm to 76.2 mm). Both round and flat steel fibers have been produced and used, as well as crimped and deformed steel fibers (C71). Various types of steels have been used with tensile strengths from 40 to 600 ksi (275 MPa to 4,100 MPa).

Steel fibers have a tendency to ball or clump together in the mixer, thus mixing has been a problem. Due to these difficulties the aspect ratio should be limited to less than about 100 and fiber contents should be limited to less than around 2 percent (C71). Furthermore, the maximum recommended coarse aggregate size is about 3/8 in. (10 mm). ACT Committee 544 has made specific recommendations about the mixing and placing procedures to be followed to reduce segregation and promote proper coating of the fibers with cement paste (C71).

Several different applications of steel fiber concrete have been utilized, including a number of pavement applications. Excellent summaries of pavement applications have been reported by Lankard and Walker (C74), Gray, et. al. (C75), and Pasko (C76). Problems do exist in the use of steel fiber concrete according to Pasko (C76).

> These problems can be crudely grouped into three major areas; economics, materials and structural.
>
> Economically, the cost per cubic yard of fiber reinforced concrete is high primarily because of the steel cost and secondly because of the high cement content, smaller aggregates, and the extra materials

to be handled such as fly ash, water reducers, and steel. The steel itself presently costs about 18 cents per pound (40 cents per kg) which, with greater demand and the use of scrap, may possibly decrease to 10 cents per pound (22 cents per kg) in the future. Tremendous gains have already been made when one remembers that several years ago the cost was 65 cents per pound (143 cents per kg) for this specialty product. Based on laboratory tests, the amount of steel necessary in the concrete appears to be between 1 and 2 percent by volume which is 133 to 265 lbs per cu yd (78 to 157 kg per m^3). A lesser amount does not provide much advantage over plain concrete and greater amounts are less effective and may cause mixing difficulties.

By comparison, a continuously reinforced concrete pavement 8 in. (200 mm) thick with a cross sectional percentage of steel of 0.64 percent (No. 5 bars on 6 in. (150 mm) centers) contains only 85 lbs per cu yd (50 kg per m^3) or about 1/3 of that in fiber reinforced concrete containing 2 percent steel by volume.

Hopefully, through the use of different sizes and shapes of fibers and new cements it may be possible to reduce the amount of steel, but this is not imminent at this time.

C-25

In regards to materials, some difficulties still exist, but great progress is being made in resolving them. . . Handling and dispersing the steel is still difficult but this could be easily overcome by developing special purpose equipment. Similarly, some changes may possibly be needed in the paving train, particularly in vibration, to improve the placement of stiffer mixtures. Some development work has already been started on building a device to automatically convey the fibers into the mixer.

While the difficulties with materials are slowly being ironed out, the structural design of fiber reinforced concrete pavements has largely been neglected to date. Much information is presently available on regular concrete pavements which can be judiciously applied to evolve designs with which the slabs could be built to resist the ravages of environment and loads.

The limited work done at the Waterway Experiment Station both in load testing and theoretical analysis indicate that the thickness of fiber reinforced concrete slabs-on-grade and overlay should be roughly one-half of the thickness required for plain concrete slabs. Although this is a good starting point it will have to be further substantiated and defined for other conditions.

C-26

The design of jointing has yet to be resolved. ...If the cracking is allowed to randomly occur and form irregular, erratic patterns, traffic will soon cause spalling and ravelling of the concrete faces which will create rough riding, unsealable cracks. Water and debris can then infiltrate the cracks and lead to pumping, faulting and blowups.

It is therefore necessary that a joint pattern be superimposed on the fiber reinforced concrete ribbon to control the cracking. Also, since the slab ends are the weakest point of a slab it is necessary to provide some type of continuity at the cracks or joints such as dowel bars or other devices.

Glass Fibers. Considerable research has been conducted on the use of glass fibers in concrete (C72, C75, C77, C78). Typical glass fibers have diameters of 0.2 to 0.6 in. (0.5 to 15 mm), but these fibers may be bonded together to produce glass fiber bundles with diameters of 0.5 to 50 in. (0.013 to 1.3 mm) (C70). Lengths of the glass fibers are similar to steel fibers (0.25 to 3 in. (6 to 76 mm), so their aspect ratios range from about 30 to 150. The glasses used for these fibers have tensile strengths from 150 to 550 ksi (1,000 MPa to 3,800 MPa). Several glass fiber compositions are in use today, two of which are in general production (C72). They are the soda - lime -

C-27

silica glass (A glass) and the borosilicate glass (E glass). Proprietary treatments have been used to prevent detrimental chemical reactions between the glass and the alkalies in the portland cement (similar to alkalie-silica aggregate reactions) (C71).

Applications of glass fiber concrete in pavements have been limited. One field test section has been reported (C79, C80), with results encouraging thus far. The glass fibers were added at the job site, in a ready-mix truck and thoroughly distributed throughout the mix in volume percentages of 1 and 1.5. No segregation or balling was observed. The only problem reported was that initially the concrete was very stiff after the addition of the fibers, and a higher water/cement ratio had to be used to provide workability. Another field application was conducted in Minnesota (see Appendix B) and some construction problems were reported.

Analysis. From the experience to date, it appears that fiber-reinforced concrete offers considerable potential for producing pavement sections much thinner than regular portland cement concrete. Practically speaking, there are problems with the construction and cost of fiber-reinforced concrete. These need to be overcome before the concept will be utilized to any appreciable extent.

If a thin overlay is deemed necessary, then fiber reinforced concrete might well be used. However, where partial or full-depth replacement is necessary, the additional cost of the fibers do not seem to warrant its extensive use at this time.

C-28

INJECTION REPAIR OF CRACKED PCC PAVEMENT

The injection of cracks with an adhesive has been used successfully as a method of in-place repair for more than 10 years. Typically, a high strength epoxy adhesive is used.

Many structural elements have been repaired, including a concrete bridge seat beam (C81), concrete floor (C82), a concrete craneway (C83), and concrete pilings (C84). Cracks as small as 0.002 in. (0.051 mm) (C83) can be repaired by using a pressure injection technique. Typically, the crack is sealed at the surface leaving small ports at regular intervals. The epoxy is then injected under pressure (50 to 300 psi or 345 kPa to 2,000 kPa) into the first port until it becomes visible at the second port and then that port is used until the epoxy is visible at the next port and so on until the crack is filled. Curing time for the adhesive is usually 7 days @ 25°C to develop tensile strengths of 6,890 psi (47.5 MPa) (C84). . Flexural strengths of 9,890 psi (68.2 MPa) can be developed in 24 hrs, increasing to 11,400 psi (78.6 MPa) at 7 days. Similar compressive strengths are developed. In most cases, greater than 80 percent of original structural capacity is restored.

Depending on the size of the crack to be repaired, the cost may be as much as $1 per lineal ft ($0.30 per m) (C85). If the pavement slab is badly cracked, some faulting is also likely to occur. This would necessitate slab jacking prior to crack repair. This fact, coupled with the relatively high cost, would seem to limit adhesive injection as an economical method of major pavement repair.

SULPHUR SYSTEMS FOR PAVEMENTS

Background

Shell Canada, Ltd. is currently developing a sulphur-asphalt-sand (SAS) paving material called "Thermopave." The techniques developed and the results of actual field tests were reported in a series of technical papers by Deme, et. al. (C86-C89). The Shell program has included the construction and monitoring of the following highway test and demonstration sections during the period of 1964 to 1974, inclusive.

Date	Location	Feature	Length*
1964	Oakville, Ontario	Surfacing (regular aggregate-rolled)	600 feet
1964	St. Boniface, Manitoba	Surfacing and Base (regular aggregates-rolled)	1000 feet
1970	Richmond, B.C.	Base and Working Platform (sand mix-screed)	1000 feet
1970	Richmond, B.C.	Surfacing (sand mix-screed)	200 feet
1971	Oakville, Ontario	Surfacing (sand mix-screed)	3250 feet
1972	Toronto, Ontatio	Base (sand mix-equipment studies)	400 feet
1972	Burlington, Ontario	Surfacing (sand mix-equipment studies)	150 feet
1972	Quebec, P.Q.	Base and Surfacing (sand mix-equipment studies)	2640 feet
1972	Tillsonburg, Ont.	Base (sand mix-structural studies)	5280
1973	Tampa, Florida	Surfacing (sand mix-equipment studies)	1000 feet

(continued)

Date	Location	Feature	Length*
1973	Sinaco, Ontario	Surfacing (sand mix-equipment studies)	1000 feet
1974	Tampa, Florida	Surfacing (sand mix-equipment studies)	1000 feet
1974	MacLean, Saskatchewan	Base and Surfacing-full depth (sand mix-low temperature studies)	2.3 miles

* 1 ft. = .305 m
1 mi. = 1.609 km

A study under the joint sponsorship of the Bureau of Mines and the Sulphur Institute was initiated on 1 May 1973 with the Texas Transportation Institute to investigate the beneficial use of sulphur in sulphur asphalt pavements and the techniques under development in Canada.

