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Planning for the Future

RICHARD W. KRESS
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•LONG BEFORE the Illinois Tollway was opened for traffic, plans were formulated for the organization of the Maintenance Division with its necessary personnel and equipment. It was organized so that it would be workable and could be efficiently operated at the lowest cost per unit of activity commensurate with established expressway maintenance standards. After $4\frac{1}{2}$ yr of operation, this planning has proved to be basically sound. The planning for this operation started with a study of the maintenance operation of existing toll facilities and of the maintenance organizations of the surrounding States. From these observations, an organization that best suited operations under conditions in the area near Lake Michigan was planned.

The equipment and personnel for roadway maintenance were based on the winter requirements. For the first 2 yr of operation this type of planning proved adequate to carry out summer activities. After the second year of operation, temporary summer help had to be employed to maintain the trees and shrubs around the buildings and interchanges and to assist in the guardrail painting program. A list was also made of men available for call-out duty during a severe winter storm period. Other than these additions and minor changes, the initial organization has remained.

Divisions and Sections

The Tollway, for maintenance purposes, was divided into three Divisions. The Divisions consist of various sections as follows: Division I, Sections 1, 2 and 8; Division II, Sections 3 and 4; and Division III, Sections 5, 6 and 7. Each Division was headed by a supervisor and each section by a foreman. The roster at each section building varies with the number of trucks assigned to that section and an additional number at the three division garages. The Maintenance Division includes personnel engaged in building maintenance, lighting, communications, toll collection equipment, and automotive and equipment maintenance.

Personnel

Assigned to duties at each of the three Division Headquarters are one electrician for roadway lighting, one electrician for building lighting, one tradesman for building maintenance, and one grader operator for section grading. Each of the eight sections is assigned personnel depending on the amount of anticipated work. Number of personnel and the size of each section are given in Table 1. Additional personnel serving the entire facility for specialized duties are one water and sewage technician, one carpenter, one welder, one stand-by generator and heating specialist, and one gradall operator. Sign erection and their maintenance are under the direction of the Traffic Engineer. The personnel engaged in this activity consists of one foreman, one shop man and six men in the field.

Each of the eight maintenance sections has its own building located approximately midway in the section. This central location eliminates the need for auxiliary stockpiles of chemicals, abrasives and loading equipment located at a distance from the building. In most cases the section buildings and storage yards are located within an interchange which enables vehicle movement in either direction without crossing the median.

Paper sponsored by Committee on Maintenance of Controlled-Access Highways.

TABLE 1

DESCRIPTION OF SECTIONS

Section	Length (mi)	Main Line Lane Mi	Lane Mi With Ramps	Service Area Restaurant	Personnel (no.)
1	23.28	93.12	102.93	No	13
2	21.80	114.50	126.00	Yes	18[a]
3	25.52	120.75	132.98	No	18[a]
4	20.90	95.16	102.53	Yes	15
5	20.36	81.46	86.55	Yes	12
6	30.67	122.68	130.68	No	14[a]
7	22.44	89.76	93.82	Yes	12
8	22.85	91.40	98.90	No	11

[a]Three workers assigned to manning the building on a 24-hr basis.

ACTIVITIES

A very close schedule of activities must be adhered to so as to accomplish all necessary seasonal work with available man power.

Pavement

Probably of greatest importance in the preservation of the highway is the constant maintenance of the 7,250,000 sq yd of concrete pavement that make up the driving lanes, bridge decks and ramps. This includes the mudjacking of depressed areas, filling of joints and cracks, and repairing scaled bridge decks. Mudjacking is the operation of drilling holes at predetermined locations in a depressed slab area and introducing a slurry of cement, limestone and fly ash under pressure to fill the void and raise the slab up to its original elevation. This operation is time consuming and, due to the erratic movement of the slab, it is sometimes necessary to pump the area at intervals of 1 or 2 wk until the desired result is obtained. Unfortunately, this jacking, especially at bridge approaches, has exceeded original planning by approximately 100 percent.

Transverse and longitudinal joint filling consists of routing out the joint-filling material and applying, under pressure, a rubber-based asphalt sealer. This sealing material must withstand pavement contraction and expansion throughout the range of annual temperatures. The prevention of the infiltration of surface water, especially during the winter and spring months, is of prime importance to forestall the entrapment of water under the slab which may eventually cause very destructive slab pumping. The granular subbase under the pavement slab is designed to prevent this type of failure. Initially, it was anticipated that the joint-filling materials would need replacement after 3 yr. At least 95 percent is still in excellent condition after 5 yr. The longitudinal separation that has developed between the concrete slab and bituminous shoulder is due to heaving and settling of the shoulder during the first few seasons following construction. Several materials have been tested to fill this separation and hot poured asphalt containing a mineral filler is presently being used. This particular activity was not in the original planning.

The bituminous shoulders have stood up very well, except in some small areas where water was trapped in the subsurface. These areas have been drained with the installation of a short piece of 6-in. perforated corrugated metal pipe or with the placing of aggregate to act as a french drain. A shoulder-sealing program has been started chiefly to liven up the top surface.

Bridge Structures

Bridge structure maintenance, other than sweeping, scupper drain cleaning and damage repair, was an activity that was thought would need very little attention for the first 5 yr. Repainting the structural steel members was to be done after 8 yr of weathering. This forecast was approximately 95 percent correct. Five percent of the bridge decks have developed scaling. Unfortunately, one that has required the most repairing is the "Mile Long Bridge," a twin structure 5,000 ft long. Patching and resurfacing has been done by use of epoxy resin for smaller areas, pneumatically placed concrete, and asphalt. Before resurfacing with the latter an epoxy resin coating is laid down to act as a moisture barrier between the old concrete and the overlay. Deck surfaces, in the future, will give more concern than originally contemplated. The repair of steel expansion devices has been necessary from the time the highway was opened.

Guardrail

Painting the guardrail, approximately 700,000 lin ft, is a considerable task and expense. Original plans indicated a painting every 4 yr. This schedule has been cut to every other year so that there would be a minimum amount of preparation. The longer period necessitates a full-scale cleaning to remove accumulated rust and corrosion. Much time was spent in experimenting with rust-inhibiting paint formulas to eliminate the time-consuming application of a prime coat.

Each of the three Divisions has a paint crew consisting of one permanent employee and two temporary summer employees. Each crew with a spray gun unit will average 1,500 lin ft of rail painting per 8-hr day for both sides and up to 2,500 lin ft for front side only.

A factory-coated galvanized beam has been placed at various locations to study its weathering capacity. Perhaps in the future it may prove economical to galvanize the present rail to eliminate the need for painting.

Experimental use of a custom-built guardrail maintainer which washes, scales, and paints the rail, has shown possibilities, especially for use on the front side.

During the past 2 yr, the approach ends on the installation have been buried to minimize fatalities and serious injuries when struck head-on at high speeds. To date, this has succeeded.

Erosion Control

From past experience, it was realized that it would take 3 to 5 yr after the project was completed to stabilize the cut and embankment slopes and to obtain a permanent turf cover. Marked progress has been made during the past four seasons in stabilizing the slopes, although there are a few critical spots that will require continued attention. Several methods have been used to correct the drainage situation, which in most cases is the basic requirement for eliminating erosion. The methods used have included the placing of a diversion ditch along the top of cut slopes, the installation of perforated metal pipe to pick up groundwater pockets, the placing of a stone blanket to act as a french drain, and the use of a curb along the edge of the shoulder to confine water to inlets from which pipes lead the water down the slopes.

A priority has been established to concentrate efforts on those eroded areas where the pavement foundation might be affected due to construction in the drainage ditches. The original target date for the stabilization of all slopes has been lengthened a couple of years due to more pressing projects.

Mowing and Turf Cover

One of the problems in highway maintenance is to keep a good cover of attractive vegetation along the roadside without letting this vegetation interfere with sight distance or result in a prohibitive labor budget. Complete control of vegetation, therefore, means having the kind of growth desired where it is wanted and eliminating it elsewhere. The aim is to produce good turf cover along the Tollway and to keep out weeds that are unsightly or designated as "noxious" by State law.

It may be desirable to eliminate vegetation entirely in places where this can be done without risk of erosion, such as around sign posts, culvert headwalls, delineators and guardrail. Vegetation may shield or hide these objects, and it takes many man-hours to keep them clear throughout the growing season by hand cutting. Aluminum and rubber grass guards have been used around delineator posts to eliminate slow close-up mowing. Total grassed area is about 4,500 acres. On the average the median will be mowed 4 or 5 times annually and the remainder of the right-of-way from fence to fence once only.

At the present time, mowing equipment includes 16 large tractors equipped with hammerknife mowers that will mow a 12-ft swath; 5 large tractors mounted on high flotation tires to mow slopes; 32 self-propelled Gravely rotary mowers, 8 of which are equipped with sickle bar attachments (These mowers are used to mow under and behind guardrail, along the shoulder between delineator posts and those areas inaccessible to our tractor driven mowers.); and 16 hand-propelled 18-in. hammerknife mowers to mow around the delineator posts of which there are approximately 20,000. Five pull-type reel gang mower units are used to mow the median and interchange areas. This mower was not practical to use for the first year or two of operation due to surface irregularities that usually disappear after the vegetation becomes more dense.

One large tractor pulling a 7-gang unit reel-type mower will mow 6 to 8 mi of 40-ft median in 1 day. A Gravely mower or small hammerknife mower on an average day will cut vegetation behind and under guardrail for a distance of approximately 1 mi and these units will mow around approximately 180 delineators in an 8-hr day.

Assuming that there are 26 delineators per mile on one edge yielding a total of 104 per mi for a divided dual-lane highway, the cost of mowing around delineators is estimated as follows:

1. Mowing with Gravely mower—$0.10 each or $10.40 per mile for each separate mowing;
2. Spraying a soil sterilant—$0.15 each or $15.60 per mile;
3. Rubber matting "grass guard"—$2.60 each with a life expectancy of 8 yr or total cost of $33.80 yearly per mi; and
4. Aluminum "grass guard"—$0.30 each with a life expectancy of 3 yr or total cost of $10.40 yearly per mi.

The use of a soil sterilant eliminates vegetation but leaves a ragged pattern around each post. Spraying would have to be done annually. Mowing by machine leaves a neat appearance and the 30-in. strip adjacent to the shoulder is also cut which is not reflected in the preceding cost.

Automotive and Roadway Equipment Maintenance

There are 21 men engaged in automotive and roadway equipment maintenance. These include the master mechanic, clerk and automotive parts clerk, 15 skilled mechanics, car washer, lubrication man and a janitor. Each of the eight section buildings has the services of a skilled mechanic with the rest of the personnel stationed at the central shop under the jurisdiction of the master mechanic. A recent addition to this section has been a roadway equipment supervisor.

The maintenance and operation of the many different pieces of automotive and roadway equipment is under the jurisdiction of the Maintenance Division. Having this operation within the Maintenance Division eliminates much duplication, especially in the field, as the roadway section clerk also keeps the records for the automotive activity.

A rigorous inspection and preventive maintenance program is an essential element of equipment management. The equipment supervisor makes routine visits to each section to determine the progress of the preventive maintenance program and offers suggestions on solving mechanical problems. After these visits, the equipment supervisor reports back to the maintenance engineer with any suggestions that he may have to improve the operation. Check sheets have been prepared for the major pieces of equipment to control the program, which is supervised by the section foreman. The technical problems in the field are administered by the roadway equipment supervisor through the section foreman and the section mechanics. Minor repairs and routine inspection are done at each of the eight maintenance buildings. Major repairs are effected in the central shop, where they are supervised by the master mechanic.

EQUIPMENT

The winter roadway equipment requisites, based on that number of trucks sufficient to spread chemicals and/or abrasives on all the mainline and ramps within 1 hr, served as the pattern. The basic equipment for each section includes two 4-wheel drive trucks of 32,000 GVW capacity equipped with a 7-cu yd hopper, under body blade, and right-hand discharge plow, with one equipped also with a right-hand wing. The other two basic units are trucks with 26,000 GVW capacity equipped with a power reversible plow and a 7-cu yd hopper body. These hopper bodies are interchangeable with a dump box for summer use. The number of 22,000 GVW trucks placed in each section was predicated on the number of interchanges within that section and total 37 for the eight sections. This type of truck has a dump box with a hydraulically operated tailgate spinner-type spreader and reversible snow plow. It is a minor task to remove the spreader, install the conventional tailgate, and have this truck available to do other maintenance hauling jobs. Other equipment includes pickup and utility trucks, front end loaders, belt conveyors and compressors.

Originally it was thought that the small pickup trucks would be replaced after 4 yr use. This has been changed to 3-yr use, principally because of the rust and corrosion of the bodies. A newer method of undercoating is now being used which it is hoped will lengthen the usable life of these trucks. The larger trucks apparently will be useful for a longer period than anticipated because of a comprehensive preventative maintenance schedule adhered to from the date of purchase. The schedule of truck replacement is now as follows: pickup trucks, 4 yr; 21,000 GVW trucks, 6 yr; 26,000 GVW trucks, 8 yr; and 32,000 GVW trucks, 10 yr.

Police patrol vehicle purchases have gone through a transition from light- to medium- to heavyweight. The light- and mediumweight vehicles were being traded in after 60,000 to 70,000 mi. During the past 18 mo, purchases have been in the heavier field (122-in. wheel base). These cars have been driven over 100,000 mi each. The cost of operation per mile, taking into consideration the depreciation and trade-in value, will be no greater than for the light vehicles.

BUILDING AND EXTERIOR LIGHTING

There are 40 buildings on the Illinois Tollway: 8 maintenance buildings, 15 plaza control buildings, 15 service areas, an administration building and a central shop. In addition to the complex electronic equipment of the communications and toll collection systems housed in many of these buildings, there is a large investment in standard mechanical and electrical equipment, including power supply, heating, air conditioning, and emergency generators. The emphasis in the maintenance of such equipment is to preserve good-operating condition rather than to wait until trouble develops. A checklist of preventive measures has been prepared for the use of the building maintenance personnel.

The ten service station buildings and the five restaurant buildings located in the five service areas are maintained under agreement with the Standard Oil Company by resident maintenance men and by contract maintenance. Standard Oil furnishes the supervision. The Commission shares the cost of the resident personnel and bears the entire expense of contract maintenance and replacement parts. Presently a study is being made by the Commission to determine the feasibility of maintaining these buildings with Commission personnel.

All of the Commission buildings and roadway lighting before 1963 were under the direct supervision of the building maintenance engineer. He had, under his direction, a complement of 12 skilled men. After April 1, 1963 the over-all supervision of the buildings and the 3,900 roadway light standards remained with the building maintenance engineer, but the direct supervision of the personnel engaged in their maintenance was given to the roadway section foremen to obtain closer supervision. The number of persons engaged in this function has remained the same. During the original planning stages for the operation of the roadway, it was estimated that at least twice as many persons would be required to effect this maintenance, exclusive of the service areas.

Although the number of personnel and the amount of equipment have remained relatively constant since the first full year of operation, the amount spent for maintenance has risen appreciably. The Engineering Department budget for the past 5 yr is as follows:

1959	$1,944,000
1960	2,202,000
1961	2,550,000
1962	2,625,000
1963	2,624,000

The figures for 1962 and 1963 seem to indicate that the commitments for the maintenance of the facility will be leveling off. This is not true because within the next year or two additional funds will be required for equipment replacement, structure painting, concrete surface repairs and other commitments that have not been necessary in the early years of operation.

A Preventive Maintenance Program for Highway Maintenance Equipment

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The preventive maintenance program for equipment used in the maintenance of California Division of Highways roads is analyzed and reported. The program used has been in effect for 10 yr, and a careful analysis has been made of the many factors affecting such a program. Cost records show a remarkable savings in repairs which naturally reflect in more efficient operating time for the maintenance forces.

•THE FUNCTIONS of the Equipment Department of the California Division of Highways include preparing equipment specifications; accepting delivery and providing for eventual replacement of equipment; repairing and maintaining more than 7,800 pieces of maintenance equipment; repairing more than 2,300 sedans used mainly by supervisory personnel; preparing and administering a rental system to pay for equipment purchase, replacement, maintenance, and repair; designing and developing new equipment not available from commercial sources; providing technical training and research; and developing and fostering the preventive maintenance program to reduce operating cost of equipment. This paper presents a brief discussion of the preventive maintenance program in the California Division of Highways for all equipment other than sedans.

PREVENTIVE MAINTENANCE

As used in this paper, preventive maintenance includes those operations, practices, and activities necessary to decrease the rate of deterioration of a machine. In the Division of Highways the preventive maintenance program includes a planned program for lubrication, servicing, inspecting, adjusting, tightening, and cleaning equipment. It also includes training personnel in operating and servicing practices which tend to prevent premature wear and breakdown and reduce repair costs to that minimum level concomitant with normal wearing out of parts.

It is the policy of the Division to assign to the operator of a particular piece of equipment a large part of the responsibility for preventive maintenance work during the period when he is using the unit. He is responsible for lubricating the machine at regularly scheduled intervals, making inspections and minor adjustments, and tightening and cleaning it. Major adjustments or any repairs are made by a traveling mechanic working in the area or at an Equipment Department shop.

The policy of frequently changing operators and depending on the individual equipment operator to do the preventive maintenance work on the particular piece of equipment to which he happens to be assigned at the time when such work is due, is both an advantage and a disadvantage. The advantage accrues to the Maintenance Department. It provides necessary flexibility in assignment of duty in terms of the need for highway repair and maintenance. Highway maintenance men can be quickly shifted from one type of work to another as storms or other conditions dictate. Personnel problems and travel expenses are avoided, which would result if a certain highway maintenance man was assigned more or less permanently to a particular machine and sent with the unit wherever it was needed.

Paper sponsored by Committee on Maintenance Equipment.

It is not present practice to provide lubrication specialists with appropriate truck-mounted servicing equipment to do the preventive maintenance work, as is common with construction contractors. Highway maintenance equipment is so scattered and subject to unanticipated moves that planning a fixed servicing schedule by such specialists is difficult. Many items must be lubricated and serviced each day before start of shift. With widely distributed equipment, these items can only be taken care of by the operator then using the equipment.

The disadvantage of frequently changing operators for a particular piece of equipment accrues to the detriment of equipment maintenance. No one is specifically responsible for the continued preventive maintenance work on a particular machine. The attitude of the highway maintenance superintendent toward the need and importance of taking care of equipment is reflected by his foremen and their crews and largely determines the amount and quality of preventive maintenance work done on the equipment.

Each highway maintenance man will perform preventive maintenance work on many different types and models of equipment during his service with the Division of Highways. One man cannot remember the many details associated with these machines. Therefore, lubrication manuals and charts must be provided for guidance.

ACTIVITIES INCLUDED IN THE PREVENTIVE MAINTENANCE PROGRAM

Major activities involved in the preventive maintenance program include:

1. In-service training of equipment operators on preventive maintenance with particular emphasis on lubrication;
2. Preparation and distribution of lubrication manuals, lubrication charts, and dash-mounted servicing record stickers, and
3. In-service training of equipment operators on mechanical features of equipment operation.

Training in Lubrication

In-service training on preventive maintenance includes a 2-day training course with particular emphasis on lubrication. It is given to all operators and other personnel directly involved with operating and maintaining highway maintenance equipment. Instruction is given in a special classroom trailer towed to the shops and to various maintenance stations throughout the State.

The course is revised and presented approximately every 5 yr. This seems sufficient to train new employees, to introduce new features of equipment and its maintenance, to bring the operating force up to date, and perhaps most important, to motivate the operating agency to fuller participation in this important money-saving work.

The course is given to approximately 2, 450 employees of the Division of Highways. Instructors are associate equipment engineers with broad backgrounds in highway equipment repair and maintenance and with special training in teaching practical preventive maintenance procedures. They appreciate the value of safety practices and emphasize this very important phase of training in all contacts with trainees.

This course was initiated in June 1953 and continued for a 3-yr period. The need for such training, and the interest and widespread enthusiasm with which it was received, are shown by the impressive decrease in average annual repair cost of $107 per inventory unit (not including sedans). This amounted to a saving of more than $1, 236, 000 during the period.

Further decrease in average unit repair costs was achieved by persistent effort in providing reference aids to the operators to perform more and better preventive maintenance work and increasing the skill of operators in handling equipment with minimum damage or abuse.

No claim is made that the preventive maintenance program is the sole cause for the impressive savings. However, the major savings in average annual unit repair costs took place concurrently with operation of this program. Other factors also contributed to the savings. The contribution of the individual factors is impossible to evaluate precisely; they work together to produce the desired result. Some of the other contributing factors and other items concerning economics will be discussed later in this paper.

Manuals and Charts

Lubrication manuals and charts are provided by the Equipment Department for each major make and model of equipment, giving detailed information on the points to be lubricated and serviced, the kind of lubricants to be used and, in many cases, the specific procedure for application.

