pared to live load effects of an HS20-44 design truck.

The model is still intact and has been moved to a new testing facility about 10 miles away. Many other experiments to investigate other structural characteristics are anticipated.

ACKNOWLEDGMENT

The study is jointly supported by the Texas State Department of Highways and Public Transportation and the FHWA under Federal Contract No. HP0010(7). John Panak and R. Sanford are the contact persons for the state and federal agencies, respectively. David Beal of New York State Department of Transportation provided valuable suggestions about model construction.

REFERENCES

1. Drawing and Details for I-Beam Bridges. Revised edition. Texas Highway Department, Austin, 1962.
2. M. Biswas, J.S.B. Iffland, R.E. Schofield, and A.E. Gregory. Precast Bridge Deck Replacement Applications. In TRB Special Report 148: Bridge Replacements with Precast Concrete Panels, TRB, National Research Council, Washington, D.C., 1974.
3. Ontario Highway Bridge Design Code, Section 7. Ontario Ministry of Transportation and Communications, Toronto, Ontario, Canada, 1978.
4. D.B. Beal. Strength of Concrete Bridge Decks. Research Report 89. Engineering Research and Development Bureau, New York State Department of Transportation, Albany, 1981.
5. R.A. Osegueda. Rapid Bridge Deck Replacement. M.S. thesis. Texas A&M University, College Station, 1983.
6. Instructions for the Use of Texas Highway Department Epoxy Binder B-102. Report of the Materials and Test Division. Texas Highway Department, Austin, July 1965.

Publication of this paper sponsored by Committee on Structures Maintenance.

Ohio Turnpike Cuyahoga River Bridge Rehabilitation

WILLIAM S. FREEH

ABSTRACT

Opened to traffic in 1955, the twin two-lane Ohio Turnpike bridges over the Cuyahoga River valley span 2,650 ft and reach as high as 175 ft above the valley floor. Each bridge is comprised of four 100-ft-long girder spans and nine 250-ft-long truss spans supported by 12 reinforced concrete piers. As the use of deicing salts increased during the 1960s, so did deterioration of the concrete portions of the bridges. The original design permitted salt water to flow directly onto the surfaces of the piers. By the mid-1970s deterioration of the piers became evident. Efforts to patch the piers and divert drainage were made, but the piers had already become so saturated with chlorides that deterioration continued. In 1980, under contract to the Ohio Turnpike Commission, Howard, Needles, Tammen & Bergendoff inspected the piers and found that about 40 percent of their surface area was spalled or near spalled. Subsequently the firm recommended methods of repair to prevent recurrence of the condition, prepared plans and specifications for shotcreting (selected alternative), and provided resident construction inspection.

The Ohio Turnpike, opened to traffic in 1955, was designed and constructed before serious consideration was given to mitigating the potentially destructive effects of deicing salts. The twin, two-lane bridges over the Cuyahoga River valley are the longest on the turnpike, spanning 2,650 ft and reaching as high as 175 ft above the valley floor. Each bridge is comprised of four 100-ft-long girder spans and nine 250-ft-long truss spans supported by 12 reinforced concrete piers. The concrete decks, when originally constructed, had open curbs for drainage.

As the use of deicing salts increased during the 1960s, so did deterioration of the concrete portions of the bridges. The deck, where the reinforcing steel is close to the surface, and the edges of the deck slab, where the salt-laden water flowed through the open curbs, were the first areas to show severe spalling. The deck was patched with relative ease, but patching the vertical edges of the slab outside the railing was difficult. To prevent salt water flowing over the fascias, the open curbs were closed in 1967 and all drainage from the decks was diverted through open toothplate-type expansion joints located above all but two of the piers. Joints were located 25 ft from piers 4S and 4N to prevent drainge falling on railroad tracks passing close to these piers.

The piers had been subjected to some salt water flowing through the joints since the first use of deicing salts, but the closing of curb openings increased the flow directly onto the surfaces of the piers. By the mid-1970s there was visible deterioration of the piers (except 4S and 4N). There were some efforts by Ohio Turnpike Commission (OTC) maintenance forces in later years to patch the piers and to divert drainage away from them, but the piers had already become so heavily saturated with chlorides that their deterioration continued.

By 1980 it was apparent that extreme measures were necessary to prevent further deterioration of the piers. The OTC selected the Howard, Needles, Tammen & Bergendoff (HNTB) Cleveland office to make a thorough inspection of the piers, recommend methods of repair to prevent recurrence of the condition, prepare plans and specifications for the repairs, and provide resident construction inspection.

With the aid of spider staging, the entire surface area of each pier (except 4S and 4N) was inspected by hammer sounding to locate delaminated areas in the apparently sound concrete. Hollow sounding areas were outlined in chalk and recorded on scale drawings of the piers as "near spall." Areas where spalling had already occurred were recorded as "spalled." The depth of most spalled areas was at the plane of the outside face of the main vertical reinforcement that ranged between 2.75 and 4 inches. Clearly, spalling and delamination were the result of expansion of the corroded reinforcement.

In the spalled areas it was observed that the horizontal tie bars were heavily corroded but that main vertical bars did not have large metal loss. Concrete was removed around a number of representative vertical bars revealing that corrosion was limited to the outside faces and that good bond remained on the inside portion of the bars. The surface area recorded as spalled or near spalled totaled about 54,000 square feet, representing approximately 40 percent of the surface area of the piers. Samples of concrete were removed from 26 locations in sound concrete and tested for salt content. Results indicated that large amounts of chloride occurred in virtually all exposed pier surfaces.