Under Phase 1A of this study (C90), basic engineering properties associated with highway pavement construction of sand-asphalt-sulphur (S-A-S) composites were evaluated through a laboratory program of mixture design, specimen preparation and testing. A limited theoretical analysis of the fatigue life of a sand-asphalt-sulphur pavement compared to a conventional asphaltic concrete system was also performed. Other activities included in this phase were:

(1) The consolidation of responsible and readily available sources of technical information and data concerning S-A-S pavements.

(2) The establishment of a literature and data bank on the general subject of the use of sulphur in pavements.

(3) Preparation of preliminary specifications and recommended design, construction and quality control procedures.

During January 1974 a final report on Phase IA was prepared in three volumes.

Volume I: "Familiarization and Verification of Existing Technology"

Volume II: "An Annotated Literature Search and Patent Review"

Volume III: "User's Manual of Preliminary Design, Construction and Quality Control Procedures."

In December 1973 Phase IB of this project was initiated which extended the objectives of Phase IA to include the evaluation of S-A-S mixtures under repeated loads. Engineering properties evaluated in this phase included stiffness, resilient modulus and flexure fatigue under constant stress and constant strain loading modes. These materials' properties were incorporated into an equivalent pavement thickness analysis from which the relative construction costs of S-A-S and conventional asphaltic concrete were compared. The final report on Phase IB (C90) was completed in August 1974 and distributed along with updated, revised copies of Volumes II and III.

The current research program which was given the designation of Phase IC was initiated in July 1974 and was designed to conduct preconstruction testing and analysis and to select a site to incorporate a sulphur-asphalt field trial into a scheduled state highway construction project. At the time of this writing a site has been selected along U.S. Highway 77 in Kenedy County, Texas, with construction tentatively scheduled for the Summer 1976. Current effort on the program has since been redirected to satisfy the immediate

technical and logistic needs of this field trial. The final report on Phase IC was published during September 1975.

Attempts have also been made to use sulphur in conjunction with asbestos and glass fibers (C91, C92). The latter provides reinforcement to the sulphur and can produce composites with flexural strengths as great as 7000 psi (48,000 kPa). Fibers have been used in asphaltic (C93) and portland cement concrete (C94) mixes with varied degrees of success. In this study attempts were made to capitalize on the advantages of sulphur in glass fiber reinforced sand-asphalt-sulphur (G-S-A-S) mixes.

Sand-Asphalt-Sulphur (S A S) Mixes

Sand-asphalt-sulphur mixes are prepared in the laboratory using two separate wet mix cycles. In the first, asphalt and sand, both preheated to 300°F (150°C), are mixed for approximately 30 seconds. This operation puts an asphalt coating on the surfaces of the sand particles. Sulphur, also at 300°F (150°C), is then added to the hot sand-asphalt and mixed for about 30 seconds*. This time is necessary in order to achieve a uniform dispersion of the sulphur throughout the mix. The molten sulphur, in addition to increasing the workability of the mix, normally fills the voids between the asphalt-coated sand particles. As the mixture cools below about 245°C (120°C), the sulphur solidifies creating a mechanical interlock between the sand particles from which the mix derives a relatively high degree of stability. This interlocking effect is shown in Figure C2, which is a photomicrograph

*Although mixtures have been prepared at temperatures up to 315°F (157°C), with temperatures greater than 300°F (150°C) hydrogen sulphide is emitted which could exceed safety standards established by the American Congress of Industrial Safety Hygenists.

of a mix matrix, illustrating how perfectly the sulphur conforms to the geometry of the voids.

Glass Fiber Reinforced Sand-Asphalt-Sulphur (G-S-A-S) Mixes

The mixing and sample preparation of glass-sand-asphalt-sulphur (G-S-A-S) mixutres should be accomplished while the temperature of the materials is between about 250 and 300°F (120 and 150°C). The former represents the melting (solidification) point of sulphur plus a tolerance to avoid structuring effects and the latter is the temperature above which the sulphur begins to undergo an abrupt and very large increase in viscosity as shown in Figure (C95), they do produce an adverse effect on the workability of the mix at temperatures above about 325°F (165°C). At mixing temperatures below 240°F (115°C), the sulphur acts as a filler, giving the material the characteristics of a high filler content mix.

Selection of a satisfactory mixture design should consider the surface area characteristics of the fiber along with its wetability. Problems encountered in the use of glass fibers with sulphur were largely related to the lack of an adhesive bond between the sulphur and glass and also due to the agglomeration of the glass fibers during mixing. Both conditions will adversely affect the physical strengths and freeze-thaw durability in the resulting mixes.

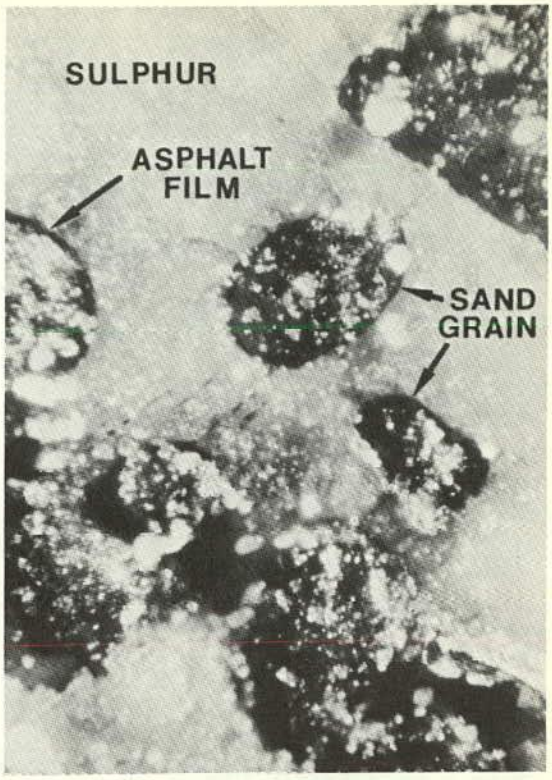

Figure C2. Photomicropraph of Sand-Asphalt-Sulphur Matrix Showing the Mechanical Interlock of Sand Particles Provided by the Sulphur. Mag. 120X.

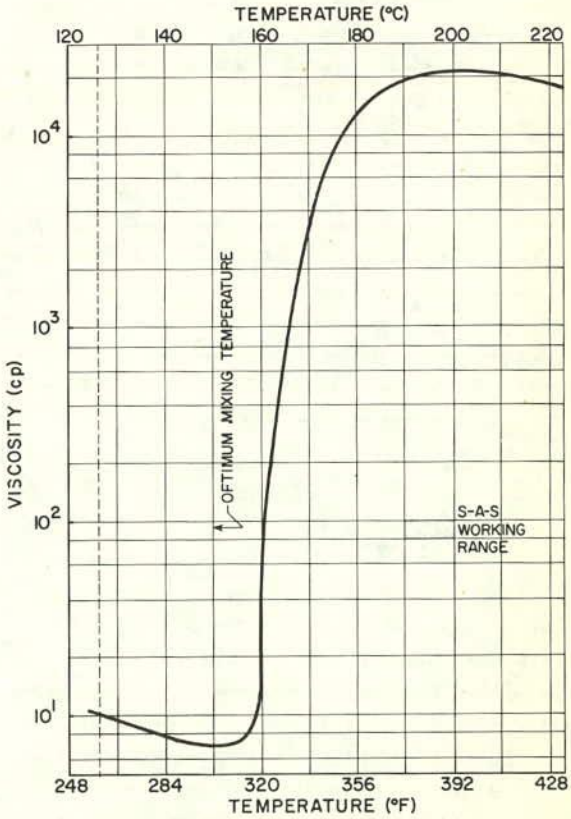

Figure C3. Viscosity-Temperature Curve for Liquid Sulphur.

Materials

The materials being studied are comprised of mixtures of sulphur, asphalt, sand, and four different types of fibers. Elemental sulphur (i.e., sulphur in the free state) of commercial grade (99.8 ± purity) was used throughout this investigation. All of the mixtures were prepared with an AC-10 asphalt conforming to 1972 Texas Highway Department Standard Specifications, Item 300 (C95). Shell claims that a wide range of penetration grade asphalts (40-300) have been suitably employed in S-A-S mixes (C96). The asphalt cement used in this study had a standard penetration of 106.