The items are grouped in progressively increasing periods of service. A simple code symbol is used to identify the items to be serviced at any one particular interval, based on hours of use or on mileage, whichever is appropriate to the particular machine. The servicing periods are chosen to coordinate conveniently with the standard 8-hr workday and 40-hr workweek used by the Division of Highways. A printed shape for each symbol rather than a color code is used to avoid the effects of color blindness, to allow a larger number of symbols, and to reduce cost of reproduction. Another simple code refers to the particular lubricant to be used in terms of the product name and number, as shown on the State specifications for lubricants and incorporated in the currently effective contract for lubricants.

Special operating notes and instructions are included to cover major mechanical features of equipment operation. Particular attention is given to safe operation and to procedures to avoid lugging, overspeeding of engines and other forms of abuse. Emphasis is placed on obtaining high production, yet keeping repair to a minimum and prolonging the useful life of equipment.

Lubrication manuals must spell out simply the information needed and must be easily and quickly understood. The average highway maintenance man is primarily concerned with getting his work done on the highway. The competent maintenance man appreciates his equipment, but is seldom interested in reading through a detailed, voluminous operator's manual to find those items needed to do the preventive maintenance work.

Each year a new contract is issued on a low-bid basis to an oil company to furnish various lubricants for State use. The varied brand names and numbers used by different oil companies for equivalent oils and greases are very confusing. To eliminate the confusion and at the same time assure adequate quality, lubricants are purchased in accordance with carefully prepared State specifications and receive permanent names and numbers assigned by the State by type.

Particular lubricants are selected to assure high-quality lubricants and to keep to a minimum the number maintained on inventory at the 274 maintenance stations and 20 equipment shops and subshops widely distributed throughout California. The latter is done to avoid unnecessary expense in providing storage and dispensing facilities and to decrease chance that the wrong lubricant might be applied. Wherever possible, multipurpose oils and grease are used.

The lubrication chart shows a line sketch of the machine with arrows leading to the lubrication and service points. Symbols show the period of service and coordinate with the symbols used in the manual and on servicing-record dash stickers. Items are coded to assure the use of the correct lubricant.

A copy of the lubrication chart is mounted in durable plastic and attached to the machine in a convenient and protected location. This is of particular value when the piece of equipment is serviced in the field. A complete set of all lubrication charts is also maintained at all highway maintenance stations and shops. The applicable chart is easily removed from the folio, used at the grease rack, and then returned.

Servicing-record dash stickers are placed in the cab where the operator can record the hours of use and the date when various items are serviced. The hourmeter reading, if available, is used as a basis for servicing; otherwise, hours of use are estimated. For equipment serviced on a mileage basis, a dash sticker is used which is coded in terms of mileage and date of last service. To supplement the record on the dash sticker, a permanent record of major periodic servicing is maintained in the equipment report book for each vehicle.

In-Service Training in Equipment Operation

Abuse of highway equipment is rarely deliberate and so can be reduced measurably by increasing mechanical knowledge and operating skill. Training in this area is being given under a program called MEFEO (Mechanical Features of Equipment Operation). This training is limited to the operation of equipment from the mechanical point of view and does not include training in the techniques of getting a highway maintenance job done, such as in forming a windrow or blading a berm. Such training is handled by the Maintenance Department

MEFEO training is given at the time a new model or a unique piece of major equipment is first put into operation. A schedule is prepared in cooperation with the appropriate equipment superintendent and the highway maintenance superintendent. Highway maintenance men who will likely operate the units are assembled in small groups at a suitable place, usually a maintenance station. Instruction is given by an associate equipment engineer familiar with the particular model and qualified to give authentic information and instruction. Each operator is usually given an opportunity to maneuver and manipulate the unit under simulated working conditions.

This method of obtaining the needed instruction has proved to be more satisfactory than that occasionally provided by a factory representative. For one thing, the shortcomings and limitations are presented, as well as the peculiar advantages. The representatives of the manufacturer, for obvious reasons, are certainly not inclined to admit or emphasize shortcomings that could be used adversely by competitors but that must be included in training for proper preventive maintenance.

The MEFEO training provides uniformly high-quality instruction coordinated with shop practices and the needs of the Maintenance Department. This activity has helped to decrease repair costs and downtime on equipment. It has proved to be particularly productive in preparing the many highway maintenance men hired for only the short snow-removal season. These men are expected to use mechanical equipment during the three working shifts per day in the heavy snow period. Many have little knowledge of such equipment, and in past years have caused much downtime and increased repairs. Now the men are given at time of hiring short, intensive training on the proper operation and use of, and preventive maintenance work on, snow-removal equipment.

Training to be effective must be a continuing activity. Having given the in-service course on lubrication and preventive maintenance to the operating forces, it would be nice to be able to check off the need and go on to new activities. However, the training must be repeated, with necessary revision. Persistent effort is needed to get unit repair costs down and to keep them down. Unit repair cost seems to be related to the interval between course presentation. Unit repair costs seem to rise about 3 yr after termination of the course. A decrease in average annual unit repair cost of $1.00 amounts to a saving of nearly $8,000. It is, therefore, economical to carry on with such training activity.

SAVINGS IN UNIT REPAIR COSTS

The following computations of savings are based on each respective year's inventory and average annual unit repair cost compared with the average annual unit repair cost for all items on inventory (excluding sedans) for FY 1952. This was the fiscal year immediately before the introduction in August 1953 of the initial phase of the preventive maintenance program. It is believed that the consequent values of savings shown are conservative because costs of labor, repair parts, and replacement component have consistently increased since that year, and computed savings have not been adjusted to include the effect of inflation.

Eleven months after the introduction of the in-service course on lubrication and preventive maintenance, the average annual unit repair cost decreased $55 for FY 1953, with a further decrease of $107 during the following year. With consideration for the units on inventory each year, this represents a saving of more than $266,000 during the first year and an additional saving of over $508,000 during the second year. The average annual repair cost per unit, starting with FY 1952 and including FY 1962, is shown in Figure 1.

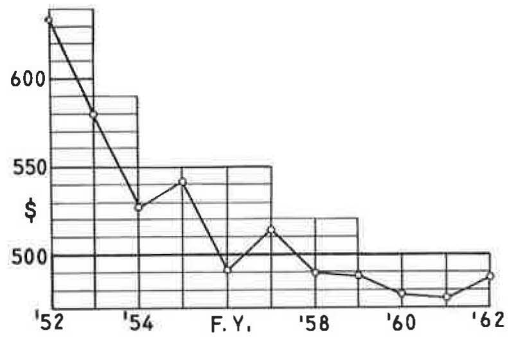

Figure 1. Average unit repair cost, all equipment except sedans.

There was an increase in unit repair cost for FY 1955. This is attributed to the unusual repairs required because of the effects of the "big flood" during the winter of that year. Many pieces of equipment were inundated and required major work to get them back into operation. In spite of an increase of approximately 2 percent, the total saving for the year was more than $461,000.

Favorable effects of the lubrication course were supplemented by introduction of lubrication manuals and lubrication charts in July 1954, the second phase of the program. These, together with the fine cooperation of the equipment superintendents, shop personnel, and especially of the Highway Maintenance Department, assisted in reducing unit repair costs $143 at the end of FY 1956. This represents a saving of more than $782,000 during the year—or a gross saving of greater than $2,018,000 during the four fiscal years following the inception of the program.

An increase in average annual unit repair cost of $23.42 for FY 1957 showed that changes in personnel and other factors influencing the effectiveness of the program were such as to justify another intensive training effort. The second presentation of the lubrication and preventive maintenance in-service course was started in April 1958. It contributed to a reduction of $24.56 in unit repair cost for FY 1958, representing a saving of more than $874,000.

MEFEO training, the third phase of the preventive maintenance program, was initiated in November 1959, and this contributed to a further reduction in average unit repair costs of $2.96 for FY 1959 and $9.65 for FY 1960. This accounts for a saving of more than $913,000 and $1,054,000 for the FY's 1959 and 1960, respectively. A further reduction in unit repair cost of $1.39 and a corresponding saving of more than $1,135,000 occurred in FY 1961.

An increase in average unit repair cost of $11.25 over the previous year took place during FY 1962. This still represents a saving of more than $1,143,000 for the year. The increase in average unit repair cost may again reflect that the major "benefit span" of the previous in-service course has been reached.

The total savings attained since adopting a strong preventive maintenance program amounts to the impressive sum of $7,822,083. This represents savings in direct repair costs only. No evaluation has been made of the savings due to decreased downtime, increased average selling price of equipment because of improved preventive maintenance, and other advantages associated with good operating condition of equipment.

The general increase in cost of labor, replacement components and repair parts leads to increased cost of repairs. The wages of the shop men have increased 49.4 percent between FY 1952 and 1962 (Fig. 2). Increase in cost of repair parts has been approximately 35 percent and of replacement components, 21 percent, during the 10-yr period.

In spite of the relatively large increase in cost of these factors, the average annual repair cost per inventory unit (not including sedans) has shown a percent reduction on a yearly basis (Fig. 2), reaching 24.93 percent in FY 1961.

When it is realized that a 1 percent decrease in unit repair cost reflects a saving of approximately $38,000 per year, the dollar value of cost-reducing activities becomes significant.

ADDITIONAL FACTORS AFFECTING REPAIR COSTS

As previously mentioned, there are many factors which affect repair costs on highway maintenance equipment in addition to preventive maintenance.

One of the nine most generally used methods for reducing excessive operating costs of equipment (1) is the minimizing of abuse of equipment by a system of discipline and merit awards. In California this can only be accomplished in a general way. The policy of frequently changing operators makes it impractical to associate an abuse with any particular person, except in flagrant cases. It also negates selection for merit awards, except in over-all improvement. The promotion of self-discipline by appealing to the sense of responsibility and providing adequate training in the mechanical features of equipment operation through MEFEO training is the strongest approach to the problem of abuse.

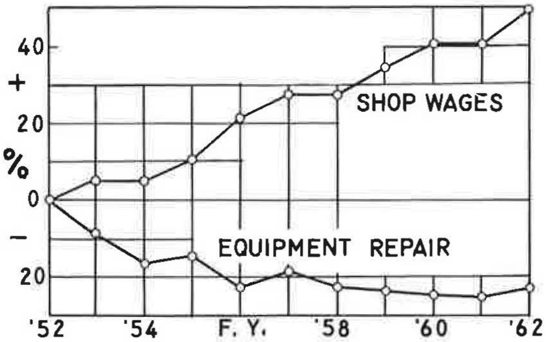

Figure 2. Change in unit costs.

There are many additional factors affecting repair costs which, in some degree, share with the preventive maintenance program in producing the impressive savings in average annual unit repair costs previously discussed. They include:

1. Improved quality of lubricants: During the 10-yr period included in this discussion, a series of State specifications were prepared which assure high-quality lubricating oil, hydraulic oil, multipurpose grease, diesel fuel, etc., for use in highway maintenance equipment. Engines are now free of sludge and there is evidence that the rate of deterioration of equipment is less ;
2. Improved shop buildings and grounds, better shop tools and repair equipment;
3. Increased knowledge and improvement in skills by the repair mechanic forces: This has been helped by in-service training courses on automotive electricity, automatic transmissions, and alternators, given at the various equipment shops and subshops. Other in-service training obtained at the Institute of Transportation and Traffic Engineering conducted by the University of California, at technical training centers, equipment service schools, and at night school courses have been of positive value;
4. Modernization and improvement in procedures and methods of handling repair parts in the stores section at shops: Mechanics spend appreciably less time waiting to obtain parts and, hence, there is less labor charged to repairs;
5. Improvements in operation of equipment by the Maintenance Department beyond the results of MEFEO training: Specific instructions from maintenance superintendents to avoid using certain equipment beyond its economic limitations have been helpful. A tendency to emphasize increase in production, even at the expense of excessive repair, has in some cases resulted in a negative effect on unit repair cost savings;
6. Improvements in equipment design and construction: There is a more general use of improvements such as torque converters in loaders and graders, better air cleaners, improved oil filters, and alternators. However, the necessary practice of purchasing equipment largely on a low-bid basis sometimes leads to equipment with high maintenance cost. Also, it should be noted that no particular improvements in equipment were made before FY 1957, yet the major reductions in annual unit repair costs took place before that year. Hence, it is difficult to establish the degree of positive influence of this factor;
7. Improvement in specifications to obtain equipment more compatible with the job required;
8. Decreased travel time for mechanics by locating resident and traveling mechanics nearer to the equipment on which they work;
9. Change in policy to eliminate regular routine periodic major overhaul given certain equipment, especially rotary snow plows: Since 1958 such equipment has been given a major overhaul only when there is definite indication that it is needed. A calculated risk is involved, but experience indicates that, in general, there has been no appreciable increase in breakdowns;

10. Decrease in the workweek: During the period studied, the introduction of the officially recognized coffee break took place, which theoretically reduces the workweek more than 2 hr. This amounts to an increase in labor cost of 5.2 percent and, hence, should increase unit repair cost. Actually, it probably has resulted in a decrease in labor costs; and

11. Change in general character of the highway maintenance fleet: Eleven years ago, the fleet consisted of about one-half construction equipment, such as graders, loaders, and tractors. Now it is made up of only about one-third construction equipment, the remainder being trucks and other equipment serviced on a mileage basis. There is little data to justify considering this change either positive or negative in its influence on average unit repair costs.

It should be remarked that the average annual unit repair cost for sedans has increased rather steadily to more than 84 percent relative to 1952 costs and reflected the effects of inflation during the 10-yr period covered in this study. The major difference between the sedan fleet and other inventory equipment, as far as maintenance is concerned, is the fact that the sedan fleet, in general, was given preventive maintenance work at commercial service stations and the other equipment was subject to the preventive maintenance program operated by the Division of Highways.

SUMMARY

An organized and continually fostered preventive maintenance program for highway maintenance equipment has proved to be a profitable activity for the California Division of Highways. The three primary phases of the preventive maintenance program are as follows:

Phase I. An in-service training course in preventive maintenance with particular emphasis on lubrication, repeated approximately each 5 yr for all personnel involved with equipment operation or repair;

Phase II. Preparation and distribution of lubrication manuals and charts for all major units of equipment; and

Phase III. Field-training classes on the mechanical features of equipment operation (MEFEO) to improve skill and knowledge of the operators and reduce repair costs from unintentional abuse.

Less than a year after the initiation of Phase I, the average annual repair cost per inventory unit of heavy equipment reduced nearly 9 percent, representing a saving of more than $266,000. Phase II was added and at the end of the second fiscal year, the annual unit repair cost reduced nearly 17 percent, representing a saving of more than $774,000 during the two fiscal years. With the introduction of Phase III, the entire program was in operation. At the end of FY 1961, the annual unit repair cost had reduced 25 percent compared to unit cost in 1952. The total saving in repair costs since the program was introduced amounts to $7,822,083, in spite of a 49 percent increase in hourly labor costs and a 35 percent increase in cost of repair parts.

It is not claimed that the preventive maintenance program is the sole cause of the impressive savings reported. However, most of the other favorable factors, such as improved repair facilities, tools and equipment, also applied to the unit repair cost for sedans which steadily increased during the 10-yr period studied in the paper. Preventive maintenance on sedans is largely accomplished by contract at private service stations and is not performed by operating personnel. This fact would seem to justify the statement that the preventive maintenance program has been an important contributing factor in saving for other highway uses more than seven million dollars since 1953.

ACKNOWLEDGMENTS

Acknowledgment is made to Earl E. Sorenson, Equipment Engineer from 1948 until his retirement in 1962, for his establishment and guidance of the strong, active, preventive maintenance program described.

Acknowledgment is also made to the staffs of the Equipment and Maintenance Departments for their cooperation in the program.

REFERENCE

1. "Manual of Uniform Highway Accounting Procedures." Amer. Assn. of State Highway Off., p. 149 (1958).

Snow and Ice Control with Chemical Mixtures And Abrasives

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•THE EFFECTIVENESS in snow and ice control of mixtures of calcium chloride and rock salt, with and without abrasives, was tested during the winter of 1958-59 by the New York State Thruway Authority, in conjunction with the Calcium Chloride Institute. Results of this 1-yr test program (1) were promising enough to justify a second 1-yr study the following winter. The second study emphasized the refinement of application to achieve the most economical use of chemical mixtures.

This paper reviews the 2-yr test program and reports on experience over the succeeding 3-yr period. An outline of winter conditions, analysis of the economic benefits to be derived from use of chemical mixtures, and general observations and recommendations for snow and ice control procedures are included.

1958-59 TEST

The original test area, the Weedsport Maintenance Section, is located in Central New York State. The Syracuse Maintenance Section to the east and Manchester Maintenance Section to the west were used as control sections. On the Weedsport section a mixture of 1 part regular flake calcium chloride (type 1) to 2 parts salt (1: 2) by volume was used either exclusively or with abrasives under all storm conditions. Straight salt alone or with abrasives was used on the control sections. Performance was observed and recorded on report forms after each storm.

By the end of the first year test program, it was clear that the use of calcium chloride in mixtures with rock salt provided faster and more effective melting action than salt alone.

1959-60 TEST

The primary modifications in the second study were (a) use of straight salt at temperatures near 30 F where salt has proven to be effective, and (b) more emphasis on the use of a chemical mixture with abrasives (salt, calcium chloride and abrasives proportioned 2 : 1 : 3 by volume). The Weedsport Maintenance Section was again used as the test area. The Manchester Section was the only control section due to equipment shifting problems in the Syracuse Section. As in 1958-59, total inches of snowfall continued well above average, and number of days with measurable snow was about the same as the previous year. Temperatures averaged about 3 F higher than during the previous winter.

This test confirmed the findings of the 1958-59 test that calcium chloride greatly speeds up the slower melting action of rock salt, particularly at temperatures of 25 F and lower. The standard 2 : 1 chemical mixture, combined with an equal quantity of abrasives, was found to be economical and effective. Premixing of chemicals and bulk storage posed no problem.

The problem of starting motors was decreased by washing equipment after storms with hot water and spraying wires and spark plugs with a silicone lubricant preservative "4X Spray" made by Dow-Corning Corporation.

Paper sponsored by Committee on Snow and Ice Control.

The use of salt in the 30 F range, and chemical mixtures with or without abrasives at lower temperatures and during ice and sleet storms, resulted in an indicated saving of $4,540 on the test section over the control section. The computation method is given in the report of results of first 1-yr test program (1, p. 5).

As shown in Table 1, actual use of calcium chloride decreased by 252 tons or 33 percent from the previous winter. Salt use showed an increase.

TABLE 1

USE OF SALT AND CALCIUM CHLORIDE FOR WINTER MAINTENANCE OF THRUWAY

Section	Chemical (tons)					
	1957-58		1958-59		1959-60	
	Salt	$CaCl_2$	Salt	$CaCl_2$	Salt	$CaCl_2$
Weedsport[a]	3,590	0	3,040	770	3,420	518
Weedsport[b]	-	-	4,340	0	4,757	0
Syracuse[a]	2,610	0	3,230	0	-	-
Manchester[a]	2,350	0	3,230	0	3,540	0

[a]Actual. [b]Theoretical.

1960-63 EXPERIENCE

Based on the excellent results during the test program, and on economic studies which indicated the possibility of an actual saving or at least no cost increase over the policy of relying on straight rock salt alone, calcium chloride-salt mixtures were incorporated into the Thruway's winter maintenance program. During 1960-61, the 559-mi Thruway began following procedures used in the Weedsport Section during the second year.

Weather Conditions

The 3-yr use of mixtures over the entire length of the Thruway System involved a greater variety of winter conditions than found in the test section. The northeast coastal influence is felt in the New York City area. The western end of the Thruway experiences Central States weather, modified somewhat by the local influence of the

TABLE 2

WEATHER CONDITIONS VS CHEMICAL USAGE ALONG THRUWAY ROUTE

Location	Dec.-March Avg. Temp. (°F)	Total Days of Snow ≥ 0.1 in.	Total Snowfall (in.)	Salt Used (tons)	$CaCl_2$ Used (tons)
		(a) 1958-59			
New York City	33.5	7	18.1	5,100	--
Albany	23.3	20	63.2	9,500	--
Syracuse	23.3	68	137.2	15,490	770
Buffalo	25.0	65	114.5	20,200	--
Total				50,290	770[a]
		(b) 1959-60			
New York City	36.3	12	33.5	5,900	--
Albany	26.7	13	60.1	10,100	--
Syracuse	27.0	72	134.9	16,030	517
Buffalo	27.5	65	115.6	21,900	--
Total				53,930	517[a]
		(c) 1960-61			
New York City	34.3	14	56.5	5,788	250
Albany	23.8	13	72.7	6,342	290
Syracuse	25.6	71	128.5	9,515	448
Buffalo	23.8	64	89.4	15,300	700
Total				36,945	1,688
		(d) 1961-62			
New York City	35.0	14	16.0	6,925	136
Albany	26.4	15	62.6	10,921	376
Syracuse	27.0	54	77.3	12,342	395
Buffalo	26.6	69	101.4	14,712	432
Total				44,800	1,339
		(e) 1962-63			
New York City	32.6	18	16.8	9,060	210
Albany	23.7	38	71.3	13,840	535
Syracuse	24.6	73	113.0	16,280	535
Buffalo	24.7	72	89.7	21,180	523
Total				60,360	1,803

[a]Weedsport only.