A particularly advantageous feature of sulphur-asphalt composites is that pavements prepared from heretofore unsuitable aggregates such as poorly graded sands usually have properties comparable or superior to pavements constructed with conventional asphaltic concrete mixes using high quality well-graded aggregates. Results indicated best results were obtained by using a poorly graded beach sand (C90). This was attributed to the relatively lower degree of angularity inherent in this material over the denser graded crushed limestones and gravels. Since low angularity is a necessary characteristic for particles embedded in a crystalline matrix, mixtures were prepared with a beach sand (Sand I) obtained from the Texas Gulf Coast area. For comparison purposes, the gradings and physical properties of this sand with a concrete sand (Sand II), a crushed limestone and a rounded gravel is shown in Figure C4 and Table C3, respectively. The properties of the Sands I and II reflect a relatively high VMA and low

unit weights as compared to the crushed limestone and gravel.

Table C3. Physical Properties of Aggregates

Designation	Aggregate Type	Specific Gravity	Voids in Mineral Aggregates (percent)	Unit Weight (pcf)*
Sand I	Blow sand	2.65	37.6	103
Sand II	Concrete sand	2.66	33.1	111
Limestone	Crushed limestone	2.65	17.8	136
Gravel	Rounded gravel	2.65	14.6	148

* 1 pcf - 16.2 kg/m^3

Three types of glass and a hammer-milled wool are being evaluated in the G-S-A-S mixtures. Glass Type A is a standard textile glass with an organic coating. The strand was chopped to a 1/2 in. (13 mm) length. Type B is the same as Type A except it was chopped to a 1/4-in. (6 mm) length. Type C is a 1/4 in. (6 mm) chopped textile glass strand with an inorganic surface coating developed by Homer Hill**. In addition to the glass a hammer-milled wool product was also evaluated. All fibers used in this phase of the project were obtained from the Owens-Corning Fiberglass Corporation, Granville, Ohio.

Laboratory Evaluation Sand-Asphalt-Sulphur (S-A-S) Systems

Mix and material properties tests of a wide variety of S-A-S

**Homer Hill Designation No. 1850-241-1.

mixtures are being performed using the aggregate and asphalt types discussed above. The specific mixture ratios evaluated ranged from 2:1 to 5:1 percent by weight of sulphur to asphalt. Initial mixtures were prepared with 7 percent by weight of fibers. The maximum amount of sulphur employed in any mixture was 20 percent by weight. The maximum amount of asphalt used was 7 percent. All fiber reinforced systems were prepared in weight ratios 80:6:14 of aggregate:asphalt:sulphur in which the 80 percent aggregate weight fraction was comprised of the sand and fiber. A number of unsuccessful attempts were made to prepare glass-sand-asphalt sulphur mixes. This was due in part to the sizing in the fibers which produced poor fiber-to-sulphur bonds. The cost of the glass as well as the need to surface treat the fibers made this appear impractical and was thus eliminated from further consideration.

Specific tests which are being performed on these mixtures include:

1.	Marshall Stability	ASTM D1559
2.	Percent Air Voids	Asphalt Institute Method (C97)
3.	Unit Weight	Texas Highway Department Test Method TEX 207-F (Rev. 1 Jan. 72)
4.	Hveem Stability	ASTM D1560
5.	Splitting Tensile Test	ASTM C496
6.	Resilient Modulus	Schmidt (C98)
7.	Freeze Thaw	ASTM C215 and ASTM C666

Table C4 is a composite which illustrates the effect of materials variables on the properties generated in tests 1 through 4. The asphalt cement used in these tests was AC-10 (106 penetration) and the aggregate was the poorly graded beach sand (Sand I). Comparative results using the concrete sand (Sand II) are also presented.

All data shown represent the average of at least three tests. These data indicate the Marshall Stability to be primarily affected by the sulphur content with a tendency to higher values as the sulphur

content increases, and lower values as the asphalt content decreases.

The properties obtained using the more densely graded concrete sand (Sand II) are also given in Table C4 for comparison purposes. Except for the 10 percent sulphur mixtures, the Marshall stabilities of the mixes using beach sand appear to be equal to or greater than for those using higher quality aggregate. This tends to become even more pronounced as sulphur content is increased. The Hveem stability was consistently higher for the beach sand mixes indicating the poorer graded aggregate to be more desirable for S-A-S pavements. This was contrary to that expected and is believed to be due to the effect of the stress concentration in the crystallized sulphur produced by the higher angularity of the surface of Sand II. This superior mechanical behavior of pavements using Sand I was also experienced in the splitting tensile tests (Test 5) to be discussed later.

The lower air void contents of the concrete sand are reflected in the higher unit weight and lower air content of the mix, both of which increase with the amount of asphalt and sulphur present. No significant difference in the Marshall flow was revealed in using the two aggregates.

Comparison with Conventional Asphaltic Concrete

Nine asphaltic concrete mixtures prepared by conventional techniques using the crushed limestone and rounded gravel aggregates shown in Figure C4 are being used as a basis for comparison with S-A-S and G-S-A-S mixtures. The Marshall Stability and flow, unit weight, percent air voids and Hveem Stability values for these mixtures with

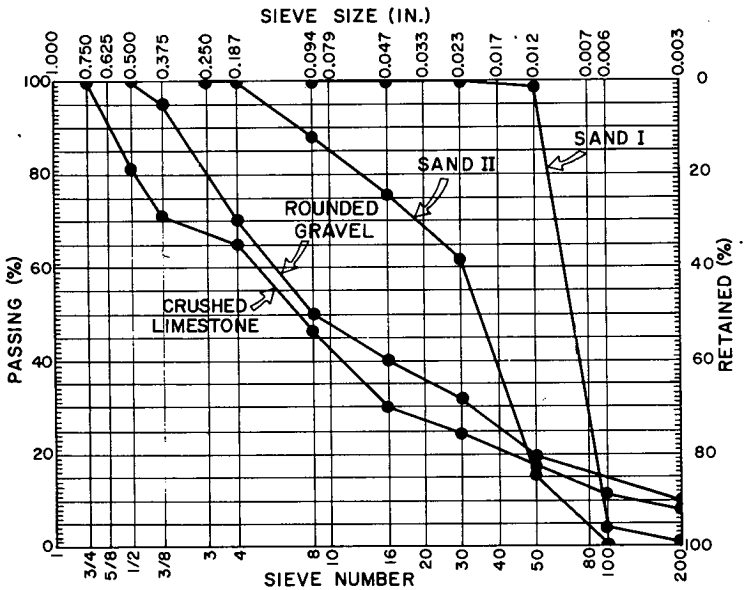

Figure C4. Aggregate Grading Chart.

*1 in. = 25.4 mm

Table C4. Sulphur-Asphalt Mix Properties With Sands I and II (AC-10 Asphalt).

Percent Weight		Marshall Stability (lb)*		Marshall Flow (0.01 in.)*		Air Voids (Percent)		Unit Wt. (pcf)*		Hveem Stability	
Asph.	Sulph.	I	II	I	II	I	II	I	II	I	II
3	0	0	--	--	--	28.4	--	113.0	--	--	--
5	0	50	--	30.0	--	25.2	--	114.5	--	--	--
6	0	300	--	25.0	--	23.9	--	114.7	--	--	--
7	0	110	--	20.0	--	21.5	--	116.7	--	--	--
8	0	88	--	27.0	--	19.0	--	118.7	--	--	--
0	10	1248	--	13.0	--	33.4	--	107.0	--	53	--
2	10	2219	1976	6.0	5.9	28.3	21.2	111.9	123.3	57	48
4	10	2100	2108	5.0	6.1	24.3	16.2	114.6	127.3	51	47
6	10	300	1231	8.0	6.3	18.2	11.6	120.3	130.4	28	36
7	10	90	998	7.0	6.8	15.9	10.5	122.0	130.1	20	30

75 compaction blows are shown in Table C5. These data reflect the superior properties inherent in the mixes using the crushed limestone. The data represented by Table C4 would indicate that, with the exception of unit weight and air voids, the properties of S-A-S mixtures can be tailored to provide the same range of values as those given in Table C5. At higher sulphur contents, the stabilities of S-A-S mixtures exceed those of the A/C with the crushed limestone. Because of the much higher VMA in Sands I and II, the air voids and unit weights of the A/C mixture could not be achieved by the S-A-S mixes. Unless otherwise noted, comparisons with other test results discussed in this report were made with the 6.5 percent asphalt mix whose design summary is also given in Table C5.

Splitting Tensile Test

Results of the splitting tensile tests (test 5) for various S-A-S mix designs are shown in Table C6. Comparisons are also made with the conventional asphaltic concrete mix design shown in Table C5.

In general, the splitting tensile strength of a mix increases with sulphur content and decreases with increasing asphalt content. It was interesting to note that beach sand (Sand I) gave higher strengths than the concrete sand (Sand II). One reason suggested for this effect was that the higher degree of particle angularity in the concrete sand particles created stress concentrations in the sulphur crystals which produced premature failure in the specimen when the load was applied. This is also consistent with the results of the Marshall and Hveem tests discussed earlier. The degree of superiority of Sand I over Sand II varies from test to test, but the consistently higher strengths

Table C4. Continued.

Percent Weight		Marshall Stability (lb)*		Marshall Flow (0.01 in.)*		Air Voids (Percent)		Unit Wt. (pcf)*		Hveem Stability	
Asph.	Sulph.	I	II	I	II	I	II	I	II	I	II
0	13.5	4607	6240	5.2	7.6	29.4	24.6	112.6	120.3	84	87
2	13.5	3692	3640	7.0	6.9	24.7	15.7	116.4	130.7	70	60
4	13.5	3300	3529	7.0	6.5	20.1	10.6	119.9	134.5	62	58
6	13.5	3280	3505	7.3	5.8	17.0	8.5	127.3	133.7	69	54
7	13.5	3224	1514	7.0	7.5	13.5	6.9	123.8	134.2	52	34
0	16	6812	9152	2.8	8.1	26.8	20.6	115.9	125.8	90	90
2	16	5044	5165	6.8	7.3	21.4	12.4	120.7	135.0	80	65
4	16	4993	3800	6.1	8.0	15.1	8.9	126.5	136.3	75	60
6	16	5131	3501	6.5	8.4	13.4	7.9	125.4	133.8	68	55
6	20	5193	4104	8.7	8.4	11.9	6.9	126.5	133.9	60	58

*1 lb = 0.454 kg 1 in. = 25.4 mm 1 pcf = 16.018 kg/m^3

Table C5. Asphaltic Concrete Mix Properties.

Asphalt Content (Percent)	Marshall Stability (lbs)*	Marshall Flow (0.01 in.)*	Unit Weight (pcf)*	Air Voids (Percent)	Hveem Stability
Gravel Asphalt Concrete - 75 Blows Compaction					
2.5	1628	7	144	9.4	41
3.0	1855	8	146	7.4	38
3.5	1793	8	148	5.2	36
4.0	1692	9	151	3.1	32
4.5	1610	12	152	1.6	20
5.0	1283	15	151	1.2	3
Crushed Limestone Asphaltic Concrete - 75 Blows Compaction					
6.0	5287	11	142	6.0	48
6.5	4983	18	145	3.7	49
7.0	5140	15	145	2.9	49

6.5% Asphalt Mix Design Summary

		% Asphalt by weight of mix
Maximum Density		6.7
Maximum Marshall Stability		5.9
4% Air Voids		6.0
	Av.	6.2

Design Asphalt Content

6.2% by weight of mix

6.5% by weight of aggregate

*1 lb = 0.454 kg 1 in. = 25.4 mm 1 pcf = 16.018 kg/m^3

of the beach sand provide an added measure of confidence in the use of this aggregate.

The results tabulated in Table C6 also give a first indication of the temperature and rate dependence of S-A-S materials.

The effect of specimen temperature on strength was determined by conducting the test at three temperatures: 20, 73 and 135°F (-7, 23 and 57°C). It is shown that the strengths decreased with temperature with the greatest change occurring between 20 and 73°F (-7 and 23°C). The strength changed by a factor of 6 for S-A-S mixes and 12 for asphaltic concrete at the lower deformation rates over this temperature range. The strengths of the asphaltic concrete samples were nearly the same as those for the S-A-S mixes with 20 percent sulphur at 73 and 135°F (23 and 57°C) but peaked at 690 psi (4800 kPa) at 20°F (-7°C).

The effect of deformation rate on splitting tensile strengths was determined using three speeds (2, 0.2 and 0.02 in./min or 51, 5.1 and 0.51 mm/min). The results shown in Table C6 indicate the strength to be directly related to the loading rates reaching a maximum of around 300 psi (2000 kPa) at 20°F (-7°C) with each rate. The 0.2 in./min (0.51 mm/min) deformation rate showed the greatest change in strength with the 20 percent S-A-S mixture and the A/C mix increasing by 20 and 40 percent, respectively.

The inability to obtain reliable strain data prevented the calculation of the stiffnesses in these mixes. A cursory examination of the data would indicate the stiffness of S-A-S to be higher than asphaltic concrete at very low rates. As the loading rates increase, the difference in

Table C6. Effect of Temperature and Deformation Rate on Splitting Tensile Strength of Various S-A-S Mix Ratios

Test Temp. (°F)*	Deformation Rate	Asphalt Percent Weight	Sulphur Percent Weight	Splitting Tensile Strength (psi)** Sand I	Sand II	Asph. Conc.
20	2	6.0	13.5	309	303	---
		6.0	16.0	312	306	---
		6.0	20.0	312	305	---
		6.2	---	---	---	690
73		6.0	13.5	116	87	---
		6.0	16.0	127	79	---
		6.0	20.0	123	87	---
		6.2	---	---	---	156
135		6.0	13.5	28	17	---
		6.0	16.0	37	18	---
		6.0	20.0	43	23	---
		6.2	---	---	---	54
20		6.0	13.5	302	267	---
		6.0	16.0	327	267	---
		6.0	20.0	315	270	---
		6.2	---	---	---	685

Table C6. Continued

Test Temp. (°F)*	Deformation Rate	Asphalt Percent Weight	Sulphur Percent Weight	Splitting Tensile Strength (psi)** Sand I	Sand II	Asph. Conc.
73	0.2	6.0	13.5	62	42	---
		6.0	16.0	79	42	---
		6.0	20.0	80	51	---
		6.2	---	---	---	84
135		6.0	13.5	13	8	---
		6.0	16.0	18	8	---
		6.0	20.0	26	11	---
		6.2	---	---	---	21
20		6.0	13.5	307	277	---
		6.0	16.0	292	283	---
		6.0	20.0	323	287	---
		6.2	---	---	---	685
73	0.02	6.0	13.5	38	23	---
		6.0	16.0	50	21	---
		6.0	20.0	54	26	---
		6.2	---	---	---	56
135		6.0	13.5	7	3	---
		6.0	16.0	9	4	---
		6.0	20.0	14	7	---
		6.2	---	---	---	17

*$t°C = (t°_F - 32)/1.8$

**1 psi = 6.89 kPa

rate dependence of the two materials produce comparatively higher changes in stiffness in the A/C than in the S-A-S system.

Results of tests conducted by Shell indicate the stiffness of S-A-S to be about 20 times greater than asphaltic concrete under constant load creep tests at 70°F (21°C) (C96). Other data (C86) shows the dynamic stiffness of S-A-S and A/C to be 660,000 and 900,000 psi (4500 and 6200 MPa), respectively, at 50 Hz. It would appear that at light to moderate loading rates the higher stiffness would exist with the S-A-S mixtures.

Resilient Modulus

The results of the resilient modulus tests (test 6) are shown in Tables C7 and C8. Table C7 shows a comparison of the stiffness measurements at 70°F (21°C) obtained using the Schmidt device with those generated in the direct tension tests at a deformation rate of 20 in/min (508 mm/min). Resilient and direct tension moduli for the asphaltic concrete mixtures were 385,000 and 415,000 psi (2700 and 2900 MPa), respectively, and were slightly lower than any of those obtained for S-A-S mixtures using Sand I. These data along with those given in Table C8 are consistent with constant strain rate direct tension test results which indicate an increase in stiffness with sulphur content and reduction in temperature. Except for an apparent inconsistency at the 68°F (20°C) test temperature, the mixtures using Sand I produce higher stiffnesses than those for Sand II. The range of moduli for the S-A-S mixtures extends from 240,000 to 1,851,000 psi (1,700 to 13,000 MPa) and reemphasizes the ability to significantly tailor the mechanical properties of these mixtures by altering the

Table C7. Comparison of Resilient and Direct Tension Moduli at Various Asphalt and Sulphur Contents.

Test Temperature: 70°F (21°C)

Percent Asphalt	Percent Sulphur	Resilient Modulus (psi)*	Direct Tension Modulus (psi)* @ 20 in/min. (508 mm/min)
0	10.0	551,000	
	13.5	1,649,000	
	20.0	4,431,000	
2	10.0	610,000	
	13.5	854,000	790,000
	16.0	---	1,500,000
	20.00	1,851,000	2,000,000
4	10.0	575,000	---
	13.5	1,194,000	1,052,000
	16.0	---	1,025,000
	20.0	1,382,000	1,500,000

(Continued)

Table C7. Continued.