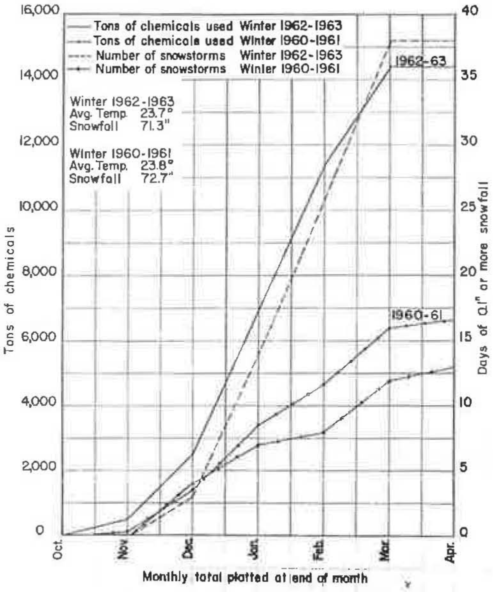

Figure 1. Chemical use vs days of snowfall.

Great Lakes. Average snowfall varies from 120 to only 31 in./yr. Table 2 compares weather conditions and chemical use on the Thruway for the 5 yr under consideration. Figure 1 shows the close correlation between total days of snowfall and tons of chemicals used during the winters of 1960-61 and 1962-63, even though total snowfall and average temperature were about the same.

Factors Influencing Calcium Chloride Use

The three winters that calcium chloride-salt mixtures have been used on the entire Thruway have been cold and dry. Maintenance crews in other than the Weedsport Section have had little chance to develop experience in the new procedures. As a result, total calcium chloride use in the 1962-63 winter was only 1,803 tons compared with 60,363 tons of salt.

Restrictions to control calcium chloride use, imposed when the mixture program was adopted, have contributed to its low use. The maintenance directive, issued in October 1960, cautioned that calcium chloride tonnage could not exceed 15 percent of salt tonnage. It also prohibited the use of calcium chloride except from December 15 through February. A final caution was that no calcium chloride be used without the consent of the section supervisor, and that none be used between the hours of 2100 and 0500. It was soon evident that these restrictions were not needed, and they are not included in the present maintenance directive. They did, however, adversely affect the total figures for use of calcium chloride and salt on the Thruway. The mixture use on the Weedsport Section (Table 3) gives an indication of the anticipated use on the entire Thruway as other personnel gain experience.

ECONOMIC BENEFITS

The normal chemical mixture, used either directly or with abrasives, contains 1 part calcium chloride to 2 parts salt by volume. This is approximately a 1:3 proportion on a weight basis due to the lighter weight per cubic foot of calcium chloride. The average cost of the normal mixture during the past winter was about $16.50 per ton or about $5.60 per ton more than the average cost of salt. The cost of the mixture when applied with an equal amount of abrasive was $9.50, or $1.40 less per ton than salt.

Savings indicated by the 1959-60 Weedsport tests compare favorably with cost studies conducted in Connecticut in 1961-62. The proportion of calcium chloride to salt used over the 5-yr period on the Weedsport Section is similar to routine use of chemical mixtures on the Ohio Turnpike.

In Connecticut, chemical mixtures and a minimum of abrasives were used at all temperatures on several test sections totaling 169 mi. Weather conditions and procedures were similar on

TABLE 3

CHEMICAL USE ON WEEDSPORT MAINTENANCE SECTION

Period	Salt (tons)	$CaCl_2$ (tons)
1958-59	3,040	770
1959-60	3,420	517
1960-61	1,222	110
1961-62	2,459	136
1962-63	3,770	223
Total	13,911	1,756
Percent	89	11

all sections. Straight salt and the normal amount of abrasives were used on control sections of 156 mi. At the end of the 1-yr test, total ice removal costs for the test sections were $131 per mile less than for the control sections. This savings reflected reduced materials costs, equipment use, man-hours and spring clean-up.

Cost studies on the Weedsport Section of the New York Thruway during the 1959-60 winter indicated an estimated saving of $71 per 2-lane mi for chemicals alone. Added savings from reduced equipment time and man-hours would undoubtedly bring the Weedsport savings to a figure at least equivalent to the Connecticut savings of $131 per mile.

STORAGE AND HANDLING PROCEDURES

Since the beginning of the test program, all calcium chloride and salt has been delivered in bulk by truck directly to the storage locations. At the Weedsport Section, the bulk calcium chloride was stored in a frame structure added to one side of the existing salt shed. A canvas rigged over the front of this shed was found to be inadequate and was later replaced with an overhead door. This provided satisfactory winter storage, but the calcium chloride did attract moisture from the air during summer months. This problem will be solved by placing a moistureproof cover tightly over the calcium chloride during the summer.

The three-sided, roofed salt storage sheds, which have been in use since the Thruway opened, have been modified by placing a divider from front to rear. Calcium chloride and/or premixed calcium chloride and salt is stored in one side with a 6-in. sand cover. Dry rock salt is stored in the other side for use in preparing mixtures or loading directly into spreaders. Minimum quantities usually stored at each maintenance section during winter months are 200 tons of rock salt and 80 tons of calcium chloride-salt mixtures.

Premixing the calcium chloride and salt is accomplished with a front-end loader or with a belt conveyor rigged with two hoppers. Calcium chloride is loaded into one hopper, salt into the other, and gates on each hopper are adjusted to obtain the desired mixture.

GENERAL OBSERVATIONS

A number of general observations have been made during and as a result of the 2-yr test program on the Weedsport Section and the addition of calcium chloride-salt mixtures to the regular winter maintenance program of the entire Thruway. The following list summarizes the benefits from and limitations to the use of calcium chloride with salt for snow and ice control:

1. Calcium chloride acts as a triggering agent and greatly speeds up the slower melting action of rock salt. Performance of chemical mixtures is superior to that of straight salt, particularly at temperatures below 30 F.
2. The use of chemical mixtures is extremely valuable during clean-up of the pavement after a storm.
3. With the faster melting action, bare pavements are obtained faster, resulting in considerable savings through prevention of accidents and slideoffs.
4. The addition of calcium chloride to rock salt reduces loss of salt due to throw off and bounce during and immediately after spreading.
5. The use of the standard chemical mixture with abrasives results in more effective use of abrasives than had been obtained with straight salt.
6. Storage of bulk calcium chloride and mixtures has not been difficult when reasonable care is exercised to protect the material from moisture.
7. Premixing of calcium chloride and salt between storms is the most practical method of obtaining a uniform mix on the road.
8. A satisfactory mixture for storm conditions, other than those controlled with straight salt, is made up of 1 part calcium chloride and 2 parts salt by volume. This is used alone or with abrasives. For hard-packed snow and heavy ice, a 1:1 mix is sometimes used.

9. Mixing of the two chemicals involves some extra handling estimated to cost \$1.25 for each ton of calcium chloride used.

10. Mixing of calcium chloride with wet salt or during rainy weather can cause severe caking. Salt with less than 2 percent moisture can be considered dry.

11. Spreaders cannot be kept loaded with a mixture in a warm garage because of caking.

12. A mixture of half chemicals and half abrasives is economical and effective but cannot be premixed.

13. No major difficulty in engine performance has been encountered since the first year of testing. All spreaders have been converted to hydraulic operation to eliminate the gasoline-powered motors. All machinery is washed after each use.

14. A reasonable proportion of calcium chloride to salt improves performance and does not increase the total cost. In fact, the use of 15 percent calcium chloride on the Weedsport Section during the 1959-60 winter resulted in a saving when compared with the control section.

15. Budget control over the more expensive calcium chloride has proved to be no more difficult than over other materials. The degree of supervision normally exercised by maintenance organizations in the application of rock salt and abrasives should be adequate for a program including calcium chloride-salt mixtures.

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Chemical Deicing of Aircraft Runways

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•THE UNITED STATES AIR FORCE must operate under all weather conditions. The Air Defense Command, in particular, must be ready to scramble high-performance jet interceptors in any kind of weather and at a moment's notice. A dry, skid-free pavement is desirable for the safe take-off and landing of these aircraft.

Snow removal and ice control on airfields is a highly organized operation in many parts of the world. Standard practice is to remove the bulk of frozen precipitation by mechanical equipment such as plows, scrapers, and rotary brooms. Basically this solves the problem for launching aircraft, some of which would otherwise have difficulty reaching take-off speed in 2 or more inches of snow. However, landing aircraft and preventing skids during high-speed taxiing is a more critical problem. To stop a high-speed aircraft within the limits of runway available, braking conditions must be such that the aircraft can decelerate quickly without ground looping or running off the pavement. Safe, quick acting deicing chemicals are needed for control of thin layers of ice and snow not removed by mechanical means or for patch ice resulting from freeze and thaw cycles. Unfortunately, such chemicals meeting USAF requirements are simply not yet available. Consequently, the Air Force has tried a variety of unorthodox mechanical methods of ice removal without success. For example an attempt has been made to break up thin layers of ice with disc harrows followed by a sweeper for ice removal. However, ice adhesion on concrete is so tenacious that reasonable quantities of ice cannot be adequately removed by this or variations of the method in the time required. Considered also have been devices for melting the ice, such as road burners and jet engine exhausts, which are also much too slow to be practical. Coatings with very low adhesion to ice have been considered for use on both aircraft and runway surfaces. This presumably would facilitate breaking up and sweeping off the ice. Fluorinated hydrocarbons and some silicone materials have resulted in reduced adhesion (though not enough) on smooth aluminum surfaces but have had essentially no effect on concrete. The relatively rough surface of concrete pavement provides a strong mechanical bond to ice in addition to the very strong chemical bond normally encountered on most surfaces. To date no chemical method of ice control equivalent in effectiveness to chloride salt deicing has appeared. Therefore, the Air Force is forced to use sodium chloride and calcium chloride in those areas where ice is prevalent and active runways must be kept open.

Although suitable for highway use, loose abrasives are not used on jet runways because of the problem of ingestion into engines. The air intake suction of jet engines is such that loose objects can be sucked right through the engine. Large objects such as nuts and bolts can cause immediate disintegration of the engine; however, even small particulate matter such as sand and cinders cause severe erosion to compressor blades and turbine buckets. To avoid the use of corrosive chemicals and still provide braking action on runways, the RCAF in Canada sprays iced runways with water, and follows immediately with a sanding vehicle. The sand is held tightly by the ice formed and provides an abrasive surface which aids in braking aircraft. The amount of water and sand dispersed must be carefully controlled and such surfaces must be swept clean of loose material as it forms to keep the ingestion and erosion problem on turbine engine components under control.

Paper sponsored by Committee on Snow and Ice Control.

Although civil engineers are concerned primarily with the effects of deicing chemicals on concrete pavements, the Air Force Materials people are much more interested in the effects of these materials on aircraft structural components. Concrete pavement deterioration is costly to repair; however, stress corrosion fractures of aircraft structures result in loss of life and loss of expensive first-line offensive and defensive weapons systems.

On low-strength materials such as the steels used in bridges and highway vehicles, corrosion is general, gradual and gives obvious evidence that maintenance is required, long before any failure occurs. In addition, quite effective preventive measures are employed in the form of protective coatings. It is common practice to undercoat vehicles, for example, and even when neglected, the corrosion problem is primarily one of annoyance and replacement cost. On the other hand, stress corrosion is extremely insidious. The USAF has experienced any number of failures from this cause, most of which showed little or no indication of corrosion on the surface before fracture of the part. Stress corrosion involves a complex interaction of sustained surface tension stresses and corrosive attack that results in rapid cracking and the premature brittle failure of a normally ductile material. The phenomenon has been observed in many metals and alloys, including both the low-alloy and stainless steels, high-strength aluminum alloys, titanium, and brass. There is apparently a direct relationship between the strength level of an alloy and its susceptibility to fracture by the stress corrosion mechanism.

The 7000 series aluminum alloys commonly used in modern aircraft are in the 65,000 psi yield strength range and above. Low-alloy steels commonly used in aircraft landing gear are all heat-treated to more than 200,000 psi and it is not uncommon to find them used in equipment at the 260,000 to 280,000 psi levels. Forgings made from materials at these high-strength levels are particularly susceptible to stress cracking because material in this form is often subjected to significant stress in the short transverse grain direction. Grain direction has a significant influence on stress cracking. An alloy is least susceptible in the longitudinal direction, more so in the transverse, and most susceptible in the short transverse. Therefore, stress cracking will occur at relatively low-stress levels if a given component is loaded in the short transverse direction and simultaneously exposed to a corrosive environment. In high-strength aluminum this stress level need be only 10 percent of its yield strength.

Components presenting the greatest difficulty are landing gear and heavy forgings such as in wing attachment fittings. Both are regularly exposed to the corrosive environment encountered on chemically treated runways during landing and take-off. Although protective coatings are often used, the erosion from particulate matter and other wear often leave these coatings chipped and damaged so that salts and other contamination can work on the base metal. Breakage of one of these components results in costly repairs, downtime and nonavailability of a weapons system, and perhaps loss of life or injury to the crew.

In view of the preceding, the following requirements for deicing compounds are desirable: (a) ability to melt ice quickly at temperatures to -40 F; (b) noncorrosive to aircraft materials; (c) no ingestion hazard to aircraft gas turbines; (d) nonlubricating on the runway; (e) minimum deteriorating effect on the pavements; (f) easy dispersal using conventional equipment; and (g) economical usage. These requirements eliminate the wholesale use of abrasives and the use of liquid deicing materials such as the glycols. Ideally, the material should be solid, free flowing, and granulated. The only material which presently meets most of these requirements is urea, but it is ineffective below 12 F.

Because the use of the ideal deicing material would be restricted to runways and taxiways, and then only for control of patch ice in most instances, it is not absolutely essential that this product compare in price with rock salt (NaCl). A higher cost material, within reason, could be justified on the basis of increased safety and reduced maintenance costs.

The Air Force Materials Laboratory has never pursued a development program for such a material; however, over the past several years a number of commercially available materials have been evaluated, including sodium chloride (NaCl), calcium chloride ($CaCl_2$), ammonium sulfate, urea, and urea-formamide-acetamide mixtures. Various proprietary corrosion inhibitors were studied; however, all were variations of the first three materials and caused severe pitting, general corrosion and stress cracking of aluminum, magnesium and steel. As a general class, the inorganic chemicals examined are all similar in this regard, differing mainly in degree. The two organic materials are relatively noncorrosive but have limited utility. Urea has a relatively high slush point (about +12 F) and the mixture is more of a paste than a free-flowing solid.

EVALUATION PROCEDURES AND DATA

As a quick and easy method of determining slush points, a saturated solution (room temperature) of each material in water is cooled in a dry-ice-in-solvent bath with constant stirring and the temperature of the solution is recorded at the point where slush forms.

A semiquantitative measurement of the ability of several materials ($CaCl_2$ as the dihydrate, NaCl, urea and the urea-formamide-acetamide mixture) to melt ice or to make it slush was also conducted.

The tests were made by measuring how far a penetrometer rod with a $\frac{3}{4}$-in. ball at the end would penetrate about 8 mm ($\frac{5}{16}$ in.) of ice. The rod was loaded to 1 lb. The materials, generally applied at a rate of $\frac{1}{4}$ psf but with some at $\frac{1}{2}$ psf, were tested until failure at 15, 0, -10, -30, and -50 F.

Summary

1. Only $CaCl_2$ was effective at all test temperatures, and was the only one of practical value below 0 F. At higher temperatures, it was more effective than the others because of quicker action.
2. NaCl is quite useful at 15 F, but heavy application is required to melt an appreciable amount of ice at 0 F. It is useless at -10 F.
3. The penetrometer sank $\frac{1}{8}$ in. into the ice at 15 F in 1 hr when the amide mixture was applied at $\frac{1}{4}$ psf, but only one-fourth as far at 0 F. Doubling the application rate produced a negligible increase at 0 F.
4. Urea was about as good as the mixed amides at 15 F, but had no effect at 0 F.
5. Under the test conditions, no slush was formed in any of the trials. The materials dissolved to whatever extent they would to form solutions, under which was hard ice.

Static Corrosion Tests

Conventional static corrosion testing on a number of metals and alloys was also conducted by immersing small coupons in a water solution of the deicing chemical, observing weight changes, discoloration, and pitting. The following results were obtained:

1. Uninhibited chlorides will etch and pit aluminum and magnesium alloys and cause general rusting of low-alloy steels.
2. Ammonium sulfate-type materials are very severe on magnesium, in some cases causing complete disintegration.
3. Some inhibited chloride solutions showed very little effect on aluminum, magnesium and steel on these unstressed specimens. This is not the case for metals under stress.
4. Urea has little or no effect on aluminum or steel. It is comparable to the blank water controls and, in fact, somewhat inhibits the general rusting of steel. Urea and other amides will cause some corrosion on copper alloys.

STRESS CORROSION EVALUATION PROCEDURES AND DATA

To establish the effect of these deicing chemicals on aircraft structural materials under stress, another series of corrosion tests were conducted. The first series of

Figure 1. Test specimen, fixture and strain gage instruments used for loading specimen.

tests were conducted using metal specimens already available. The deicers used were NaCl, $CaCl_2$ and urea.

2014 Aluminum Alloy

Procedure. —2014-T6 aluminum alloy machined specimens cut from the longitudinal direction were used. The specimen, its fixture, a jig vise used for applying stress to the specimen fixture and strain gage instruments are shown in Figure 1. Each specimen was loaded to 80 percent of yield strength, determined by averaging data from three stress-strain curves for each specimen. After loading, each specimen and fixture was completely painted with stop-off lacquer except for the gage length of the specimen. Salt solution concentrations were arbitrarily selected at saturation for 32 F.

The stress corrosion test was conducted at room temperature, which varied between 68 and 78 F. The specimens were alternately immersed in nitrogen gas-agitated solutions for 10 min, then air-dried for 20 min. The specimens were removed, washed with distilled water, and dried at the end of each day because observations were made on the day shift only. After 230 cycles, or 115 hr of testing, some of the specimens were removed from their fixtures and pulled in the tensile testing machine.

Results. —Specimens exposed to saturated NaCl solutions showed pitting after only 20 hr of testing (Table 1). Specimens in the saturated urea solution did not show any general corrosion before 100 hr of testing. Whereas general corrosion was well advanced on the NaCl specimens, there was no visible evidence of stress corrosion cracking at the time they were checked for physical properties by tensile testing. The ductility of all exposed specimens was reduced by about the same

TABLE 1

RESULTS OF STRESS CORROSION TESTS ON 2014 ALUMINUM ALLOYS

Specimen Number	Salt Sol.	Temperature (°C)	Ultimate Strength (psi)	Yield Strength (psi)	Elongation (%)	Pitting Initiated After (hr)
2014T-02	-	-	82,500	62,400	16.0	-[a]
2014T-04	-	-	84,100	65,200	-	-[a]
2014T-13	-	-	82,100	62,400	16.0	-[a]
2014T-14	-	-	85,600	64,700	16.0	-[a]
2014T-03	NaCl	21-26.5	82,200	64,600	9.3	20
2014T-05	NaCl	21.5-26	81,000	63,200	9.3	20
2014T-06	NaCl	21.5-26.5	80,900	64,300	9.3	20
2014T-07	$CaCl_2$	21-26.5	-	-	-	25
2014T-08	$CaCl_2$	22-27.5	82,900	64,900	9.3	25
2014T-09	$CaCl_2$	19-24.5	-	-	-	25
2014T-10	Urea	21.5-26.5	-	-	-	100
2014T-11	Urea	21.5-26.5	81,900	62,800	11.0	100
2014T-12	Urea	19.5-26	-	-	-	100

[a]Unexposed control specimen.

Figure 2. Specimen corroded by exposure to NaCl solution and pulled.

Figure 3. Specimen corroded by exposure to $CaCl_2$ and pulled.

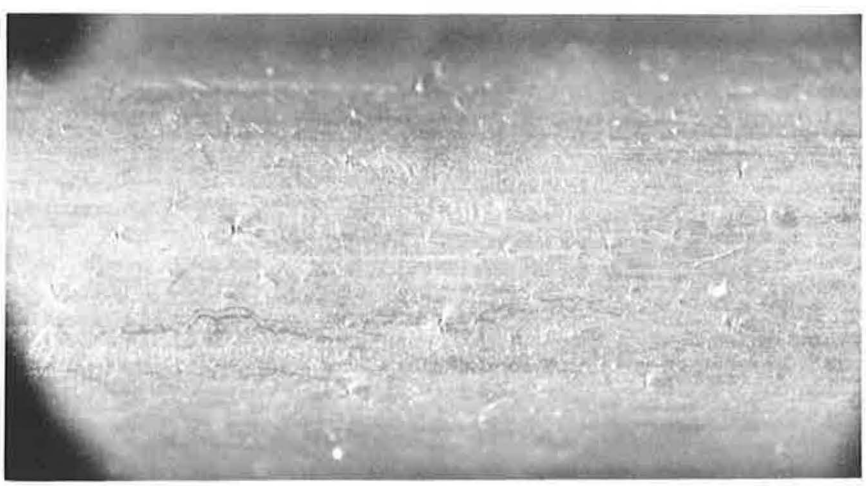

Figure 4. Specimen corroded by exposure to urea solution and pulled.

amount as indicated by the elongation data. A distinctive corrosion pattern was produced by each type of salt solution. A comparison of these surfaces shows that the specimen exposed to NaCl (Fig. 2) was appreciably more corroded than the specimens exposed to either $CaCl_2$ (Fig. 3) or urea (Fig. 4) solutions.

General corrosion was most severe with NaCl and least severe with urea. Whereas there was no visible evidence of stress corrosion cracking, all corrosion tested specimens tested for tensile strength showed increased brittleness over the unexposed specimens pulled for determining the yield strength. The lack of stress corrosion cracking was attributed to the use of specimens cut from the longitudinal direction, which is the least susceptible to this type of cracking. Because aircraft structural components are also stressed in the transverse direction, all subsequent test specimens were prepared with that in mind.

7079-T6 High Strength Aluminum Alloy

A second series of tests was conducted using specimens cut from the long transverse direction (Fig. 5) of a forged billet of 7079 aluminum in the T-6 condition (i.e., solution heat-treated and artificially aged). As in the initial test series, the aluminum test specimens were preloaded to 80 percent of their yield strength and exposed at room temperature under alternate immersion in solutions (saturated at 32 F) of NaCl, $CaCl_2$, urea and an amide mixture. The tests were conducted on a 24-hr day, 5-day week basis.

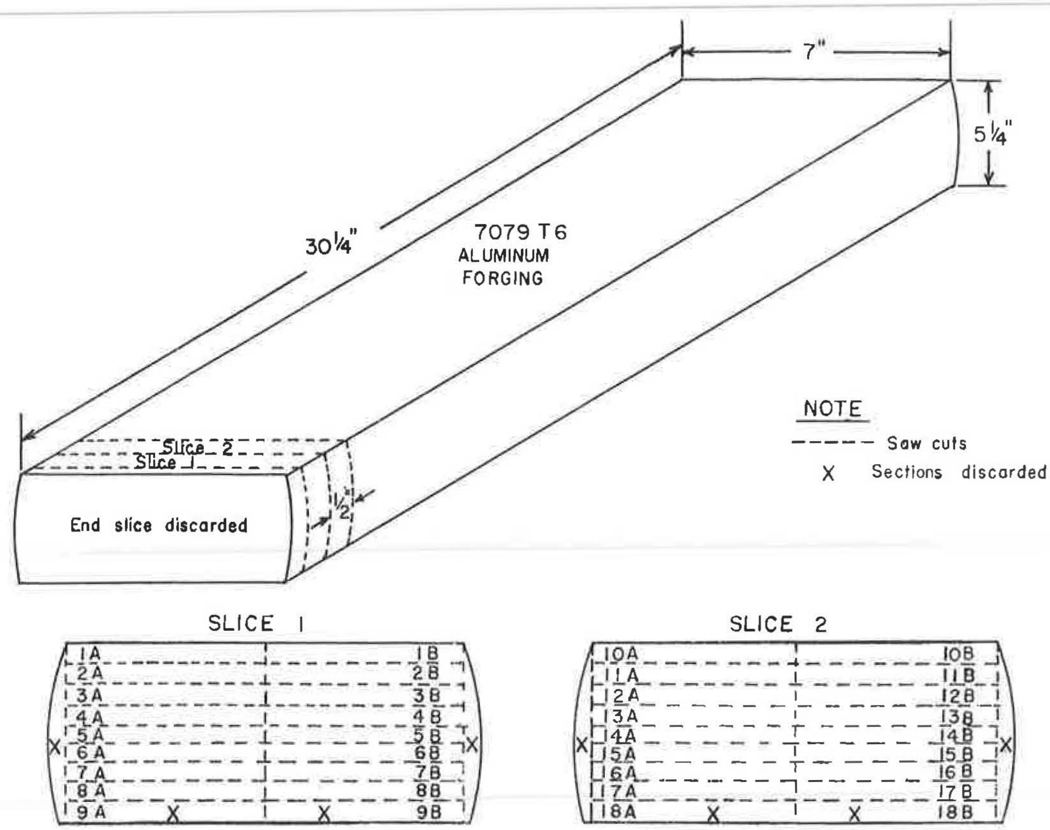

Figure 5. Location of tensile blanks cut in long transverse direction from 7079-T6 aluminum forging.

Procedure.—A 5 ¼- by 7- by 30 ¼-in. forged billet of 7079-T6 aluminum alloy was procured. The physical properties of this forging surpassed minimum requirements of AMS 4138, which specified a minimum tensile strength of 70,000 psi and a yield strength of 60,000 psi with a 6 percent elongation in the long transverse direction.

Tensile specimen blanks were cut from the long transverse direction of this forging, as shown in Figure 5. A ¾-in. slice was cut from the end of the forging and discarded. Because specimens cut from end slices often exhibit erratic effects due to unusual end grains produced during forging, the next two ½-in. slices were sawed into specimen blanks located as shown. Blanks 9 and 18 proved undersized and were discarded. All remaining specimen blanks were then machined to ASTM Standard (E8-57T) small-size specimen dimensions. The gage length sections of each specimen were carefully polished with No. 600 metallographic paper.

The machined specimens were then cleaned with acetone and two strain gages were cemented, using Armstrong Cement A-2, 180° apart on the gage lengths of each specimen. The specimens were then mounted in jigs and loaded to the proper stress level. Once the loaded specimens were stabilized, the strain gages were removed. The jig and everything except the test gage section of the specimens were coated with stop-off lacquer. Following this, the jigs were mounted in the apparatus (Fig. 6) which provided alternate immersion in the test solutions and air-drying of the specimens.

Room-temperature solutions of NaCl, $CaCl_2$, urea and the amide mixture were used as the corrosive media. Solutions were prepared by dissolving the following quantities of each ice melting material in 2 liters of distilled water: NaCl (technical grade) 700 gm; $CaCl_2$ (technical grade), 1,180 gm; urea (technical grade), 1,560 gm; and mixed

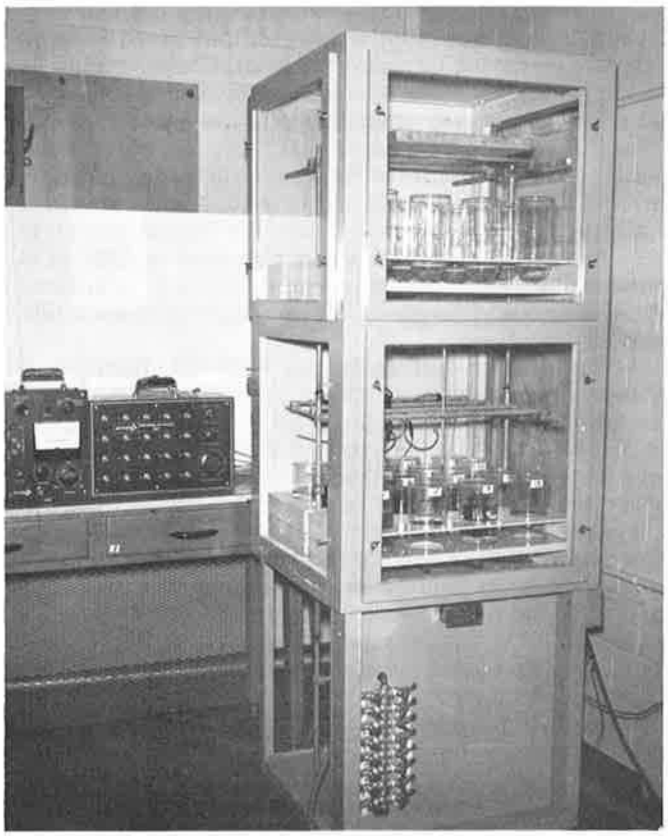

Figure 6. Apparatus for alternate immersion of specimen in ice melting solutions and suspending in air.

TABLE 2

RESULTS OF TENSILE TESTS ON 7079-T6 ALUMINUM

Specimen	Tensile Strength (psi)	Yield Strength (psi)	Elongation (%)
1A	74,700	64,400	7
4A	74,100	63,700	9.4
8B	73,700	63,900	7
11B	74,900	63,700	8
14B	73,400	63,500	7
Avg.	74,200	63,800	8

TABLE 3

TIMES-TO-FAILURE OF 7079-T6 ALUMINUM ALLOY SPECIMENS SUBJECTED TO CYCLIC STRESS CORROSION[a]

Salt Sol.	Specimen	Time-to-Failure (hr)
$CaCl_2$	13B	82
	15A	220
	5B	318
	2B	493
	Average	333
NaCl	5A	138
	13A	238
	2A	588
	Average	321
Mixed Amides	17B	536
	6A	543
	4B	938[b]
	Average (pseudo)	672[b]
Urea	14A	355
	3A	938[b]
	11A	938[b]
	12A	938[b]
	Average (pseudo)	792

[a]In saturated solutions at 32 F of ice melting salts.
[b]Test halted before specimen failure to start new series of stress corrosion tests.

amides, 1,180 gm. The quantities of ice melting material used for each 2 liters of water were based on the approximate solubility of each material at 32 F, an approximation of the condition encountered on runways. Distilled water was used for the preparation of each solution because it was felt that distilled, rather than tap, water would more closely simulate melted snow or ice found on runways. To minimize stratification of the test salt solutions, each beaker contained a fritted glass plate bottom, and a small amount of nitrogen was bubbled continuously through the solution during the test period.

The stress corrosion tests consisted of intermittent immersion (10 min) in the salt solution followed by an air-drying period of 20 min. These cycles were repeated until the specimen failed by breaking. The specimen was then washed in distilled water and subjected to visual and microscopic examination of the fractured and exposed lateral surfaces.

Results.—Table 2 gives the results of tensile tests on five specimens run for the purpose of determining the average yield strength (0.2 percent offset) of specimens cut from the two slices shown in Figure 5. Both the tensile and yield strengths of the test specimens were greater than the minimum required by AMS 4138 specification. On the basis of the average yield strength given in Table 2, a preload stress of 51,000 psi (80 percent of the average yield strength) was applied to each test specimen exposed to the ice melting solutions.

Table 3 compares the time-to-failure for the 7079-T6 aluminum alloy specimens tested in the four ice melting solutions. These results show that $CaCl_2$ and NaCl have equally deleterious effects on the life of 7079-T6 aluminum alloy. For example, the average time-to-failure for specimens exposed to $CaCl_2$ and NaCl solution was 333 and 321 hr, respectively. This is less than one-half the pseudo average life (672+ hr) of specimens exposed to solutions of the mixed amide ice melting material, and even less than the pseudo average life (792+ hr) of specimens exposed to solutions of urea. The term "pseudo average life" is used here since the life of several specimens included in the averages given in Table 3 were undetermined (no failure). Several of the specimens exposed to the latter two solutions had not failed in 938 hr, the time at which the test was halted to permit starting a new test series. The actual average time-to-failure for a specimen exposed to the urea and mixed amide ice melting materials has not yet been determined.

Figures 7 and 8 are photomicrographs (60X) representative of the 7079-T6 forging from which all specimens were cut. Figure 7 shows a view of the gross grain structure of the billet parallel to the long axis of forging. The grains, while not well defined, appear to be generally greater than 0.075 in. long. Even at this low magnification their length is greater than the field shown. Such grain size is considered typical for the alloy in a T-6 condition. Figure 8, taken perpendicular to the long axis of the 7079 forging, shows the distribution and size of the grains in Figure 7 from an end

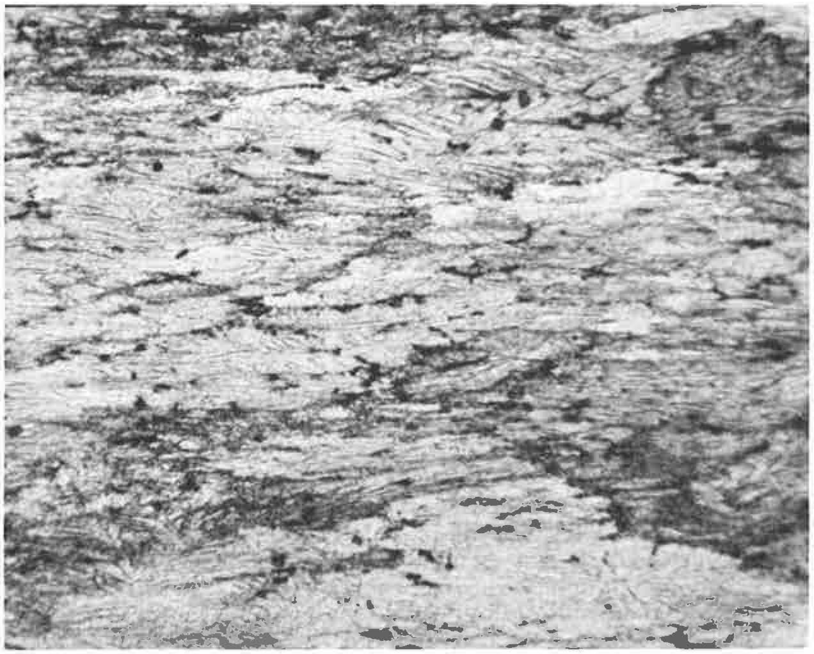

Figure 7. Photomicrograph of typical structure(parallel to long axis)of forged 7079-T6 billet (60X)—Keller's etch.

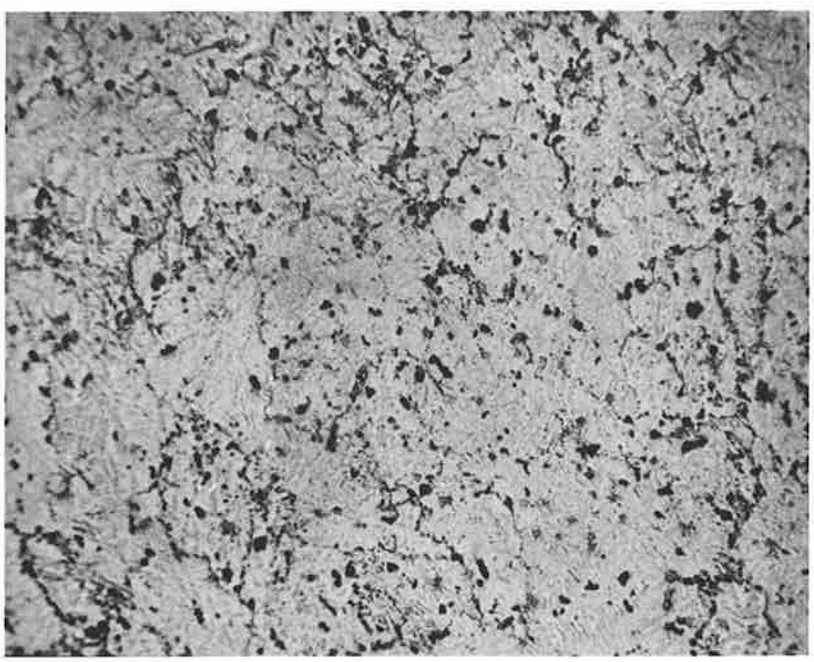

Figure 8. Photomicrograph of typical structure (perpendicular to long axis) of forged 7079-T6 billet (60X)—Keller's etch.

on view. More importantly, it shows the distribution, size and number of intermetallic constituents and inclusions found in the test specimens.

$CaCl_2$ Solution. —The surface of all specimens exposed to this solution appeared dull or matte after the first 50 hr of cyclic exposure. Extensive surface pitting was noted on the specimen that failed at 82 hr. As the life of the specimen exposed to this solution increased, surface pitting was deeper and somewhat more extensive. Pitting seemed to take place intergranularly and also at the site of the larger inclusions. The test specimens showed no evidence of cone-cup or ductile failures. Failure was judged to result from an increase in stress resulting from a decrease in the area of the specimen as pitting depth and area of pitting attack increased. The presence of numerous inclusions in an alloy of this type offers sites for initiation of pitting-type corrosion.

Figure 9 is a photomicrograph of specimen 5B which failed after 318-hr exposure to the test solution. The large pit shown here is 0.014 in. deep by 0.008 in. wide and is located about ¼ in. from the fracture zone. There appear to be numerous such pits scattered over the test gage length. Several smaller pits may be seen in the surface of the specimen in both directions from the large pit.

NaCl Solution. —The surface of test specimens exposed to this solution also appeared dull or matte within the first 50 hr of testing. Extensive pitting over the gage length surface occurred in all cases. Such pitting appeared to increase in severity or depth of penetration with lengthening time of exposure to the solution. As with the specimens tested in $CaCl_2$ solution, all exhibited brittle-type failures.

Figure 10 is a photomicrograph at 60X of specimen 13A, which failed after cyclic exposure of 238 hr in NaCl solution. This specimen appears different from that of Figure 9, as it was etched with Keller's solution. The pit shown is about 0.005 in. deep by 0.002 in. wide and was the only one found on the particular cross-section. In general, substantially fewer pits were found in the general area of the pit shown. In this series, surface pitting was less frequent and extensive on specimens exposed to NaCl solutions than ones exposed to $CaCl_2$ solutions.

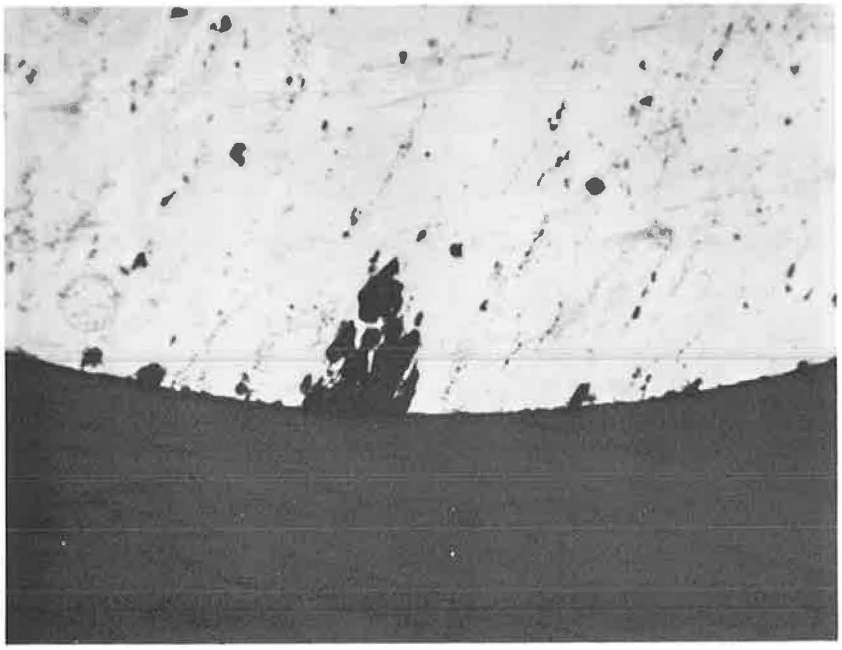

Figure 9. Photomicrograph of unetched cross-section of specimen 5B after cyclic exposure under stress to $CaCl_2$ solution for 318 hr (60X).

Figure 10. Photomicrograph of cross-section of specimen 13A after cyclic exposure under stress to NaCl solution for 238 hr (60X)—HF etch.

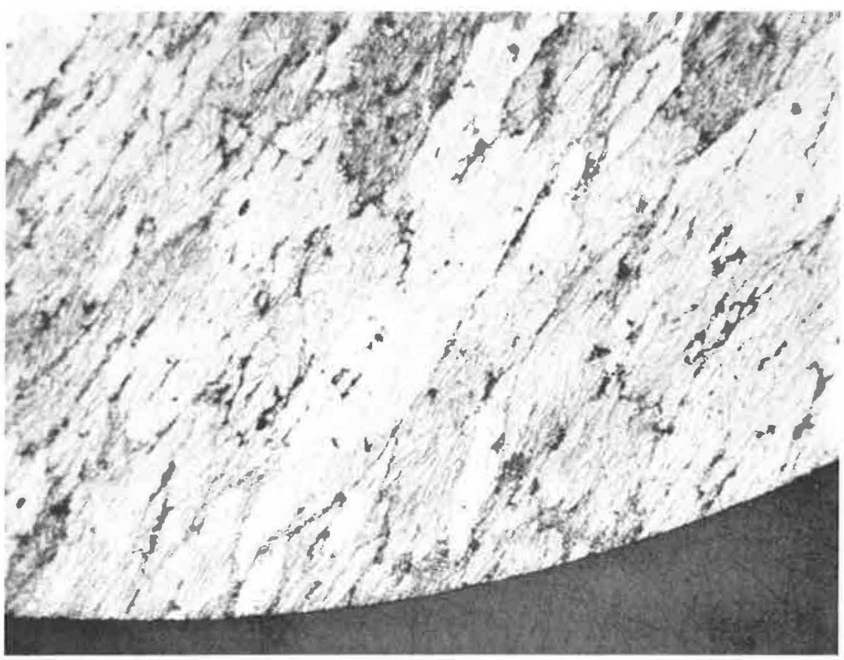

Figure 11. Photomicrograph of specimen exposed for 536-hr cyclic tests in solution of sample D ice melting product (60X)—Keller's etch.