Percent Asphalt	Percent Sulphur	Resilient Modulus (psi)*	Direct Tension Modulus (psi)* @ 20 in/min. (508 mm/min)
6	10.0	481,000	---
	13.5	441,000	460,000
	20.0	542,000	---
	13.5	513,000 (Sand II)	330,000
Asphaltic Concrete		385,000	415,000

*1 psi = 6.89 kPa

Table C8. Resilient Modulus Versus Mixture Design and Test Temperature.

Percent Asphalt (by weight)	Percent Sulphur (by weight)	Resilient Modulus psi X 10^{-3}* 40°F (4°C)	68°F (20°C)	90°F (32°C)
0	10	1086	1102	1082
	13.5	1929	1649	1171
	20	----	4431	2061
2	10	----	610	571
	13.5	1321	854	594
	20	1704	1851	966
4	10	----	575	462
	13.5	1051	1194	449
	20	1403	1382	653
6	10	696	481	311
	13.5 (Sand I)	885	441	294
	13.5 (Sand II)	805	513	240
	20	864	542	413

*1 psi = 6.89 kPa

sulphur and asphalt contents. Moduli for sulphur-sand systems (i.e. percent asphalt) ranged from 1.08 X 10^6 to 4.4 X 10^6 psi (7,000 to 30,000 MPa).

Freeze-Thaw Test

A limited number of mixes were subjected to a series of freeze-thaw cycles (test 7) in a chest-type freezer at a rate of 6 to 10 cycles per day. The fundamental transverse frequency at zero cycles, η, was measured at the beginning of that test. Additional values were also measured at 50, 70, 110 and 150 cycles. The tests results plotted in terms of the fundamental transverse frequency-squared versus the number of freeze-thaw cycles are shown in Figure C5 and indicate a rapid drop of η_c^2 values after 50 cycles of freezing and thawing. Since $(\eta_c)^2$ is directly related to the change in dynamic modulus, E_c, in accordance with:

$$\%E_c = \frac{\eta_c}{\eta}^2 \times 100\% ,$$

this sudden drop indicates an adverse change in structural integrity of the mixtures after 50 cycles. Shortly after these initial tests were run, the freeze-thaw equipment began to malfunction. Therefore, it is not possible to make any definite statements at this writing regarding the effect of other mix variables on the freeze-thaw durability. It should be noted that improved resistance to freezing and thawing was achieved from mixes with higher sulphur contents. This was attributed to the lower air voids contents associated with these mixes which in turn decreases their moisture susceptibility. In contrast to concrete, water has an adverse effect on the quality of the aggregate-asphalt bond. The high VMA (36-39%) associated with beach sands produce mixes with high air voids content which made them extremely vulnerable to freeze-thaw cycling.

Glass Fiber Reinforced Sand-Asphalt-Sulphur (G-S-A-S) Systems

During the preparation of the G-S-A-S mixtures it became difficult to obtain samples without incurring regions of dry (unwetted) agglomerated glass-fibers. Part of this effect was due to the poor bond quality between the sulphur and the glass which were influenced by the surface coatings on the glass. These coatings are applied during manufacture as a lubricant and to alter the surface tension within fiber bundles. Both the organic and inorganic surface coatings produced the same effect. This is consistent with the results of Dale and Ludwig (C91) who investigated a number of additives to improve the wetting and strengths of the sulphur-glass bond. One class of additives which was found to improve both wetting and strength were terpene polymers and includes Alpha-Pinene, Beta-Pinene and Dipentene. Since the use of additives could add significantly to the cost of producing a structurally adequate G-S-A-S composite, no further consideration was given to these systems.

Conclusions - Sand-Asphalt-Sulphur Systems

A series of standard pavement evaluation tests have been performed on a large number of mixtures comprised of different percent-by-weight ratios of sand, asphalt and sulphur. The testing program performed in this task was designed to qualitatively evaluate the influence of both material and process variables on engineering properties. The following represent some of the conclusions reached:

1. Sand-asphalt-sulphur pavement made with inexpensive, poorly-graded sands have been shown to have properties at least equal to or better than conventional asphaltic concrete.

2. For best workability and strength, processing should be

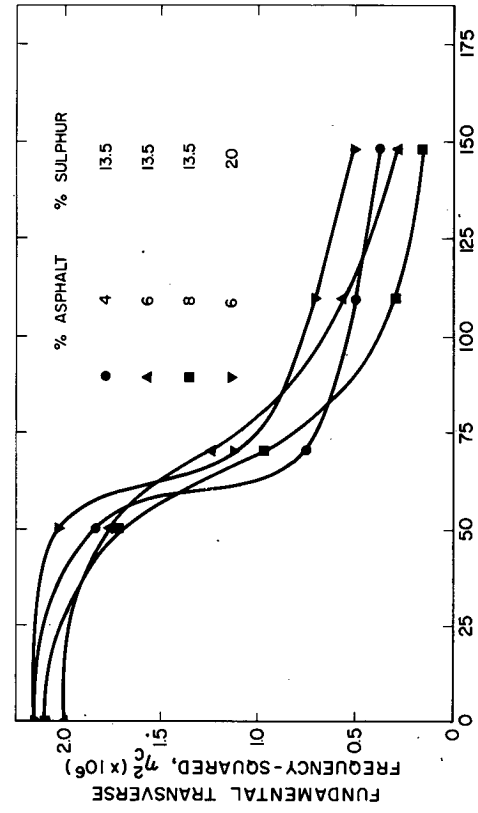

Figure C5. Fundamental Transverse Frequency-Squared Versus Number of Freeze Thaw Cycles.

accomplished at 250°F $\leq$ T $\leq$ 300°F (120°C $\leq$ T $\leq$ 150°C).

3. No adverse trends in properties were indicated after 28 days of post-cure.

4. The design properties of a mix can be tailored to a predetermined stability, percent air void or unit weight through adjustment of sulphur content.

5. Preparation of S-A-S materials should be accomplished with a minimum of compaction or densification effort. This could result in a significant economic advantage of S-A-S over asphaltic concrete during construction.

6. Economics of using S-A-S as a pavement material depends to a large extent on availability of low cost aggregate, and the cost and accessibility of sulphur to user.

Conclusions - Glass Fiber Reinforced Sand-Asphalt-Sulphur Systems

Problems were consistently encountered in trying to produce acceptable G-S-A-S mixtures. Primary source of the problem was the coatings on the fibers which produced a poor fiber to sulphur bond. The need to utilize special fiber surface treatments or additives to improve wetability eliminated this concept for further consideration in this project.

This does not imply that glass fibers if suitably treated will not enhance the physical strength of a sulphur-asphalt composite. When the surface coatings of the glass are removed either through burning or the use of additives they produced a significant improvement in the flexural strength of sulphur. This area is suggested for future study.

POLYPROPYLENE FABRIC

Reliable surface patching of distressed pavements is a primary concern of maintenance engineers. Many times, the distress in the original pavement will propagate under repetitive traffic loads causing the new patches to fail prematurely. Most often, the new surface fails because surface water penetrates the pavement and weakens the base or softens the subgrade. To prevent this, two basic remedies are required. First, the new patch must be waterproof and second, it must be reinforced to resist cracking.

Petromat, a non-woven polypropylene fabric developed by the Phillips Petroleum Company, has been shown to exhibit excellent moisture sealing qualities. In addition, the fabric possesses a high tensile strength which reinforces the system to retard crack reflection and, therefore, can be used for both construction and maintenance. Physical properties and characteristics of this material are:

Tensile Strength (either direction)	50 lb/in. of width (8,800 N/m)
Elastic Recovery at 15 lb (wet or dry)	100 percent
Weight	3-5 lb per sq yd (1.6-2.7 kg/m^2)
Color	Black
Standard Widths	75 and 150 in. (1.9 and 3.8 m)
Length/roll	100 yd (91.4 m)

In telephone communications with Dale Levy, a sales representative for the Phillips Petroleum Company, it was further established that as of May 1975, the cost of Petromat was \$0.46 per sq yd (\$0.38/m^2) and \$0.42 per sq yd (\$0.50/m^2) for quantities over 80,000 sq yd (67,000 m^2). The average construction cost for Petromat in place was approximately \$0.80 per sq yd (\$0.95/m^2), as generated from the following breakdown suggested by Mr. Levy.

Fabrication and Fr ight	\$ 0.49
Asphalt	\$ 0.10
Labor and Equipment	\$ 0.12
Contingencies	\$ 0.09
Total	\$ 0.80

Petromat fabric can be placed using inexpensive equipment in a relatively simple operation by regular maintenance personnel. The process uses a cationic asphalt emulsion applied by hand-held sprays by means of a hose connected directly to the distributor manifold. This is followed by a coating of aggregate which acts as a cover material over the fabric. An initial liquid asphaltic surface application is also made prior to placing the Petromat to establish a bonding surface.

The City of Overland Park, an 85,000-population suburb of Kansas City, first employed the Petromat surface treatment in 1970 when 1,000

sq yd (800 m^2) were placed on old asphalt pavements which had developed crack patterns. The method eliminated reflection of cracks through the overlay. The use jumped to 2,400 sq yd (2,000 m^2) in 1971 and 4,000 sq yd (3,300 m^2) in 1972 as a consequence of the good results obtained.

A spokesman for the city public works department has stated, "We've found that this fabric under the overlay not only eliminates crack transfer through the overlay but helps speed patching and restoration of usable streets. A patch job is done in 30 minutes, whereas the standard cut-and-patch method could take all day and involve partial closure of the street for two days."

Successful uses of Petromat have been achieved at two other locations; Bartlesville, Oklahoma, and on an overpass over I-70 in Talmadge, Kansas. In the latter, severe hair-line cracking of a bridge deck indicated a future trouble area. Traffic includes automobiles, trucks for hauling feed and cattle, and farm equipment. The West half was treated with Petromat fabric surface with chip seal to forestall anticipated concrete spalling. Very little change has occurred since installation in July 1967. A ranch service road near Bartlesville, Oklahoma was also treated. A Petromat fabric surface patch was placed over a distressed area prior to seal coating the entire road. After two severe winters the previously distressed area is still in excellent condition while the previously good surface has failed almost beyond repair. Typical cross sections for resurfacing are shown in Figure C6.

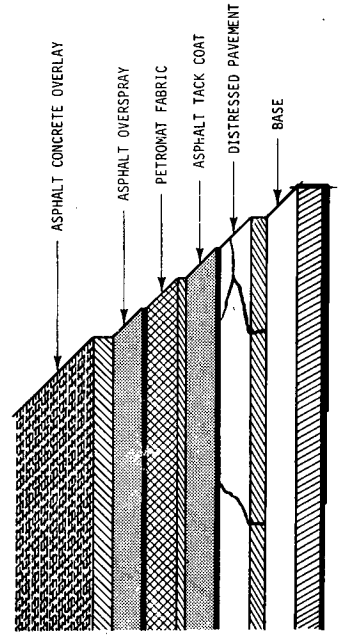

Figure C6. Resurfacing Using Petromat Fabric.

In September 1973 a high quality asphaltic concrete pavement was constructed by the Texas Transportation Institute on a section of the portland cement concrete airport runway system of the Texas A&M Research Annex. This pavement was constructed in partial fulfillment of the requirements of the Federal Highway Administration project entitled "Field Test and Evaluation Center for Central and Southern States". Part of the resurfacing utilized the Petromat technique.

The results of these tests show that whereas nearly 90 percent of the untreated joints transmitted reflective cracks after nine months, only 18 percent were transmitted through the Petromat. The rate at which the distress was generated was significantly lower in the treated surface. These results would indicate the effectiveness of Petromat as a surface treatment for retarding the initiation and propagation of reflective cracks.

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APPENDIX D

STRESS ANALYSIS OF THE PAVEMENT SECTIONS

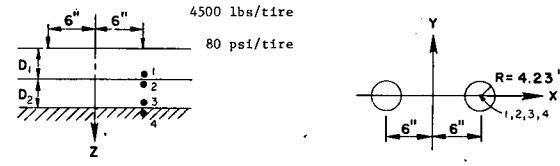
Stress analyses for interior loading were made with the BISTRO program which was developed by the Koninklijke Shell Laboratorium, Netherlands. This program solves for the displacements stresses and strains in a linearly elastic multi-layered pavement structural system. The stress analyses at a joint were performed using a two-dimensional plain strain linearly elastic finite element computer program. The wheel loads which were applied immediately adjacent to the joint were simulated as a strip load with uniform pressure which was calculated assuming that the entire load on each dual wheel arrangement was spread over the two tire contact areas and the pavement surface in between them. Comparison of the stresses calculated with BISTRO and with this strip load assumption showed that the latter are consistently about 5 percent lower, a result that was considered acceptably close for the purposes of this analysis.

The loading diagrams for the three wheel loads used in the BISTRO program are shown in Figure D1. The finite element layout and the loading diagrams for the joint stress analysis are shown in Figures D2 and D3, respectively.

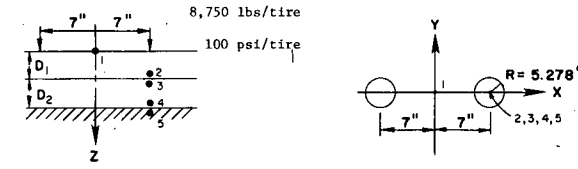
The physical properties used for the various materials are given in Table D1 and D2.

Typical partial outputs of the two programs are shown in Tables D3, D4, and D5. For the BISTRO program the values given are the principle stresses and strains. In the finite element program the

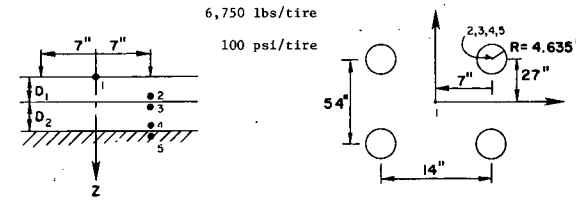
Dual Wheel 18 kip Single Axle



Dual Wheel 35 kip Single Axle

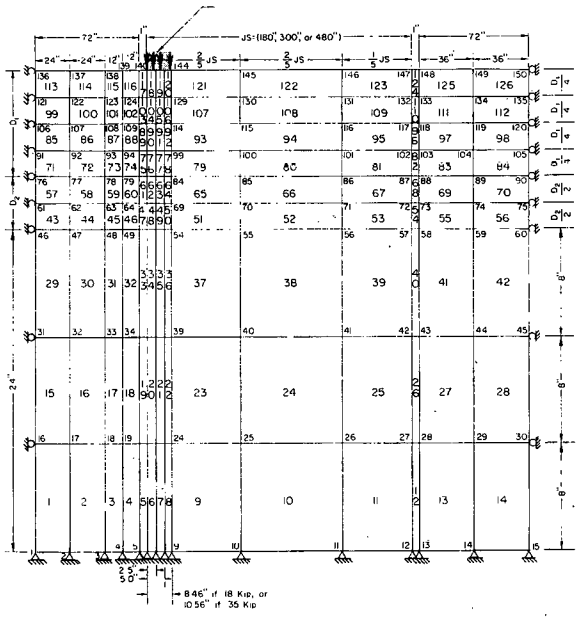


Tandem Wheel 54 kip Dual Axle



1 lb = .454 kg 1 in. = 25.4 mm
1 psi = 6,895 Pa

Figure D1. Loading Diagrams for BISTRO Program.



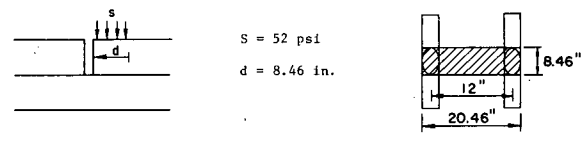
1 in. = 25.4 mm

1 kip = 454 kg

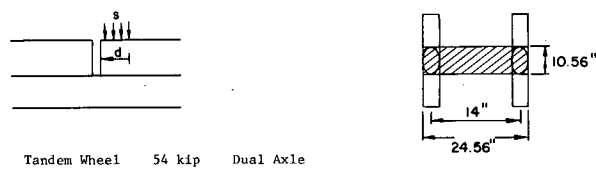
Figure D2. Finite Element Layout.

Note: The shaded portion in the slab is the joint.

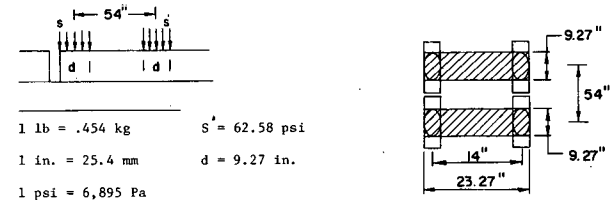
Dual Wheel 18 kip Single Axle



Dual Wheel 35 kip Single Axle



Tandem Wheel 54 kip Dual Axle



1 lb = .454 kg S = 62.58 psi
1 in. = 25.4 mm d = 9.27 in.
1 psi = 6,895 Pa

Figure D3. Loading Diagrams for Finite Element Program.