Mixed Amide Material. —This solution showed less outward evidence of surface or corrosion attack than any of the four solutions evaluated. The surface of each test specimen exposed to this solution appeared as shiny and metallic at the end of more than 900 hr of testing as it was at the start of the test. No sign of surface pitting was found on any of the test specimens.

Figure 11, a photomicrograph of specimen 17B which failed only after 536 hr of testing, shows a complete absence of any surface pitting in the area illustrated. A complete examination of several cross-sections of this and other specimens did not reveal a single area of attack by pitting. However, the specimen did fail in a brittle manner, typical of all specimens tested in this program. The method of attack or mode of failure is not understood as of this date.

Urea Solution. —There was considerable metallic lustre on the surface of specimens exposed to urea solutions, even after 500-hr cyclic exposure to the test solution, but the surfaces of these specimens were not as metallic in lustre as those exposed to the mixed amide solution. As indicated in Figure 12, severe pitting of the specimen surface occurred with the urea solution. Whereas the pitting attack was quite widespread and rather homogenous with solutions of $CaCl_2$ and NaCl, the urea solution seemed to cause patches of more severe pitting. Deep pits in the surface were found. Despite this, three out of four of the test specimens did not fail in 938 hr of exposure. Failure of specimen 14A may have resulted when several surface pits became sufficiently aligned to reduce the specimen cross-sectional area to a point where its ultimate tensile strength was exceeded.

Discussion. —Notwithstanding the limited number of specimens exposed under stress conditions in solutions of various ice melting materials, there is little doubt that both NaCl and $CaCl_2$ are significantly more corrosive to 7079-T6 alloy than mixed amide and urea ice melting materials. The severe general (surface dulling) and localized (pitting) attack by solutions of the alkali halide ice melting salts was quite evident and appreciably greater than experienced with either the mixed amide or the urea-type ice melting materials.

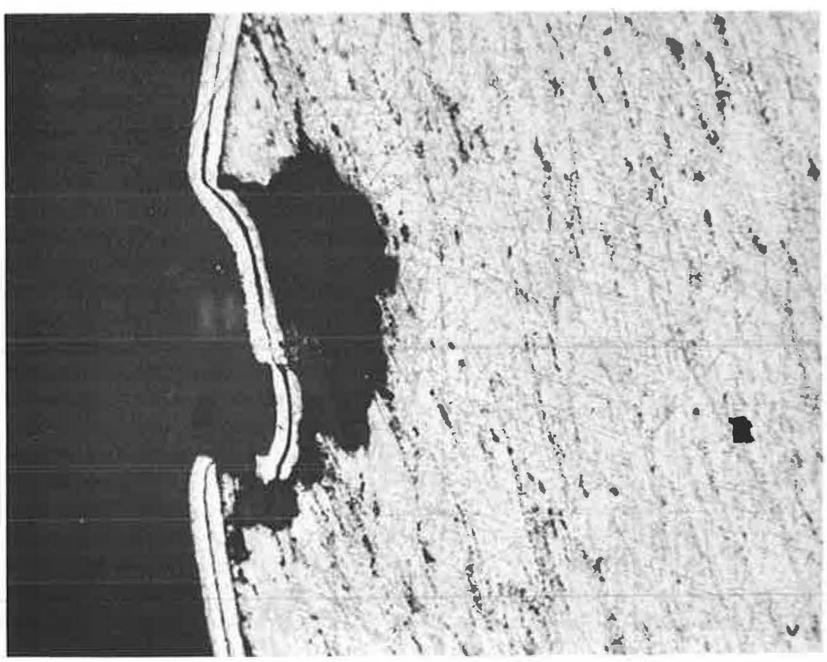

Figure 12. Photomicrograph of specimen exposed for 938+ hr cyclic tests in solution of urea ice melting compound (60X)—Keller's etch.

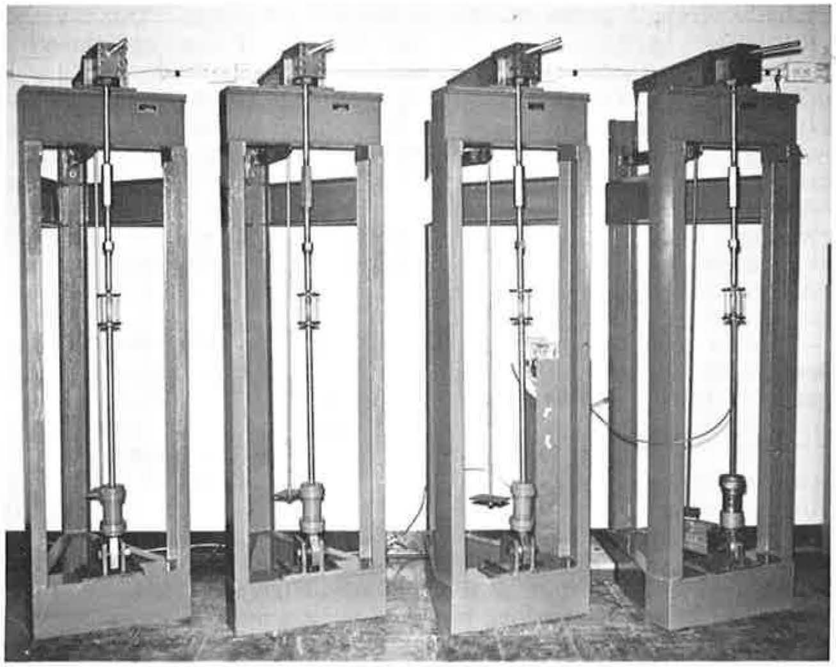

Figure 13. Stress corrosion stands for testing steel specimens.

On the basis of test results, the average stress corrosion life of 7079-T6 exposed to NaCl and $CaCl_2$ solutions was less than one-half the pseudo average life of specimens exposed to the solution of the urea ice melting material.

AISI 4340 High-Strength Steel

The next set of corrosion tests was to determine the effects of NaCl, $CaCl_2$ and urea on stressed high-strength steel parts. A program was set up to determine stress corrosion susceptibility or failure vs time-to-failure by stressing the specimens to 80 percent of yield strength.

Apparatus.—Special stands (Fig. 13) were fabricated to provide the loadings to stress specimens to the required level. Small glass containers with rubber ends contained the salt solutions around the specimen. These containers were fed from a reservoir system which alternately raised and lowered the solution according to a predetermined cycle. This cycle, repeated until the specimen failed, consisted of exposing the specimen to the salt solution for 10 min, draining the solution from around the specimen and allowing it to air-dry for 20 min. Each test specimen was under constant load for the duration of the test.

Procedure.—AISI 4340 alloy steel was selected because of its susceptibility to stress corrosion cracking when heat-treated to high-strength levels. The specimen blanks were cut from a steel forging in a direction transverse to the length of the forging. Location of the specimen within the forging was recorded and each specimen was given a number so that at any future time its location within the forging could be determined. The blanks were then rough machined into 1/4-in. diam tensile test (Fig. 14) bars and heat-

Figure 14. Modification of ASTM (E8-57T) miniature test bar.

treated to a tensile strength in the 260,000 to 280,000 psi range. This strength corresponds to a Rockwell hardness of Rc 54. The hardness of each specimen was recorded before grinding the gage length. So that the stress corrosion tests could be conducted at stress levels of approximately 80 percent of yield strength, it was necessary to determine the actual tensile and yield strength of a number of specimens. As all specimens were approximately of the same hardness, it was considered valid to take the average value of the tested specimens as the approximate yield strength for the specimens used in the stress corrosion test. Each stand was calibrated with a strain gage load cell (Fig. 15) and the weights required to obtain the necessary load for each test specimen were put in place. The load cell was then replaced by the test specimen. A record was maintained for each stand to show room-temperature fluctuation with time and the total time-to-failure.

Results.—Table 4 gives the stress corrosion data on the highly heat-treated 4340 alloy steel tested. Solutions of the following ice melting compounds were evaluated: (a) NaCl, 700 gm in 2 liters of distilled water; (b) $CaCl_2$, 1,180 gm in 2 liters of distilled water; (c) urea, 1,560 gm in 2 liters of distilled water; and (d) tap water from Dayton city water. Twelve ¼-in. diam test specimens were failed in these tests. The average time-to-failure per specimen exposed to the $CaCl_2$ solution was 4.5 hr. This compares with an average time-to-failure per specimen of 53.5 hr for test bars exposed to the NaCl solution, 237 hr for urea and 130 hr for tap water.

On the basis of visual examinations of the failed specimens, some ductility was exhibited, evidenced by "shear lips" at the fractured surfaces (Figs. 16, 17, 18).

Discussion.—As regards corrosion, test data show a definite superiority for urea ice melting compounds over NaCl and $CaCl_2$. Further, $CaCl_2$ appears to have a more severe effect on failure rate than does NaCl. It is felt that the relative severity of attack of $CaCl_2$ vs NaCl is not in itself as valid as the significant differences in failures under stress corrosion conditions between chloride compounds and urea. Both $CaCl_2$ and NaCl have a more pronounced and detrimental effect on the life of high-strength AISI 4340 alloy tested than does urea. The apparent readiness of this highly heat-treated alloy to fail when loaded above 190,000 psi and exposed to corrosive ice melting materials should be recognized. It should be noted that all of the data collected thus far have been obtained using transverse grain specimens. Specimens cut from the longitudinal forging direction would most likely exhibit greater resistance to stress corrosion cracking.

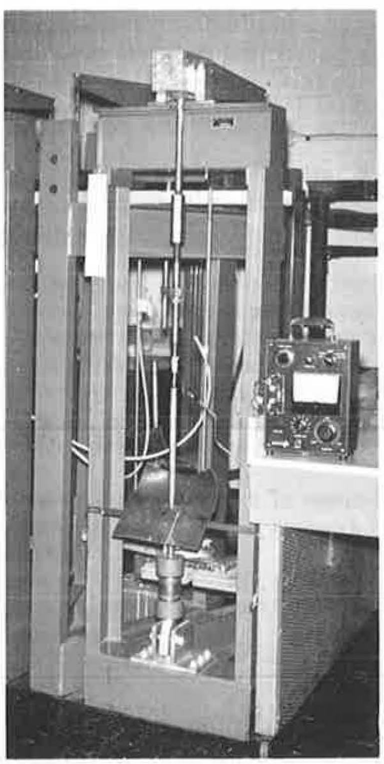

Figure 15. Strain gage load cell.

TABLE 4

STRESS CORROSION OF AISI 4340 STEEL

Corrosive Media	Specimen	Time-to-Failure (hr)
$CaCl_2$ sol.	41	6.25
	93	3.50
	11	3.75
NaCl sol.	27	67.50
	120	40.50
	89	53.50
Urea sol.	121	92.00
	101	277.5
	92	342.0
Tap water	17	102.00
	28	148.00
	126	140.00

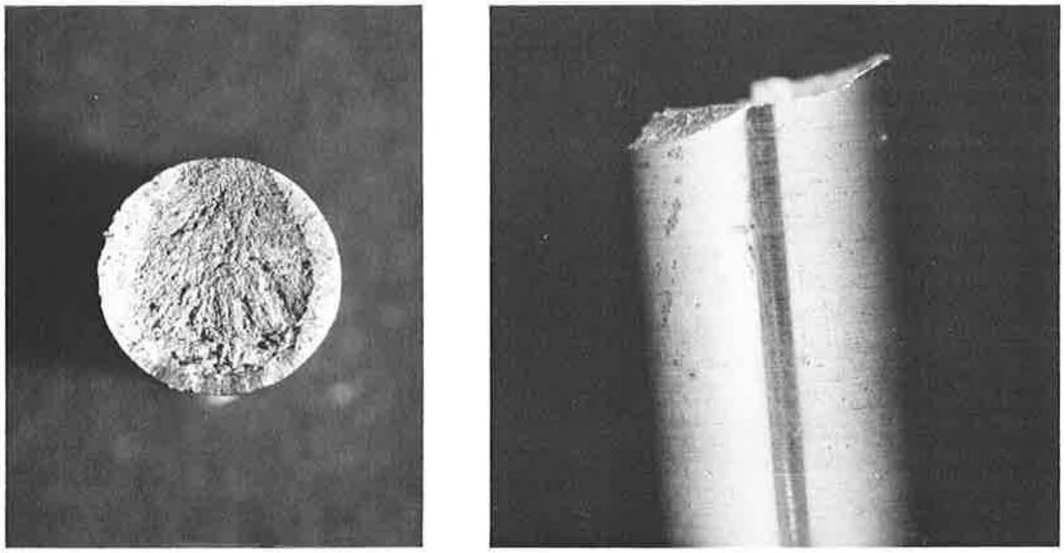

Figure 16. End-on and lateral view of fractured test bars subjected to stress in $CaCl_2$ solution, for less than 5 hr.

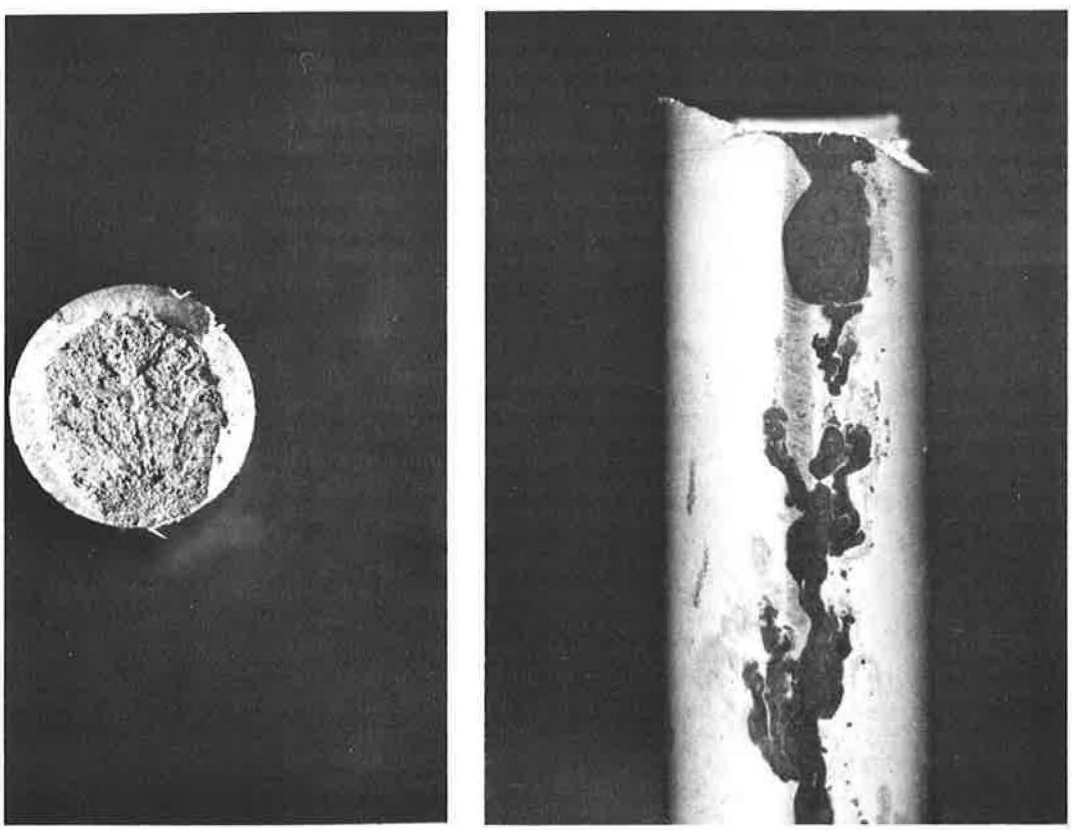

Figure 17. End-on and lateral view of fractured test bars subjected to stress in solution of Dayton tap water, exposed for over 100 hr.

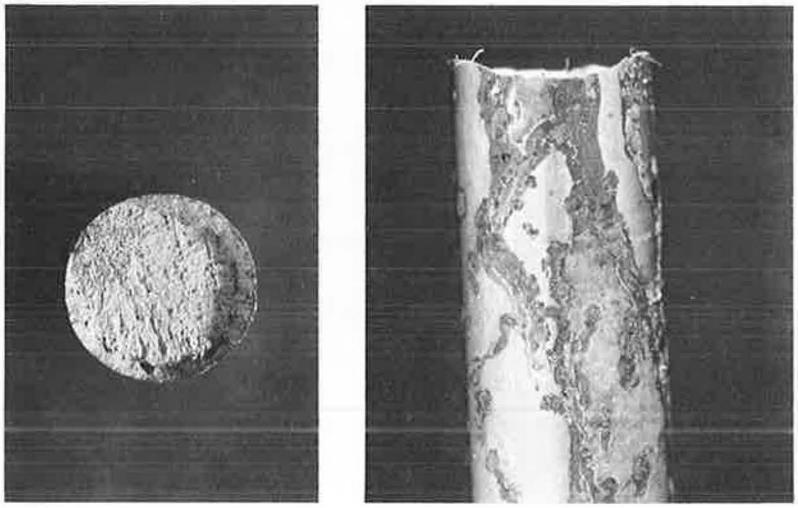

Figure 18. End-on and lateral view of fractured test bars subjected to stress in solution of urea, exposed for over 250 hr.

Since the completion of the particular work reported here, the Federal Aviation Authority has sponsored a program for the development of an effective noncorrosive deicing material for aircraft runways. The contract for this work was awarded to the Monsanto Chemical Corp. and they are presently conducting a systematic screening of candidate deicing materials based on requirements similar to those listed previously in this paper. It is still too early in the program to predict what success might be achieved; however, both the FAA and the USAF are hoping to test the promising new materials in service next winter.

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Discussion

COMMITTEE ON SNOW AND ICE CONTROL, Highway Research Board. —The Committee and the Department of Maintenance sponsored this paper because the test procedures and findings it describes warranted presentation and publication. There is no disagreement with the author's findings, but there is some concern that the report might be misinterpreted. This discussion is presented to clarify the meaning of some of the report's findings to highway organizations.

The study was limited to the behavior of highly stressed aluminum alloys and high-tensile steels. Among other things, it discusses the susceptibility of aircraft metals to stress corrosion caused by chloride compounds.

Automobile steels, of course, are not subject to the high-stress conditions found in airframe components. As a result, special corrosion problems encountered in clearing runways have no pertinence to snow and ice removal on streets and highways. The committee feels obliged to point out that highway officials should not use the report's information on airframe corrosion as the basis for modifying their use of chlorides for removing snow and ice from vehicular pavements. Those responsible for purchasing deicing compounds should also note that costs quoted for urea and other compounds are several times those given for either NaCl or $CaCl_2$.

W. P. CONRARDY, Closure.—No issue is taken with the preceding remarks. Application must be the governing factor in the selection of deicing materials. In the case of highway pavements, maximum effectiveness at lowest cost is the important criterion. The paper attempted to point out that the low-strength materials used in vehicles, bridges, etc., rarely if ever fail by catastrophic stress corrosion cracking. In addition, because weight is not a factor, protective coatings may be extensively used. Moreover, the enormous amount of deicing materials purchased for ice control on United States highways is such that even fractional increases in deicing materials costs represent an increase in operating cost that is difficult to justify.

On the other hand, aircraft are weight critical. Materials are at their maximum strength level in many cases. This makes them prone to failure by stress corrosion. This, of course, can and does result in lost lives and equipment. In addition, relatively much smaller amounts of the more expensive materials would be used, thus limiting the over-all increase in cost. This combination of factors does justify the use of the more expensive material for airport runways.

Rehabilitation of Deteriorated Bridge Slabs

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A review is presented of the methods utilized by the Central Maintenance Engineering Division of The Port of New York Authority to rehabilitate concrete bridge decks, improve riding surfaces and provide maximum protection against effects of deicing chemicals and the elements.

•THE PORT of New York Authority, a bi-State agency, was established in 1921 by New York and New Jersey. The Authority, beside other responsibilities, performs functions relating to the coordination of transportation throughout the Port District, including the planning and future development of that requiring the cooperation of the two States.