Table D1. Physical Properties for Stress Analysis.

Material	Modulus of Elasticity (E)(psi)	Poisson's Ratio (μ)
Asphalt Cement Concrete	460,000	.35
Sulphur Asphalt Concrete	465,000	.35
Rigid Urethane Foam	Range 4,800 to 30,000	.10
Portland Cement Concrete	3,000,000 (base course only)	.20
	4,000,000	.15
	6,000,000	.15

1 psi = 6,895 Pa

Table D2. Assumptions for Pavement Stress Analysis.

Item	Condition Assumed	Minimum Criteria	Candidate System/Material
Subgrade	Periodically or Continuously Saturated E = 5000 psi μ = .45		None
Base	4 to 20 in. thick May be placed directly on Subgrade	(1) Structural Adequacy in the Presence of Water	(1) Dense Graded Asphaltic Concrete (2) Sulphur Asphalt Systems (3) Rigid Foam if placed on Subgrade
Surface	4 to 10 in. thick	(1) Durable (2) Wear Resistant (3) Smooth Riding PSI > 4 (4) Sufficient Texture for High Skid Resistance	(1) Flexible Dense Graded Asphaltic Concrete (2) Rigid Rapid Hardening Concretes; with or without reinforcement including fibers; Cast in place or precast

1 psi = 6,895 Pa

1 in. = 25.4 mm

Table D3. Streamlined Dual-Wheel BISTRO.

Dual-Wheel Load 18 kip Single Axle - Number of Layers 3

Layer	Elastic Modulus	Poisson's Ratio	Thickness
1	4000000	0.15	8.0
2	465000	0.35	16.0
3	5000	0.45	0.0

Output No.	Layer	Depth (in.)	Sigma 1 (psi)	Sigma 3 (psi)	Epsln 1 (μ in/in)	Epsln 3 (μ in/in)
1	1	8	57	-12	13	- 7
2	2	8	2	-12	13	-29
3	2	24	15	0	21	-22
4	3	24	-1	0	21	-51

1 psi = 6,895 Pa

1 in. = 25.4 mm

Table D4. Nodal Displacements - Finite Element Analysis.

Node	U = X-Disp. (in).	V = Y-Disp. (in.)
46	0.0	-.0098
47	-.00064	-.0104
48	-.00104	-.0120
49	-.00104	-.0131
50	-.00047	-.0141
51	-.00037	-.0142
52	-.00013	-.0142
53	.00011	-.0141
54	.00044	-.0139
55	.00082	-.0070
56	.00066	-.0005
57	.00022	.0021
58	.00015	.0021
59	-.00012	.0020
60	0.0	.0019
76	0.0	-.0098
77	-.00013	-.0104
78	-.00020	-.0120
79	-.00026	-.0131
80	-.00033	-.0144
81	-.00043	-.0148

(Continued)

Table D4. Continued

Node	U = X-Disp. (in.)	V = Y-Disp. (in.)
82	-.00046	-.0146
83	-.00047	-.0144
84	-.00047	-.0140
85	-.00041	-.0071
86	-.00035	-.0005
87	-.00029	.0022
88	-.00016	.0021
89	-.00006	.0020
90	0.0	.0019

1 in. 25.4 mm

Table D5. Stresses at Element Centroids - Finite Element Analysis.

Element	X	Y	Sigma 1 (psi)	Sigma 2 (psi)
29	12.00	20.00	-1	-3
30	36.00	20.00	-1	-3
31	54.00	20.00	-1	-3
32	66.00	20.00	-1	-4
33	72.50	20.00	-1	-4
34	74.25	20.00	-1	-4
35	76.75	20.00	-1	-4
36	79.73	20.00	-1	-4
37	113.23	20.00	-1	-3
38	181.00	20.00	0	-1
39	235.00	20.00	0	0
40	253.50	20.00	0	0
41	272.00	20.00	1	0
42	308.00	20.00	1	0
57	12.00	33.00	-2	-6
58	36.00	33.00	0	-6
59	54.00	33.00	6	-7
60	66.00	33.00	12	-13
61	72.50	33.00	9	-58
62	74.25	33.00	-1	-26
63	76.75	33.00	4	-13

(Continued)

Table D5. Continued

Element	X	Y	Sigma 1 (psi)	Sigma 2 (psi)
64	79.63	33.00	7	-8
65	113.23	33.00	3	-5
66	181.00	33.00	-1	-1
67	235.00	33.00	2	-2
68	253.50	33.00	34	12
69	272.00	33.00	3	-2
70	308.00	33.00	1	0

1 psi = 6,895 Pa

displacement values are for the nodes and the stress values are for the certroid of the elements.

The two programs indicated reasonable feasibility of using the rigid foams or similar materials as base and sub-base coarses under heavy loads. However, some caution should be exercised because the fatigue characteristics are not available for these materials.

Using a fatigue criteria for the asphalt, sulphur asphalt and portland cement concretes indicates that almost all combinations of these materials could be used.

When the joint filler material was assigned an E value the same as the portland cement concrete the finite element program generated very similar stresses to the BISTRO system for the same wheel load. Thus giving reasonable confidence in the results of the finite element system.

APPENDIX E

PARTIAL DEPTH PRECAST PANELS

The "Klarcrete" cutter is a British device designed to cut rectangular holes up to 4 in. (100 mm) deep in Portland cement concrete pavements. The machine is physically described in Table E1. After the hole is "cut" then a precast portland cement concrete block of appropriate thickness is cemented in place using either rich cement mortar or epoxy. Less than four hours total time are required for these types of repairs.

Virginia (E1) has used the Klarcrete cutter to repair spalled areas. The major findings of the Virginia study are:

1. Precast concrete patching is feasible.

2. The Klarcrete machines used to cut the holes did a creditable job.

3. Although the installation of partial-depth precast patches was deemed successful, more projects are needed in the 300-500 sq ft (27.87 - 46.5 m^2) range to develop a methodology for increasing output.

4. Although the epoxy obtained from the United States sources met AASHTO standards and apparently worked well, the three-hour curing time was excessive and defeated one of the purposes of the experiment. However, the four gallons (15ℓ) shipped from England cured quickly enough to allow traffic on the patches in 30 minutes, which proves that rapid opening to traffic is possible.

5. The most efficient makeup of men and machines was one operator

Table E1. Technical Details of Klarcrete General Purpose Concrete Repair Machine. (From Reference E2.)

Item	Value
Overall length	14 ft. (4.3 m)
Length without tiller	10 ft 4.5 in. (3.16 m)
Width	7 ft 6 in. (2.29 m)
Overall height	4 ft 6 in. (1.37 m)
Weight	4,000 lb (1,800 kg)
Power required, compressed air[a]	600 cfm at 100 psi (0.28 m^3/s at 689 kPa)
Cutting heads[b]	
Diameter	2 1/4 in. (57.15 mm)
Distance between heads	2 1/4 in. (57.15 mm)
Air consumption per head	30-35 cfm at 100 psi (0.014-0.017 m^3/s at 689 kPa)
Life	100 ± 25 hr
Strokes per minute	1,500
Maximum cutting depth	4 in. (102 mm)
Accuracy	± 1/8 in. (±3.2 mm)
Width of cut	4.5 to 49.5 in. (0.11 to 1.3 m)

[a]The Klarcrete machine is pneumatically and hydraulically operated and pneumatically controlled on a sequential system.

[b]There are 11 tungsten carbide cutting heads, composed of a motor and a head; each head (except the first one) can be controlled individually. The cutting face is made up of 6 tips intersecting in the center of the head at 60 degrees. Depth of the cut depends on forward speed of the machine.

for each machine and a patching crew of three men, one of whom assisted the machine operators when necessary. The two machines kept one patching crew busy.

6. Precast concrete patching projects should have a bench type masonry saw on the job site capable of sawing precast slabs to exact dimensions. It is not practical to try to cast all of the different size slabs that may be needed.

7. The steel mesh in the pavement did not cause a problem in precast patching.

8. Adequate supplies of materials should be stockpiled prior to beginning the project.

9. To assure a neat job a canvas covering should be spread on the road surface where epoxy is being mixed to catch the drippings.

10. Preparation of holes by the Klarcrete machines to exact dimensions for cast-in-place concrete patching is an unnecessary sophistication.

The process has potential at the present time for small partial depth repairs (less than 12 sq ft by 4 in. deep or 1 m^2 by 100 mm deep). However, it would seem to be too limited for the larger problems of major surface repairs.

PARTIAL DEPTH REHABILITATION - ACC PAVEMENTS

Partial depth rehabilitation for ACC pavements are mostly to restore riding quality, restore skid resistance or seal the surface as opposed to improving structural capacity.