The Authority operates the following interstate vehicular crossings: George Washington Bridge, Holland Tunnel, Lincoln Tunnel and three bridges connecting Staten Island to New Jersey. These bridges are the Bayonne Bridge, famous for its arch span-type construction, and the Goethals and Outerbridge Crossing Bridges, both of truss-type design. These bridges are all more than 30 yr old and are constructed with concrete roadways varying in design, including the supporting structural steel members. Roadway maintenance procedures followed over the years have varied. Chemical deicing agents have been used for approximately 20 yr to assist in ice and snow control. Chemical attack, freeze and thaw cycles, traffic wear, and expansion and contraction factors have made numerous bridge deck repairs necessary.

Port Authority bridge deck maintenance rehabilitation projects presently under way or recently completed were planned to follow procedures utilizing the latest maintenance engineering technology. In each case a bridge roadway is studied as to past maintenance history and present structural condition, and the most feasible economical rehabilitation procedure is determined to provide necessary repairs, maximum preventive maintenance controls and a proper roadway surface. Diversification in construction techniques resulted from differences in original construction design, variation in traffic conditions, availability of roadway for construction purposes and construction costs.

Individual problems relative to bridge deck deterioration required further investigation to set the scope of each project. These problems included: (a) extent of concrete cement deterioration; (b) patterns of internal planes of concrete failure, both horizontal and diagonal; (c) extent of polishing surfaces; (d) condition of reinforcing steel; (e) condition of supporting steel members and steel forming composite designs; (f) locations and extent of joint failures; and (g) location and extent of concrete failures on underside of bridge slabs.

Two methods of replacement of deteriorated bridge deck concrete are presently specified. An epoxy patching compound consisting of a coal tar epoxy resin binder and clean sharp sand, mixed in proportion of 1 : 4, is installed as a surface patch with a limiting depth of approximately 1 in. Before placement, the perimeter of the patch is saw cut and the patch area is painted with liquid epoxy coal tar. In all other areas, portland cement concrete is specified. There are variations in the mix design of the concrete depending on requirements of bridge deck design or on construction limita-

Paper sponsored by Committee on Effect of Ice Control.

TABLE 1

CONCRETE DESIGN

Type	Wt Cement (lb)	Slump (in.)	Air (%)	Water (gal)	Water Reducing Additive (lb)	Metallic Agg. (lb)	Aggregate (lb)	
							Fine	Coarse
Shallow patch	94[a]	3	3-6	5[b]	-	-	126	227
Full slab	94[a]	3	3-6	5[b]	-	-	155	280
Non-shrink	658[c]	1	5-6	30	3.5	120	1,000	2,000

[a]Type III. [b]Gal/bag of cement. [c]Type I.

tions. When concrete must be installed where traffic conditions limit working hours, high early strength cement and accelerators are specified. Early set is also considered when designing a mix to limit vibration effects of active bridge decks or when working days are limited by scheduling requirements. Before pouring concrete, patch perimeters are saw cut and patch areas are coated with a polysulfide epoxy adhesive. Shallow concrete patches are installed unless deterioration is extensive and a full slab thickness replacement is required. The control of shrinkage in concrete where large areas of patching is encountered has required the use of additives such as iron filings, water-reducing agents, retarders and plasticizers. These mixes are similar to that used by the New Jersey Turnpike Authority. Designs of various concretes used and aggregate gradations for these concretes are given in Tables 1 and 2.

Two methods of waterproofing concrete bridge decks are presently specified. A grit tar system used most extensively consists of a prime coat of emulsified coal tar applied at the rate of 0.01 gal/sq yd, followed by an emulsified coal tar, especially formulated and mixed with sand in the proportion of 3 parts emulsion to 2 parts of sand, by volume, and spread at the rate of 0.50 gal/sq yd. The treatment is applied to either air-blown or sand-blasted surfaces. An epoxy coal tar coating mixed with sand has also been installed to a limited degree as a concrete sealer. Present lower costs and the important properties of short setting time and durability of this coating will result in its greater use in future projects.

Joint sealing is considered of primary importance in waterproofing bridge decks. Critical joints are presently sealed with a polysulfide joint sealer and established procedures require contractors to supply technically qualified supervision in working with this material. Proper mixing techniques must be closely adhered to, and temperature must be closely controlled. To obtain properly sealed joints, an evaluation must be made of the structural strength of the bonding surfaces. Polysulfide joints are normally specified to be sandblasted before installation. The actual design of the joint itself is considered important to obtain maximum life. A rubber asphalt joint filler is installed in areas not sealed with the polysulfide filler.

Bridge roadway pavements placed over a waterproofed concrete deck vary with requirements. Asphaltic concrete is used where the original construction of a bridge allows a thick pavement overlay or where it is permitted or required by curb heights and drainage patterns. Improvement in drainage, roadway profile, load distribution, control of reflection cracks, wear characteristics and skid resistance are all factors governing the use of this material.

TABLE 2

AGGREGATE GRADATION USED IN CONCRETES

Concrete Type	Passing Sieve (% by wt)													
	Fine						Coarse							
	No. 4	No. 8	No. 16	No. 30	No. 50	No. 100	1½ in.	1 in.	¾ in.	⅝ in.	½ in.	⅜ in.	No. 4	No. 8
Shallow patch:														
Min.	95	75	45	25	10	0	-	-	-	-	100	85	10	0
Max.	100	95	80	60	30	8	-	-	-	-	-	100	30	10
Full slab:														
Min.	95	75	45	25	10	0	100	90	60	-	-	15	0	-
Max.	100	95	80	60	30	8	-	100	80	-	-	40	5	-
Non-shrink:														
Min.	-	90	82	20	0	0	-	100	-	95	-	30	0	0
Max.	100	100	100	60	10	2	-	-	-	100	-	50	5	-

TABLE 3

AGGREGATE GRADATION OF STONE-FILLED SILICA SAND ASPHALT[a]

Sieve Size	Total Passing[b] (%)
3/8 in.	100
No. 4	85 - 100
No. 8	69 - 85
No. 30	40 - 59
No. 50	34 - 40
No. 100	9 - 18
No. 200	3 - 8

[a]Asphalt cement, 85 - 100 penetration.
[b]Combined aggregate including filler.

A thin overlay of stone-filled silica sand asphalt (Table 3) is recommended where drainage designs are favorable, bridge designs limit dead load, high skid resistance is desirable or a low-cost pavement is required. Due to its limited thickness, this pavement can evaporate absorbed water more completely than asphaltic concretes. It also reflects very readily deterioration in the underlying concrete surfaces, thereby warning of maintenance problems.

An epoxy coal tar type of thin resurfacer has been tested and is appropriate for areas requiring high skid resistance, heavy-duty wear characteristics (such as on high-speed curves), a quick setting material, and a waterproofer.

Monel tube drains 1½ in. in diameter are installed vertically through concrete bridge decks and flush with the top of the concrete surfaces of bridges overlaid with asphaltic concrete. These drains are located at predetermined distances along the curb line and at low points. There is a very limited use of these drains on thin overlays.

GEORGE WASHINGTON BRIDGE

Two concrete rehabilitation contracts have been completed on the upper deck of the George Washington Bridge. Work performed provided a roadway that required minimum maintenance until a scheduled major rehabilitation program was undertaken. The concrete deck of this bridge is of the bulb beam-type of design.

After the severe winter of 1960-61, a thorough inspection was made of the bridge roadway. Concrete repair work was deemed necessary but far beyond the scope of bridge maintenance personnel. A contract was prepared that specified concrete repairs to the extent that only normal maintenance would be required during the following year. Available working areas and allowable working hours were very restricted due to the extensive amount of construction then under way on the second deck and new bridge approaches.

Contract work was scheduled from 7:00 PM to 6:00 AM so that all 8 lanes of the bridge were available during peak traffic hours. A minimum of 6 hr for curing concrete before opening the roadway to traffic was specified. Approximately 3,000 sq ft of concrete patches were installed, varying in size from 2 to 100 sq ft. Depth of patch was normally 2 to 3 in.

Areas that required patching were located by visual observation of concrete deterioration and the size of each patch was determined by sounding with hammers to determine limits. Patch areas were prepared by saw cutting a 1- to 1½-in. vertical edge around the perimeter, followed by the removal of deteriorated concrete with pneumatic chipping hammers down to minimum depth located at the bottom of top reinforcing rods. Before placement of concrete, the entire patch area was thoroughly cleaned and coated with polysulfide epoxy adhesive and extended to a point 3 to 4 in. beyond the perimeter of each patch. After concrete placement, this coating was painted over the saw cut edge to seal the joint. Concrete was mixed on the job using Type III cement and a slump of 3 to 4 in. The aggregate was mixed with a 1 : 5 ratio of Sika-Set additive accelerator to water. To counteract the effects of cold weather, when necessary, the amount of Sika-Set additive was increased and the patches were covered with electric heating blankets to aid in curing. Patches were cured by applying a solution of Antisol white-pigmented curing compound after the concrete surface had lost its sheen. Patches installed under this contract have an excellent performance record over a 3-yr period with only an approximate 1 percent failure rate.

A rescheduling of the major rehabilitation program for the top deck of the Bridge made necessary a second roadway repair contract in 1962, a continuation of the work performed in 1961. This work was more extensive; the entire deck was sounded for deterioration and all temporary patches of asphaltic concrete were made permanent with portland cement concrete. The purpose of this contract was to limit required work under the major rehabilitation program and provide a roadway that would require a minimum of maintenance over the next few years.

Concrete repair work performed under this second contract totaled approximately 6,000 sq ft of patch repair. Portions of this work could be scheduled during the day shift after the second deck of the bridge was opened to traffic. A recent check of patches installed under this second contract showed the same excellent performance.

APPROACH BRIDGE—GEORGE WASHINGTON BRIDGE

A 10-yr old bridge, approximately 400 ft long, spans the New Jersey approach to the George Washington Bridge from the Palisades Interstate Parkway. Test patches were removed from the 3-in. asphaltic concrete wearing surface to permit examination of the concrete deck. Repairs were found to be required at low points along curb areas and at transverse joints. During the construction season of 1963, the asphaltic concrete wearing surface was removed, concrete deck was sounded, and epoxy coal tar compound or concrete patches were installed. Concrete mix design and patching procedures were similar to those specified on the George Washington Bridge repair work, including the use of high early strength cement and Sika-Set accelerator to speed the curing process. An air-entraining agent and a maximum slump of 3 in. were specified. All steel expansion joints were ramped to permit traffic to pass through the work site during peak traffic hours each morning.

The concrete deck was sealed with a grit tar treatment, after sandblasting the deck. Monel drains were installed and a joint of polysulfide formed or saw cut at the junction of the concrete deck and curb. Existing joints, including sidewalk and both vertical and horizontal joints of each curb, were routed, cleaned and repoured with polysulfide epoxy or rubber asphalt. An epoxy coal tar was used to level and seal an area of the concrete deck where surface depressions were uncovered.

A new wearing surface of a dense-mix asphaltic concrete was placed in two layers, utilizing rubber-tired paving equipment or padded tracks. Each layer was rolled with a three-wheel, a pneumatic-tired and a tandem roller in that order. A rubber asphalt joint filler was poured along both curbs in pre-grooved joints in the asphaltic concrete to provide added protection against water seepage.

GOETHALS BRIDGE

A major maintenance project was completed on the Goethals Bridge in 1963. The bridge roadway had been overlaid with a ½-in. silica sand asphalt thin resurfacer in 1957, but by 1963 the surface pavement started to show indications of wear and concrete deterioration. Due to expected increases in traffic on completion of the new Verranzano-Narrows Bridge, it was decided to rehabilitate the concrete deck and provide a new riding surface.

The silica sand asphalt pavement was removed by heater planers and infrared heaters. Engineers sounded the concrete deck to locate areas of concrete deterioration and repairs were made using an epoxy coal tar patching compound for thin surface patches (Fig. 1) and portland cement concrete for deep patches.

Polysulfide joint sealer was installed in a joint formed by saw cutting along the junction of the curb and existing concrete deck. All transverse joints were routed, cleaned and repoured with a joint filler of polysulfide or rubber asphalt. Curb joints were cleaned out by specially adapted grinding equipment and repoured with polysulfide joint filler (Fig. 2). The concrete deck was cleaned by sandblasting before application of a grit tar waterproofing system. A ¾-in. wearing surface of stone-filled silica sand asphalt was placed over a tack coat of asphalt cement. Rubber-tired paving equipment was required in addition to steel-wheel and pneumatic-tired rollers. All transverse joints poured with polysulfide were reflected through the new pavement by saw cutting a ¾-in. wide joint in the new pavement and were then poured flush with a rubber asphalt joint filler. This was done to control reflection cracking.

Figure 1. Goethals Bridge, epoxy coal tar patching compound installed as deck repair.

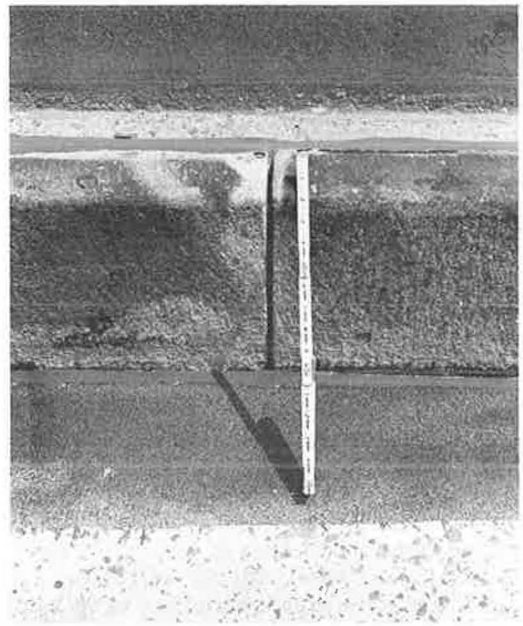

Figure 2. Goethals Bridge, newly poured polysulfide joint at curb and deck junction, polysulfide joint back of curb and cleaned vertical joint adjacent to epoxy coal tar patch.

A cant strip was formed along both curbs with the new resurfacing material to establish a drainage line approximately 4 in. out from the curb. In addition, a pre-grooved joint was formed along the junction point of curb and resurfacing and poured with asphalt joint filler. Only one-third of the bridge had to be closed for the duration of this project.

BAYONNE BRIDGE

A concrete deck rehabilitation program is presently under way on the Bayonne Bridge, which crosses from Richmond, Staten Island, to Bayonne, N. J. The concrete deck of the 1, 675-ft main span of the bridge was rehabilitated in 1962. The rehabilitation of 5, 000 ft of approach viaduct is now in progress.

The main span of the bridge deck, resurfaced in 1957 with a ½-in. overlay of silica sand asphalt pavement, was heater planed and sandblasted. This 6-yr old thin resurfacer pavement gave evidence of normal signs of wear and definite indications of deterioration in the underlying concrete deck. The exposed deck was sounded and deteriorated concrete was replaced.

All rusted exposed reinforcing steel was either wire brushed, sandblasted or replaced if damaged. Exposed steel curb against which new concrete was poured was painted with polysulfide epoxy adhesive. Water-curing techniques were followed. A polysulfide joint was constructed at the junction point of curb and concrete slab. All other joints, as well as the complete deck, were treated by the same processes as in the Goethals Bridge project.

The concrete deck of the approach viaducts to the Bayonne Bridge is presently being rehabilitated, utilizing methods similar to that on the main span. In 1948, the approaches were patched along both curbs with asphaltic concrete due to excessive spalling of the concrete surface. The approaches were then resurfaced in 1955 and 1956 with 3 in. of asphaltic concrete after raising steel expansion joints.

The present contract requires the removal of the asphaltic concrete overlay, replacement of the existing steel plate-type expansion joints with finger joints installed

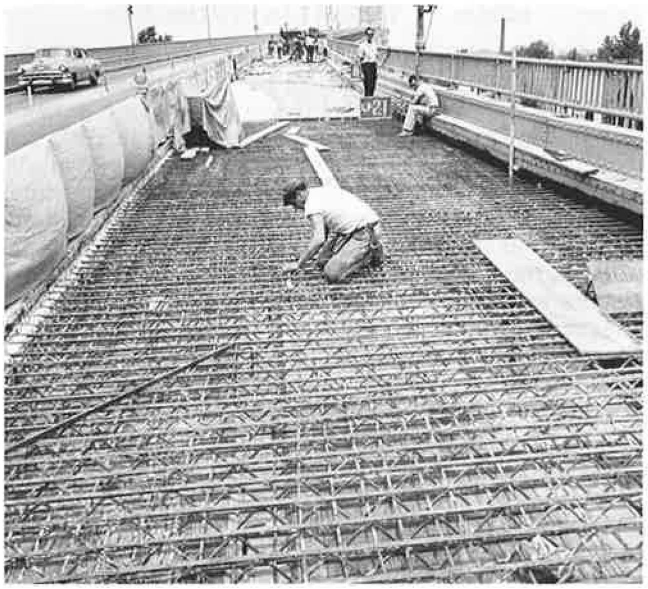

Figure 3. Bayonne Bridge, concrete rehabilitation of full slab depth.

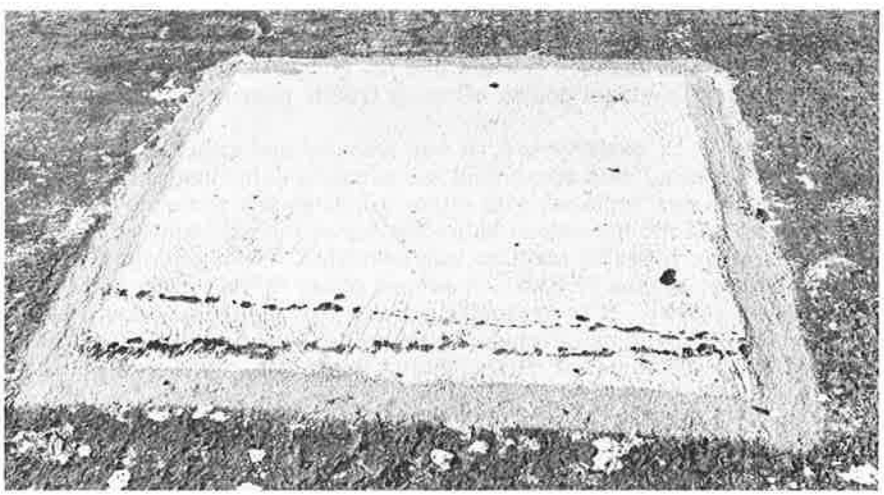

Figure 4. Bayonne Bridge, concrete patch with perimeter seal of sanded polysulfide adhesive.

to the original grade of the concrete deck, repair of the concrete deck where deteriorated, waterproofing of the deck and placing of a new thin resurfacer pavement. The concrete deck was sounded thoroughly with hammers to locate areas of deterioration. The extent of deterioration on this project was found greater than that originally contemplated, with both full slab depth replacement (Fig. 3) and area patching necessary. Due to the unexpected increase in deteriorated concrete requiring patching and the actual size of patches (Fig. 4), the mix design for these areas was changed from a high early strength cement concrete to a special non-shrink type.

LINCOLN TUNNEL APPROACHES

A concrete bridge deck rehabilitation project is also under way at the Lincoln Tunnel. The New Jersey approach to the tunnel is partially constructed on elevated structures having weight limitations that restrict the thickness of a pavement overlay.

The concrete deck of the viaducts required resurfacing in 1959 due to polishing aggregate and rough surfaces. Two overlays of a thin resurfacer of silica sand asphalt have been placed to improve the riding surface. The overlay pavements developed adherence problems, lamination failures and, on steep high-speed curves, short wear life. A research program to find an improved thin resurfacer was instituted by the Authority and resulted in the development of several new pavement designs. Six test strips of thin pavements were installed in 1962 on a high-speed curve of the Helix approach to the New Jersey entrance of the tunnel. The characteristics of the pavements are being evaluated, but of primary concern is the wearing life. These pavements include a ¾-in. strip of stone-filled silica sand asphalt alone, with an asbestos filler or with Miradon or Wyton as binder, asphaltic concrete and a thin pavement of epoxy coal tar resin with both sand and emery chip as grit. Final conclusions have not been reached as to the most serviceable because the pavements have only been under test through one very light winter's weather. There are indications that the asphaltic concrete strip has shoved slightly, the Wyton mix has worn and lost a high percentage of its surface fines, and the epoxy coal tar area has several spots where underlying deteriorated concrete has failed. The entire test strip of epoxy coal tar was not sandblasted before application of the coating, and those areas that received no surface preparation, except for heater planing of old resurfacer, have a high percentage of coating adherence failure. Further evaluation, after this winter, should provide a more realistic comparison between these test pavements. The remainder of the test strips are still in good condition.

The old silica sand pavement resurfacer was removed this past year from the roadway by heater planers and infrared heaters and the concrete deck was rehabilitated. Work was performed by contract during off-peak traffic hours, primarily during nights and weekends.

After the concrete deck was exposed, it was sounded and areas of deterioration up to 1 in. deep were replaced with epoxy coal tar patching compound. Deterioration exceeding this limitation was replaced with either a non-shrink concrete mix or, if areas were limited in size and did not extend below the top of the top reinforcing bars, a concrete mix with an accelerator additive was specified. This project was completed during the construction season of 1963. A second phase to the repair work for the Helix is scheduled for 1964. It is presently planned that bridge slabs will be coated with a waterproofing resurfacer pavement of epoxy coal tar and sand. Concrete slabs constructed on fill will be surfaced with asphaltic concrete.

CONCLUSION

This summarizes some of the projects undertaken by The Port of New York Authority to protect and extend the life of its concrete bridge decks. Procedures followed under these projects will no doubt be improved on future projects.

Thin Bonded Concrete Repairs on the Ohio Turnpike

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•THIN BONDED concrete patches are used on the Ohio Turnpike to repair damage to bridge deck and roadway pavement largely resulting from scaling and/or spalling due to freezing and thawing.

The severity of such scaling or spalling may be related directly to snow and ice removal methods, concrete quality control during construction, amount of entrained air and adequacy of cover over steel. Highway signs warning that "Bridge Deck Freezes Before Roadway" are fairly common and indicate the destructive force of additional freeze and thaw cycles to which bridge deck concrete is subjected.

Bridges on the Ohio Turnpike are no exception to those of other systems. Some difficulty has been experienced on 99 of 314 structures. The Ohio Turnpike consists of 241 mi of four-lane roadway built in 62 contract sections by 27 different contractors.

Thin bonded concrete repairs, as compared with full-depth repairs, are particularly suited to turnpike maintenance operations. On heavily traveled highways, all lanes must be available to traffic as much of the time as possible. Thin bonded repair work with high early strength concrete makes possible the reopening of sections within 6 to 24 hr. Much of the work can be done at night during low-peak traffic hours. Because structural strength is already available in the existing structure or pavement, the repaired work can be opened as soon as it will not erode. It does not have to develop significant flexural strength.

This type of repair is less costly than removal and replacement of pavement, especially on bridges. The method is simple and straightforward. Maintenance crews, once properly trained, can do quality work economically without constant supervision.

Economic considerations resulted in a somewhat unusual type of thin bonded concrete overlay on the Ohio Turnpike to correct the uneven riding surface of twin bridge decks carrying east- and westbound traffic over Raccoon Creek. Both structures are of three-span continuous steel beam construction. Because the rolled-in camber in the beams was not taken into consideration when the concrete decks were placed, the finished grade line was not acceptable. The remedy appeared to be reconstruction of the decks. However, after much consultation, it was decided to correct the grade line by placing a thin bonded concrete overlay on the completed decks. The overlay varied in thickness from $3\frac{1}{2}$ to $6\frac{1}{2}$ in. This work was done in May 1955, and there was in 1963 no evidence of failure in the decks of either bridge.

Scaling, spalling and popouts are among the principal types of damage encountered in concrete surfaces. Scaling affects the finished surface and results in a rough ride, as well as being unsightly. The use of chemicals in snow and ice control increases the severity of damage when the concrete has insufficient air content. Some areas on a bridge deck may also show surface scaling because of improper concrete finishing techniques. Plastic shrinkage cracking may be caused by surface drying before the application of curing compound or other curing methods.

A common type of surface spalling on concrete bridge decks is preceded by hairline cracks above reinforcing bars. Rusting develops an expanding layer around the metal with the result that otherwise good quality concrete is spalled away in a line above the

Paper sponsored by Committee on Maintenance of Portland Cement Concrete Pavements.

bar. A design criterion calling for more cover over reinforcing bars would lessen this type of spalling, as would closer attention to the placing of steel during construction. Popouts are attributable to the presence of unsound particles of coarse aggregate in the concrete. The Ohio Turnpike has experienced popouts in a 3-mi section of pavement but not on bridge decks. Normally, the testing of materials will prevent or minimize this type of difficulty.

Heretofore, a rather general practice for repairing scaling, spalls, or other surface failures in concrete pavement has been to use bituminous materials. Although such repairs are quick and easy, they are usually of a temporary nature and sometimes only hide the problem. Moreover, bituminous surfaces appear to absorb and hold ice control chemicals in contact with the concrete surface over a greater length of time, thus accelerating the destructive force already at work. The major reason for using concrete in the repair of concrete surfaces is that the material is compatible and, therefore, more durable. Color likenesses are retained and costs are reasonable.

With respect to bridge decks, it is important that repairs definitely arrest further deterioration. Cumulative layers of surface thickness are undesirable because of increased dead load. Due to the heavily reinforced design of concrete bridge decks and the forming required, the cost of construction is high compared to that of pavements. Complete replacement of a deck is an extremely expensive procedure, and also creates untenable traffic problems. Thin bonded concrete repairs are effective and overcome these problems.

The first experience of the Ohio Turnpike with thin bonded concrete repairs was in 1959 under contract. This work was performed on a bridge deck and on some areas of deeply scaled roadway pavement within an interchange. Methods used were those recommended by the Portland Cement Association. Most of the original repairs performed satisfactorily and the procedure is still considered basically sound. In 1960 additional work on a bridge deck was done by Ohio Turnpike maintenance forces.

In 1961 it was decided to train a crew in each of the eight turnpike maintenance sections to perform repair work in concrete. Through cooperation with Portland Cement Association engineers, workmen were given careful preparatory training which helped correct misconceptions they might have had about concrete, such as the notion that new concrete cannot be made to bond to old. PCA field representatives served as consultants when the crews did their first patching jobs. Discouragement that otherwise might have resulted from early mistakes was avoided and good morale was maintained. Every 2 yr, the repair techniques are reviewed.

METHOD OF REPAIR

The success or failure of a patch depends on the soundness of the underlying material to which the patch will be bonded. A concrete patch or a thin concrete overlay should not be attempted on any concrete slab that appears to be deteriorating throughout. Damage caused by minor scaling, popping, spalling, or mechanical, fire or chemical damage can generally be repaired with bonded patches.

Before repairs can be accomplished, all deteriorated or defective concrete must be removed. One of the most common errors in repair procedures is the failure to remove completely all unsound concrete.

The extent of the damaged area is determined by sounding and is outlined by sawing to a depth of about 1 in. with a concrete saw. The outlined area must include all adjacent unsound material. Parallel lines are sawed 1 in. deep and about 1 ft apart in each direction within the outlined area to aid in breaking out the deteriorated concrete. This concrete is removed by a jack hammer to a depth where sound material is encountered over the entire area. Care must be taken not to damage the reinforcing steel.

If the difficulty has resulted from reinforcing steel not having enough concrete cover, such as spalling on a bridge deck, the concrete must be excavated deep enough to permit forcing of the bars to a level where at least $1\frac{1}{2}$ in. of the new patching concrete will cover all reinforcing bars. Careful use of a sledge hammer and wooden block is needed to prevent damage to the bars. It is sometimes necessary to tie the bars to lower reinforcing steel to hold them down while the concrete is being placed. In some cases, the elevation of the new concrete may be raised sufficiently to provide the necessary cover.

The excavated area is thoroughly cleaned with an air hose and by hand. The edges of the saw cut are wire brushed and all loose material is blown out. Water under pressure is directed into the areas and then blown out with air. The damp area shows up any remaining loose material, which is taken out with a hand pick. The surface is then treated with a solution of 20 percent muriatic acid. When the foaming action has stopped, the surface is washed repeatedly with clean water until all acid is removed and the surface shows neutral reaction to pH paper. All free water is then blown from the surface, which is now ready for patching.

For best bond, the concrete surface should be relatively dry at the time of patching. If the concrete has gone through an extended drying period, some moistening may be advisable, but the surface should be clear of any free water before applying the bonding agent.

Starting in 1962, a number of areas to be repaired were patched without prior acid etching. The grout bonding coat was applied directly over the previously washed and dried excavated area after it had been thoroughly cleaned. Figure 1 shows the results of a number of bond tests with different bonding techniques (1).

Before placing the new concrete, a bonding coat of grout (a cement and water mixture) about $\frac{1}{16}$ to $\frac{1}{8}$ in. thick is thoroughly broomed into the concrete surface to coat it uniformly and to remove air pockets. The grout is not spread so far ahead that it changes color by drying before the concrete is placed. In hot, dry weather the old concrete surface is dampened by light sprinkling just ahead of the grouting. However, care is taken to prevent accumulations of free water.

The cement used is Type IIIA air-entraining portland cement. The aggregate is a well-graded silica sand.

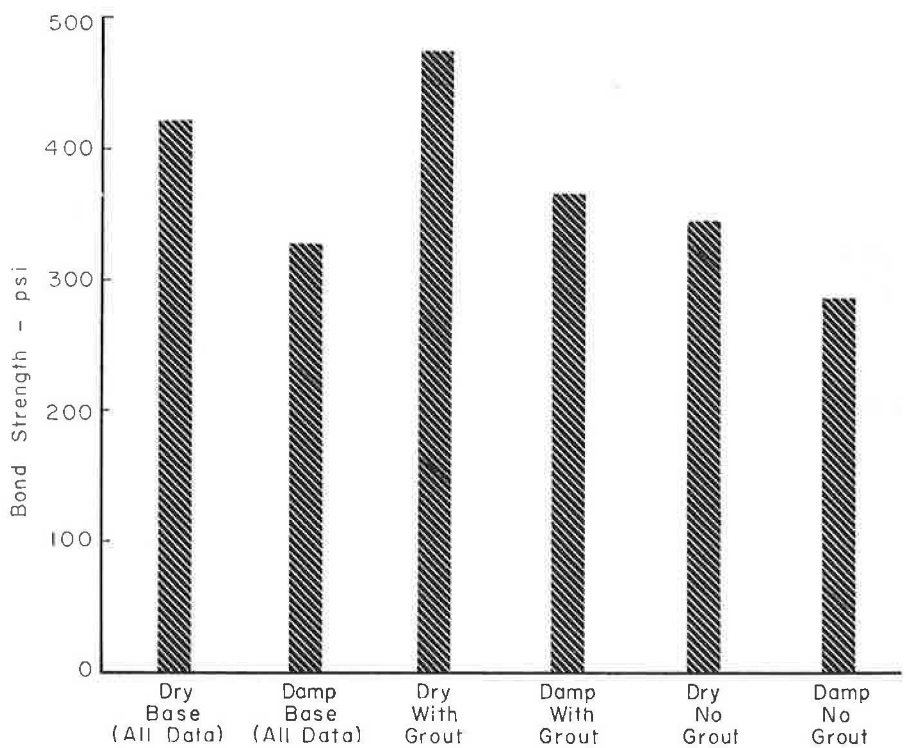

Figure 1. Effect on bond of grout and moisture condition of base.

TABLE 1

AGGREGATE GRADATION OF CONCRETE FOR PATCHING

Sieve Size	Total Passing (%)
3/8 in.	100.0
No. 4	95.4
No. 8	72.3
No. 16	49.1
No. 30	36.4
No. 50	25.7
No. 100	7.1
No. 200	0.5
Decantation	1.4

The original aggregate gradation was modified in 1963 to provide a more workable mix by the addition of more fines. Present gradation is given in Table 1. The mix proportions are 3 cu ft of aggregate to each sack of air-entraining cement, with an air-entraining admixture added during mixing. About 8 to 11 percent air in the mortar is necessary for a durable patch. The concrete materials are batched directly from the truck bed into the charging hopper of the one-sack mixer. The concrete is mixed long enough to insure thorough blending and full generation of entrained air.

The slump of the mixed concrete is such that a handful of concrete, when squeezed into a ball in the palm of the hand, remains in the shape of a ball when the palm is opened, with only slight moistening of the hand. Concrete is tested for sufficient air

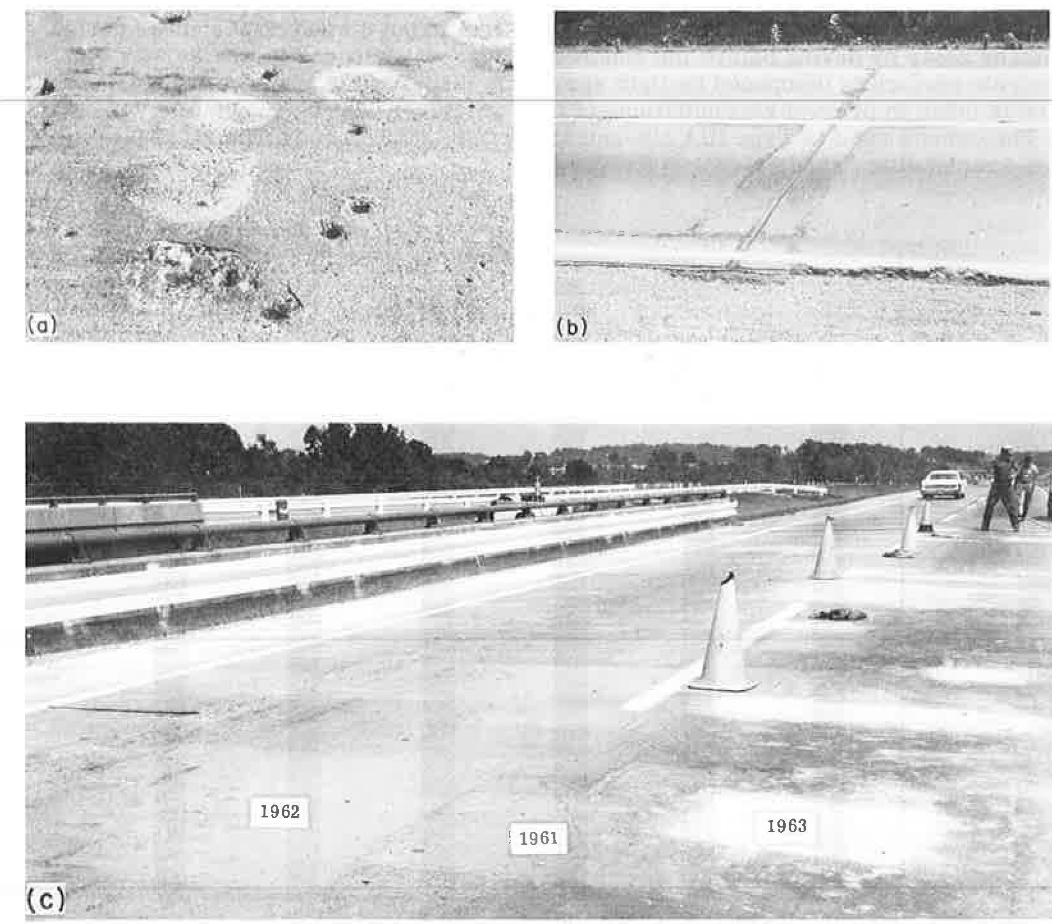

Figure 2. Various applications of thin bonded repair techniques: (a) popouts, existing and as repaired; (b) joint repair; and (c) bridge deck patches made at three different seasons.

before being placed in the patch. No admixture other than the air-entraining agent is added to the concrete.

The concrete is placed in the patch on the bonding coat as soon as possible and spread smoothly. Excess is removed with a vibratory strikeoff. The concrete after the passage of the strikeoff has a dense and firmly compacted surface. It is then darbied with wooden floats. A steel trowel need not be used in any such patching. The texture of a wood-floated patch will give good skid resistance and will match adjacent surfaces.

Immediately on completion of the finishing operations, curing is begun by spraying the patch with a white-pigmented compound. When it is not practical to apply this membrane immediately, the patched area is covered with damp burlap until it can be done.

In the repair of popouts, an air hammer is used to break out the soft material which caused the popout. This is blown out, the popout is acid etched, thoroughly washed, painted with mortar, and the patching concrete is placed. The method of repair is unique in that it is not desirable to remove more surrounding concrete materials. Therefore, a thorough cleaning-out of the area and the placement of concrete with feather edges are necessary. Because of this feather edge finish, the curing compound must be applied immediately on placement of the patch material. Very good results have been obtained in this type of popout repair. Repair work is stopped in the early afternoon, so that about 4-hr daytime curing may take place before traffic is permitted on the latest applied concrete repair material.

Various applications of thin bonded repair techniques, all of which are serving satisfactorily, are shown in Figure 2.

The experience of the Ohio Turnpike Commission in using thin bonded concrete repair methods since 1959 has been good. Enough of these repairs have been made on the Ohio Turnpike to demonstrate that the principle is sound. Very rigid controls are set up for use of this method of repair, and when followed carefully, the end results are good and the repairs are expected to be permanent.

REFERENCE

1. Felt, E. J., "Repair of Concrete Pavement." Portland Cement Assn., Devel. Dept. Bull. D44, p. 145.

Evaluation of Welded-Wire Fabric in Bituminous Concrete Resurfacing

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Respectively, Engineer of Research and Development and Research Engineer, Illinois Division of Highways

•AN EXPERIMENTAL project was undertaken in Illinois to determine the effectiveness of welded-wire fabric in the prevention or retardation of reflection cracking in bituminous concrete resurfacing on an old portland cement concrete pavement. "Reflection" cracks are those cracks that commonly appear in bituminous resurfacing immediately over cracks and joints in the portland cement concrete serving as a base.

Various methods have been employed by highway agencies to prevent or minimize reflection cracking, including the use of a granular cushion coarse, subsealing or mudjacking of the concrete slabs, extra-thick overlays, and welded-wire fabric. This study is concerned with one type of welded-wire fabric as the dependent variable in the experiment, as to its presence or absence and to its effective width, with all other factors considered to be essentially the same.

Illinois for many years has been engaged in an extensive program of rehabilitating many miles of existing portland cement concrete pavement by widening when necessary and resurfacing with bituminous concrete. Some form of rehabilitation had become necessary owing to poor riding quality and high maintenance costs. The rehabilitation program has not only made possible the restoration of the riding quality of these pavements but has extended substantially the service lives of existing highway facilities. Many more miles of existing pavement each year can be rehabilitated by widening and resurfacing than would have been possible if complete reconstruction were attempted.

Although the performance of nearly all resurfaced pavements has been satisfactory, it has been demonstrated that reflection cracking occurs after only relatively short periods of service life, particularly on routes carrying high volumes of heavy truck traffic. The reflection cracks occur not only over the cracks and joints that exist in the original pavements, but also over the longitudinal joint formed when widening strips are used to furnish a wider base for resurfacing. Reflection cracks in themselves do not appear to affect the riding quality of a pavement to any great extent, but experience has shown that these cracks progress to a stage of distress known as belt cracking (a pair or series of closely spaced parallel cracks) and ultimately to the spalling or dislodgement of material between these cracks. The advanced state of deterioration seriously affects the riding quality, reduces the potential service life of a resurfaced pavement, and intensifies the maintenance problem.

Engineering literature indicates that the installation of welded-wire fabric in a bituminous resurfacing over concrete probably was introduced in Texas in 1946. Numerous other installations of the kind have been placed since that time, using various sizes and configurations of wire-mesh reinforcing. In some instances, the reinforcing has been placed over the entire pavement surface, whereas in others, strips have been placed only over joints, cracks, and deteriorated areas. The location of the reinforcement in reference to a horizontal plane has been varied, sometimes being at the old pavement surface and sometimes being between the layers of the bituminous concrete resurfacing.

Encouraging reports from many of the earlier projects led to the establishment of the study involving the use of welded-wire fabric in an experimental construction project of bituminous concrete resurfacing in Illinois. After 5 yr under traffic, this installation is showing superior performance at locations where the fabric was used.

Paper sponsored by Committee on Salvaging Old Pavements by Resurfacing.

DESCRIPTION AND LAYOUT OF PROJECT

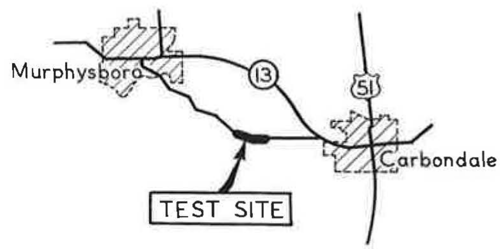

Figure 1. General site location.

This experimental installation was incorporated in the widening and resurfacing of Section 12RS-1, SBI Route 13, Jackson County, for the rehabilitation of an old portland cement concrete pavement. The project is located about 60 mi north of the southernmost tip of Illinois, between Murphysboro and Carbondale (Fig. 1). The annual precipitation at the site averages about 44 in., with little variation in the average monthly rainfall throughout the year. In the winter, the soil seldom remains frozen for any extended period, and there are relatively few freeze-thaw cycles. Normally, the average depth of frost penetration is less than 10 in.

The topography at the site is typical of the upland, rolling portions of Jackson County and the surrounding areas. The soils are light-colored and moderately slowly permeable, developed from thick to moderately thick loess. Silty loam soils of the A-6 group predominate.