There are a number of techniques for partial depth replacement of ACC pavements. There are repavers which remove a portion of the thick-

ness, and either adds asphalt and lays down the same material in a single pass, or adds additional hot mix and lays down an overlay consisting of the old material and new material (see Cutler Repaver, page 57).

Planers may also be used to remove the unsound or rough material. Usually these removals are relatively thin (less than 4 in. or 100 mm) however, depths up to 13 in. (330 mm) have been removed.

After removal, the surface is either recycled (see Appendix C) or replaced in a conventional manner (as described in Appendix C). In the replacement process a scarifier is sometimes used to roughen the surface and promote bonding both with and without tack costs.

There are several equipment types available among which are:

1. Heater-Planer or Scarifier - Jim Jackson, Inc., Little Rock, Arkansas.

2. Planer - Galion Manufacturing Company, Galion, Ohio.

3. Repavers - Cutler Repaving, Inc., Lawrence, Kansas.

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APPENDIX F

SUBGRADE TREATMENTS

LIME STABILIZATION

Lime is the oldest stabilizing agent in use today. It has been used to strengthen structural systems, to provide serviceable working tables for construction equipment, and to reduce the swelling potential of certain clay soils. The most commonly used form is hydrated lime (calcium hydroxide) which, when mixed with plastic soil, causes two recognizable stages of reaction.

The first stage, called "soil modification", occurs within a few minutes to an hour after intimate contact with the soil. This stage is marked by a flocculation, agglomeration, and granulation of the soil particles, a reduction in plasticity and swelling potential, and considerable improvement in workability (F1). In the subsequent stage, referred to as "soil cementation", a compacted mixture slowly develops strength as a result of pozzolanic reaction and the formation of new compounds over a long period of time, sometimes years (F1).

In most applications, adequate stabilization can be accomplished with 6 to 8 in. (150 to 200 mm) of treated material. Typically, the lime is distributed over the soil and mixed with the soft, wet subgrade soil (AASTO groups A-6 and A-7) using light equipment. Then, it is intimately mixed and compacted with normal compaction equipment. If the soil is very soft, it may be allowed to "mellow" for a day or two after the preliminary mixing, in order to allow for the

development of the drying action.

The effectiveness of lime stabilization has been conclusively demonstrated. However, to provide effective swell control, it has been suggested that expansive soils should be treated to a depth of 5 ft (1.5 m) (F2). To accomplish this depth of treatment conventional methods are usually inadequate or too expensive, hence, other techniques have been devised.

Drill-time stabilization was developed as a maintenance measure to treat pavement failures due to a lack of stability in wet, plastic subgrades (F1). The procedure required 6 to 9 in. (150 to 230 mm) diameter holes drilled through the existing pavement and 30 to 50 in. (760 to 1270 mm) into the subgrade. These holes, spaced on 5 ft. (1.5 m) centers, are then partially filled with lime and water to form a slurry. The holes are then backfilled, tamped and patched. It is claimed that the lime migrates through the subgrade to cause sufficient stabilization, thus preventing additional movement. The results have been erratic. The major limitation seems to be the inability of the lime to distribute throughout the soil mass (F3).

In order to obtain a better distribution of the lime, a pressure injection lime (PIL) procedure was developed (F3). In the PIL procedure, a lime-water slurry (typically 3 lbs lime per gal of water or 360 kg/m^3) is pumped through injection rods into the soil. The injection rods are spaced 3 to 5 ft (1 to 1.5 m) apart and the slurry is usually pumped under 50-200 psi (345-1400 kPa) pressure.

It is apparent that openings must be present in the soil mass if adequate slurry distribution is to be obtained. Field test reports

include about as many successes as they do failures to achieve desired results (F3).

An excellent state of the art report (F3), citing 36 references (many less than 3 years old), sums up PIL as follows:

> There are conflicting reports concerning the effectiveness of PIL treatment of expansive soils. The proposed treatment mechanisms ("prewetting", the development of soil-lime moisture barriers, effective swell restraint with the formation of limited quantities of soil-lime reaction products) have validity. It therefore seems logical to conclude the PIL may be an effective swell control procedure under certain circumstances. The condition most favoring the achievement of successful PIL treatment of expansive soils is the presence of an extensive fissure and crack network into which the lime slurry can be successfully injected. The relative significance of the "prewetting" and "soil-lime pozzolanic reaction" aspects of PIL treatment have not been established. If soil-lime pozzolanic reactions are essential to achieving an effective application perhaps that fact can be used to evaluate the potential success of an anticipated treatment and explain the apparent conflicting PIL experience reports.

Due to the necessity of drilling a network of holes at 5 ft (1.5 m) intervals and the limited suitability of PIL in saturated fine grained soils, it would appear to have limited applicability to

the solution of rapid repair of urban freeways.

ELECTRO-OSMOTIC STABILIZATION

In the mid 30's Casagrande patented in Germany (and later the U. S.) a method of hardening clayey soils (F4, F5). He and many others worked on the process of electro-chemical soil stabilization during the next 30 years. More recently, O'Bannon (F6, F7) has reported the results of field tests in Arizona on stabilization of Chinle Clay by electro-osmotic treatment.

The process causes a base exchange of ions to reduce the swelling characteristics of clays. Among the advantages of electro-osmotic stabilization are:

1. The stabilization of existing highways can be accomplished without the necessity of removing and reworking the in-place material.

2. Once the treatment and stabilization of expansive clays had been accomplished, the results would irreversible over the engineering life of the highway.

3. After treatment and stabilization of the expansive clay, radical changes in moisture content of the subgrade would not cause excessive changes in volume.

Basically, the system consists of an anode capable of dispersing a saturated solution of a water soluble salt (K Cl, Ca Cl_2, Mg Cl, Al Cl_3) and a cathode placed up to 18 ft (5.5 m) away. The two are coupled with a DC power source capable of producing a voltage gradient of at least .5 volt/in. (.02 volt/mm).

O'Bannon found that about 0.5 percent KCl by weight of soil was required to stabilize the Chenle clays. His work also revealed that a

horizontal arrangement if anodes and cathodes were the most effective.

Stabilization to a depth greater than 3 ft(1 m) below subgrade is usually not necessary. In 1969 in a field test by O'Bannon, a Chenle clay was treated to a depth of 3 ft (1 m) at an estimated cost of $5.432 per lineal foot ($17.82 per m) of treated section. (Cost based on a section of 200 ft (60 m) long by 40 ft (12 m) wide and is the non-recoverable items.) The treatment required about one week. At todays prices the treatment would cost considerably more.

The process does work and can be installed with a minimum of interrupted to traffic flow. It is envisioned that this process could be used to treat "soft spots" along a pavement section to stabilize the subgrade prior to removal and replacement of the pavement. Also, this technique could probably be used to stabilize expansive soils under pavements where the only rehabilitation work to be done is to restore riding quality as described in Chapter Two.

FIBER MATS

A number of fiber filled membranes are currently manufactured for various construction uses. Polypropylene is the basic component in most of the fabrics. (See Appendix C)

The marketing brochures for some of these fabrics promote the following uses:

(1) Moisture barriers for concrete construction.

(2) Construction covers.

(3) Road support membrane.

(4) Strengthening membrane in layered pavements.

(5) Pond and ditch liners.

Some published results include:

(1) The prevention of reflection cracking in asphalt patches

(F8). In this installation a fabric mat was placed over an asphalt pavement exhibiting alligator cracking. The mat was bedded in liquid asphalt and covered with a sprayed-on cationic emulsion. A conventional, hot-mix overlay was then placed on top of the mat. Good results were obtained.

(2) The military has encapsulated a soil layer in a plastic fabric for use as a roadbed over very soft soils (F9). The process, termed membrane enveloped soil layer (MESAL), consists of a subgrade upon which a foundation layer of compacted soil lies between waterproof membranes forming a waterproof encapsulated soil system. A 12 in. (300 mm) lift of soil compacted to 95 percent of AASTO T-180 will support 5 ton (4,500 kg) 6 x 6 military cargo truck loaded to its maximum weight for highway travel.

In relation to rapid rehabilitation of urban freeways, there are two proposed uses. One would be to use the fabric directly on a soft subgrade as a load distribution membrane to reduce the thickness requirement for base or subbase courses. The membranes could be a continuous strip or it could be used intermittently if the subgrade were only soft in spots. The second application would be to use the fabric as a reinforcing layer between layers of asphalt concrete to assist load distribution and reduce thickness requirements.

The products discussed here are commercially produced by several manufacturers including Phillips Petroleum Company under the trade name of Petromat, DuPont under the trade name of Typar and Celanese Fibers under the trade name of Mirafi.

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