The original improvement was constructed in 1925, with the then-current practice of light cut-and-fill sections. The pavement, placed directly on the silty loam soils, was constructed of plain concrete 18 ft wide and to a 9- by 6- by 9-in. cross-section. No transverse joints were used except for construction stops. Steel bars, ¾ in. in diameter, were installed at each pavement edge. A full-depth metal plate was used to form a center longitudinal joint, with the adjoining slabs being tied by deformed steel bars. The pavement underwent extensive portland cement concrete patching and maintenance repair in the 1940's. At the time of resurfacing, it had numerous wide transverse cracks accompanied with serious spalling and faulting. Intermittent areas of scaling had occurred throughout the entire length, and many of the old patches were severely raveled.

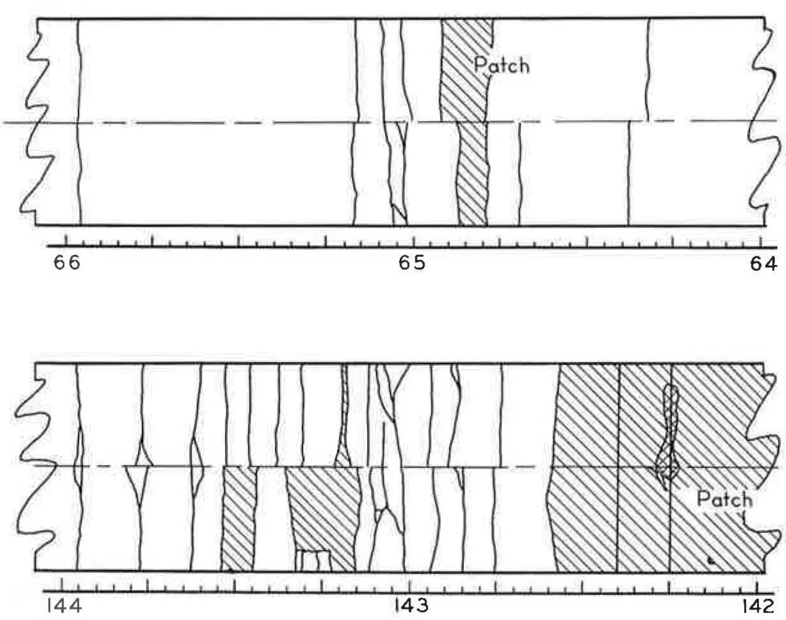

Figure 2. Typical condition of old concrete pavement.

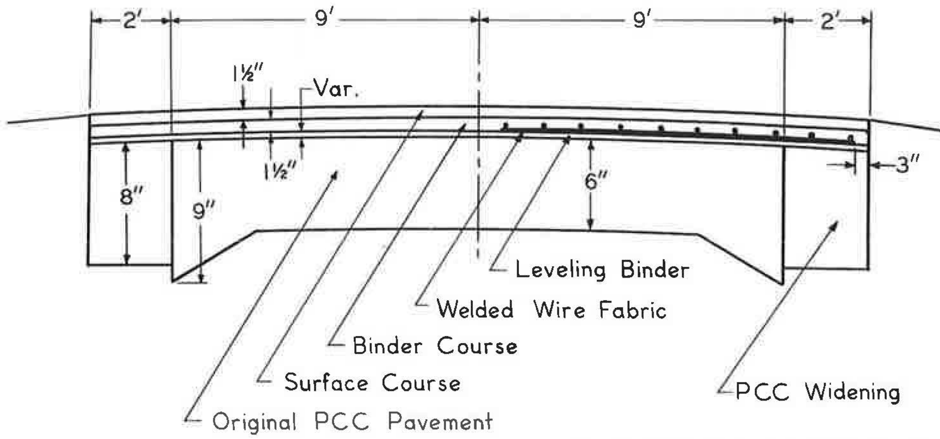

Figure 3. Typical cross-section.

A comprehensive condition survey of the existing pavement was made before resurfacing. A representative field survey sheet typical of the condition of the old concrete pavement is shown in Figure 2.

The 1958 rehabilitation consisted of widening the existing pavement to 22 ft by a 2-ft width of 8-in. thick portland cement concrete along each edge, and resurfacing the entire width with a 3-in. thickness of bituminous concrete, Illinois subclass I-11. A typical cross-section of the rehabilitated pavement structure is shown in Figure 3.

The experimental portion of the project consisted of two sections of welded-wire fabric placement and two control sections without fabric. The two sections with fabric included a 2,500-ft section of continuous placement of sheets of several different widths and an 18,000-ft section of intermittent placement in which sheets 10½ ft in width were placed in the 11-ft lanes at 19 severely distressed locations. The 2,500-ft section was subdivided into five equal lengths for placement of the different widths of fabric. In the first four subsections, the fabric was placed successively 3½, 4½, 5½, and 10½ ft wide in the westbound lane. A 10½-ft width was placed in both the east and westbound lanes in the fifth subsection. In the 18,000-ft section, all sheets were 10½ ft in width, the lengths of fabric at the individual locations varying from 6 to 120 ft.

The two sections with no fabric installation included a 3,500-ft length in which the interval between transverse cracks in the existing pavement averaged about 50 ft, and a 3,900-ft section in which this interval was about 13 ft. The latter section was selected as the control for the analysis of transverse reflection cracking because its average crack interval was the same as that in the sections containing the fabric. Details of the project layout are shown in Figure 4.

CONSTRUCTION

All construction in the experimental area preliminary to the placement of the welded-wire fabric was completed in September 1958. This work included bituminous skin patch removal, cleaning and resealing of joints and cracks, construction of many full-depth concrete patches, placement of the portland cement concrete widening, application of the asphalt tack coat, and placement of the leveling binder course.

The welded-wire fabric conformed to AASHO Designation: M55 and consisted of 10-gage longitudinal wires on 3-in. centers and 10-gage transverse wires on 6-in. centers. The fabric was cut from 15- by 10½-ft sheets on the job site as needed into 3½-, 4½-, and 5½-ft widths. The individual sheets of fabric were placed directly over the leveling binder course with the transverse wires in contact with the surface to prevent the tracks of the paving machine from snagging the fabric.

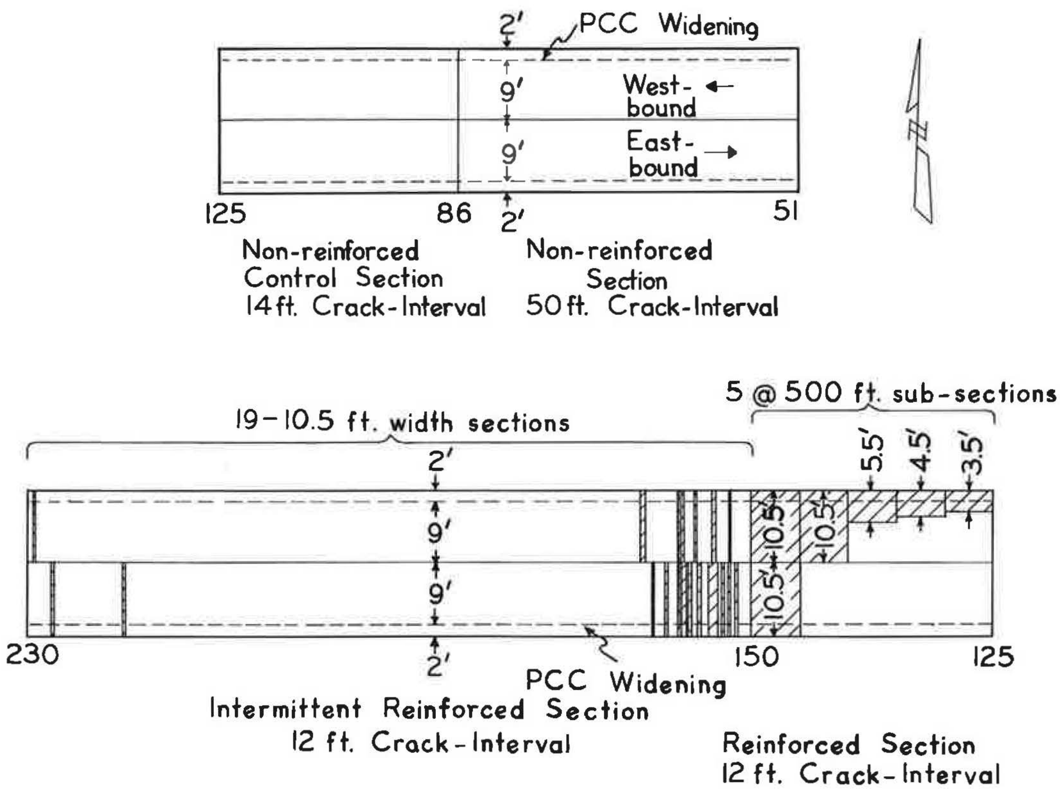

Figure 4. Details of project layout.

The bituminous concrete binder and surface course mixtures were produced in a hot-mix plant and conformed to the Illinois specification for fine dense-graded aggregate type mixtures, subclass I-11. Each mixture was produced from two sizes of a crushed stone aggregate, a coarse sand, a fine blend sand, and a mineral filler (limestone dust) in combination with a 70-85 penetration grade paving asphalt. All material of the binder-course mixture was required to pass a 1-in. sieve opening; all surface-course mixture was required to pass a ¾-in. opening. The leveling binder mixture was the same as the surface mixture.

The mix designs were established by the Marshall method and conformed to Illinois standard design criteria which set a minimum stability value of 1500 and a flow value of 8 to 16. The mix formulas and tolerance limits are given in Table 1.

A special device was developed for holding the wire fabric in place during the placement of the binder course mixture. The hold-down device, a three-runner sled of channel irons, was placed between the tracks of the bituminous paver and pulled along over the fabric by chains attached to special brackets mounted on the paver. In addition, a 9-ft runner of light-weight railroad rail was pulled along outside of each track of the paver

TABLE 1

COMPOSITION OF BITUMINOUS CONCRETE MIXTURES

Material	Binder Course Mixture		Surface Course Mixture	
	%	Tolerance (%)	%	Tolerance (%)
Passing 1-in., retained on ½-in. sieve	34.5	±5	-	-
Passing ½-in., retained on No. 10	32.3	±5	61.5	±3.0
Passing No. 10, retained on No. 200	25.0	±3	28.9	±3.0
Passing No. 200 sieve	4.2	±3	5.0	±1.5
Bitumen PA-6 (70-85 penetration)	4.0	±0.3	4.6	±0.3

to hold the edges of the fabric firmly in place. The sheets of fabric were placed a short distance ahead of the paver so that only the unloading truck needed to drive over them. The positioning of the fabric did not delay appreciably the paving operations.

Spot checks, made by temporary removal of small areas of the binder course to expose the fabric before the rolling operation, revealed that the fabric was tight against the leveling course. At some locations, the fabric was slightly embedded in the leveling course, apparently due to the heat and tamping during placement of the binder course.

Some distortion of the fabric occurred during the installation of the first few sheets of the $4\frac{1}{2}$-ft width. The inside edge of these sheets occasionally curled up between the outside and center runners of the hold-down sled. This curled edge snagged on the cross brace of the sled frame, humping the fabric and causing the spreading screw of the paver to further wrinkle it. This difficulty was eliminated by shifting these sheets inward a few additional inches beyond the normal 3 in. that other sheets were being set in from the edge of the base. This positioned the inner edge of the fabric under the center runner of the sled.

Occasionally, the sheets of fabric were skidded ahead and buckled between the wheels of the unloading truck, particularly on downgrades. This condition was remedied by applying less tension of the truck brakes while the paver pushed the truck during unloading operations. Little difficulty was encountered in placing the binder course over the isolated single pieces of fabric in the areas of intermittent placement.

TRAFFIC

The pavement involved (formerly Ill. 13) has carried a volume of traffic averaging approximately 1,700 veh/day since resurfacing, including about 15 percent commercial vehicles. The volume and character of the traffic traveling in each direction appears to be about equal.

In an effort to present the data in a manner that reveals the most evidence of actual performance, the dependent variable has been compared with length of service and the accumulation of 18-kip equivalent single-axle loads representative of the mixed traffic loadings. Equivalency factors for converting single- and tandem-axle loadings into equivalent 18-kip single-axle application were derived from the AASHO Road Test performance equations (1). The volume and composition of traffic used in this conversion are given in Table 2.

OBSERVATIONS AND MEASUREMENTS

The observations and measurements include (a) a condition survey of the existing concrete pavement made just prior to widening and resurfacing in 1958, (b) annual surveys from 1959 to 1963 of the conditions of the bituminous concrete resurfacing, (c) rut depth measurements taken in the wheelpaths, and (d) road smoothness measurements taken with the BPR-type Illinois roadometer.

Special field sketch sheets were used for each condition survey to show transverse and longitudinal cracking. Prints of the pre-resurfacing survey sheets were used as underlays for the field sheets of the annual surveys of the resurfacing. Therefore, cracks could be tabulated as reflected or as occurring at previously uncracked locations.

Transverse cracks were tabulated and counted as reflection cracks if they extended half-way or more across the lane. Longitudinal reflection cracks were tabulated as continuous if the total length of short intermittent reflected cracks were at least half the potential crack length. The sketched lengths of longitudinal reflection cracking were tabulated to the nearest 5-ft increment.

TABLE 2

TRAFFIC VOLUME AND COMPOSITION

Year	Total Avg. Daily Traffic	Avg. Daily Truck Traffic		Accumulated 18-Kip Equiv. Single-Axle Loads
		Single Unit	Multi-Unit	
1959	1,650	290	10	7,500
1960	1,650	290	10	15,045
1961	1,700	240	10	21,580
1962	1,700	240	10	28,115
1963	1,700	240	10	34,650

The road smoothness measurements were taken during the annual surveys of 1962 and 1963.

Rut depth measurements were taken in the wheelpaths of both lanes in 1962 and 1963. In general, these measurements were made at 500-ft intervals.

ANALYSIS OF DATA

The condition survey of the old portland cement concrete pavement before resurfacing revealed numerous wide transverse cracks and associated spalling and faulting. There were two general transverse crack intervals, one at nearly 50 ft, between Stations 51 and 86, and the other at approximately 10 to 15 ft, between Stations 86 and 230 (Fig. 4). No reason was found for the wide difference in crack interval between the two locations.

The survey data on longitudinal reflection cracking over the longitudinal widening joints were summarized and analyzed for the two sections with fabric and for the two sections without fabric reinforcement. The progression of longitudinal reflection cracking over the widening joint for both the reinforced and nonreinforced sections is shown graphically in Figure 5.

Reflection cracking over the longitudinal widening joint in the areas containing the welded-wire fabric was not apparent until the fourth survey in 1962, at which time 0.2 percent of the total possible length had reflected. In 1963, after 5 yr of service, the reflected amount was still less than 1 percent. Reflection cracking over the widening joint in areas outside the fabric reinforcement, recorded in 1959 after 1 yr of service, amounted to 0.3 percent of the total possible length. In 1960, the recorded value was approximately 10 percent. This cracking distress progressed to 57 percent in the third year of service, to 63 percent in the fourth year, and to nearly 70 percent by the fifth year, 1963. During these years, single-axle loads accumulated at the rate of about 7,500/yr. All widths of fabric reinforcement appear to be about equally effective in controlling cracking over the widening joint.

The analysis of transverse reflection cracking was limited to the two sections of pavement of initial long and short average joint and crack interval when no reinforcement was used, and to the reinforced sections having near-full-lane-width (10½ ft) fabric. The nonreinforced section of shorter joint and crack interval and the fabric-reinforced section initially had about the same average interval of 13 ft. The other nonreinforced section initially had an average joint and crack interval of about 50 ft. It did not seem appropriate to include in this analysis the sections in which the shorter widths of fabric covered the original transverse joints and cracks only partially. The progression of transverse reflection cracks, expressed as a percentage of the original number of transverse joints and cracks, with time and the accumulation of 18-kip equivalent axle loads is shown in Figure 6. From this figure, it is apparent that the welded-wire fabric has been beneficial in reducing the growth of transverse reflection cracking up to the fifth year of service, with the percentage reflected being about one-fourth that in the 50-ft crack-interval section, and less than one-half the value of the 13-ft crack-interval control section.

It must be pointed out in comparing the two nonreinforced sections that the percentage of transverse cracks reflected is no indication of the over-all surface con-

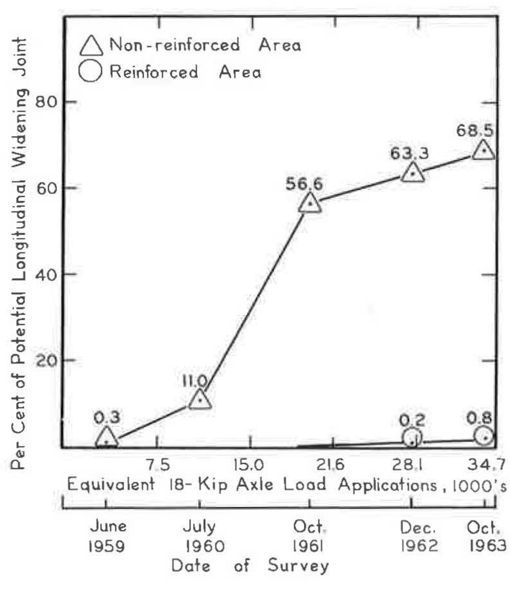

Figure 5. Progression of longitudinal reflection cracking over widening joint.

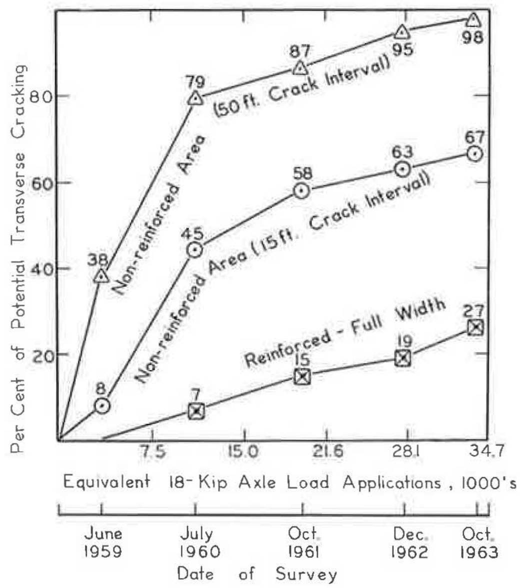

Figure 6. Progression of transverse reflection cracking.

dition of the two areas of pavement because these two areas differ greatly in the average length of uncracked sections, which was 47 ft in the originally longer crack-interval section, 20 ft in the control section, and 70 ft in the continuous full-lane reinforced sections after 5 yr of service.

Smoothness measurements recorded in 1962 and 1963 by the Illinois roadometer have indicated little variation in the riding quality throughout the project. The roughness indices indicated subjective ratings of "slightly rough" in both reinforced and nonreinforced sections. There is no apparent correlation trend that would demonstrate the usefulness of fabric to retard depreciation in the riding quality after 5 yr of service. However, as the cracking progresses to spalling and dislodgement of surface particles, the control of reflection cracking demonstrated to date by the fabric reinforcement may be reflected in the riding quality.

Rut depth measurements up to the fifth year of service, with maximum values less than 0.1 in. on both reinforced and nonreinforced sections, offer no evidence of the effectiveness of fabric to resist rutting or shoving in the bituminous overlay.

DISCUSSION AND FINDINGS

Welded-wire fabric can be incorporated in bituminous concrete resurfacing construction with conventional equipment and without appreciable difficulty. However, some type of hold-down device or a means of securing the fabric to the leveling binder course is required to prevent the tracks and augers of the paving machine from catching in the fabric. During paving operations, the truck driver should carefully control tension on the truck brakes, especially on downgrades, to avoid shifting the wire fabric.

After 5 yr of service under moderate traffic, the welded-wire fabric had practically eliminated the formation of reflected cracks over the longitudinal widening joint and had reduced substantially the reflection cracking over transverse cracks and joints.

The reflected cracks in the areas of no fabric will probably progress to more serious deterioration, as demonstrated by past performance of resurfaced pavements, and, therefore, will eventually affect the riding quality. The degree to which the fabric will be able to control or prevent this progression undoubtedly will provide a real measure of the benefits derived from the use of welded-wire fabric in bituminous concrete resurfacing.

Although this experimental project might eventually provide some definite data regarding the long-term beneficial effects of welded-wire fabric reinforcement, some consideration should be given in future research to incorporating reinforcement in bituminous concrete surfacing over pavements of various crack intervals and stages of deterioration. The inclusion of control sections of various thicknesses of resurfacing to determine possible cost-benefit relationships should also be considered. The cost of the welded-wire fabric installation on this particular project was about the same as the cost of an additional inch of resurfacing thickness.

REFERENCE

1. Chastain, W. E., Sr., "Application of Road Test Formulas in Structural Design of Pavement." HRB Spec. Rept. 73, pp. 299-313 (1962).