Several alternatives for repair of the piers were considered. The repair would have to provide protection against continuing deterioration resulting from the high chloride content found in much of the concrete that is sound at present. A routine repair alternative would have been to remove all deteriorated concrete down to sound concrete; replace heavily corroded reinforcement; and replace all missing concrete with conventional concrete, pneumatically placed mortar (shotcrete), or preplaced aggregate and pressure-injected grout. A latex additive could have been used to improve the bond and reduce the permeability of the shotcrete patches.

Following this type of repair, all surfaces of the piers could have been waterproofed to prevent additional water and chloride penetration. However, some moisture would still penetrate the concrete surfaces, migrate through parts of the piers, and combine with the chlorides already present in the sound concrete to cause corrosion of the reinforcement. This solution was therefore not considered sufficiently effective.

The second alternative considered was the use of cathodic protection. Cathodic protection reverses the electrochemical process of corrosion. Two organizations that specialize in cathodic protection and have installed successful systems in bridge decks were consulted. Although neither had ever attempted such a system on a vertical concrete face, both were of the opinion that a successful system could be devised.

The Harco Corporation, one of the cathodic protection specialists consulted, was hired by the OTC to do a corrosion evaluation and cathodic protection feasibility field study on sample piers. Harco prepared a report that stated that it would be feasible to protect the piers cathodically; however, for the cathodic protection to function properly, chlorides would have to be added to the new patches to closely match the chloride content of the adjoining concrete because cathodic protection is most effective in wet, chloride-laden concrete that serves as a conductor. It was also noted that if new concrete patches without chloride were placed adjacent to chloride-laden concrete and if cathodic protection were not used in these areas, a severe battery action would be created, significantly accelerating corrosion.

The method proposed for cathodically protecting the piers included removing all loose and deteriorated concrete, patching with new concrete containing chlorides, installing a system of wires on the surfaces, and then coating the surfaces of the piers with a conductive material. The recommended coating material would have been extremely high in carbonacious material and black in color. The cathodic protection alternative was rejected because it was expensive, experimental, had an obtrusive color, and would require an unknown amount of maintenance.

A third alternative, chloride removal, was considered. Two methods of removal have been successful in laboratory tests. One method uses a flushing technique with ion-free water. The other method involves an electrochemical process. The latter did show promise for bridge decks but would have been impractical for the large vertical surfaces of the piers.

Because the remaining sound concrete did contain potentially destructive amounts of chloride, the cost to remove and replace all surface concrete was investigated. It was found that a much better unit price could be obtained for removing and replacing the entire surface than for treating only the spalled and near spalled areas. Certainly the aesthetics of a patch job would have been undesirable.

HNTB's recommendation to the OTC was that all surface concrete on all piers (except piers 4S and 4N) be removed to a minimum of 1.5 in. behind the main reinforcing steel; that all surfaces be sandblasted; and that the surfaces be restored with either plain shotcrete or preplaced aggregate and pressure-injected grout, ensuring that the steel reinforcement would not be in contact with any chloride-contaminated concrete. To maintain the structural integrity of the piers it would be necessary to design a sequence of removal and replacement. The OTC accepted the recommendation and authorized HNTB to prepare plans and specifications.

It has been the experience of HNTB that some repair projects using shotcrete have not given good long-term results. For this project the objective was to achieve a repair that would last 50 years or more, and because this project would be one of the largest such pier repair projects ever undertaken, HNTB, with concurrence of the OTC, engaged the services of Thomas J. Reading, a nationally recognized authority on shotcrete. Reading has had some 30 years experience in this field, was formerly Chairman of the American Concrete Institute (ACI) Committee on Shotcrete, and is an active member of that committee. He assisted in writing the shotcrete specification, field testing and qualifying the nozzlemen, and organizing other field controls.

Plans were prepared showing the construction sequence necessary to maintain structural integrity, and bids were taken on the two recommended alternative methods of repair. Only one bid for the preplaced aggregate alternative was received, and it was significantly higher than the low bid of $2,731,176 for shotcrete repair submitted by the Pressure Concrete Construction Company. Pressure Concrete was awarded the contract and the project was begun on May 17, 1982.

Type 1A cement was specified. It was supplied in tank trucks for all piers east of the Cuyahoga River by the Dundee Cement Company and in standard sacks for the piers west of the river by the Medusa Cement Company. The fine aggregate, a natural sand, had to meet the requirements of the ACI specifications. Samples were taken for testing from all stockpiles in the supplier's yard and again at the site. Moisture content was frequently checked and was usually between 3.5 and 4.5 percent. The sand was covered to protect it from rain because an increase in moisture content would hamper pumping the sand-cement mix through the hoses. The water-cement ratio varied between 0.35 and 0.45 by weight. The mixing and curing water was hauled to the site in tank trucks from a local municipal supply.

Specifications required that cement and sand be batched by weight. The contractor was granted permission to batch volumetrically, provided periodic weight checks were made to ensure the specified ratio of cement to sand. Frequent calibration of the batching equipment--Concrete Mobiles--was found to be necessary. Periodically separate samples of cement and sand were taken and weighed just before combining. Additional tests were made to determine the ratio of cement to sand in the mixture before it was discharged into the shotcreting machine.

Dry-mix (i.e., except for the free moisture in the sand, all mixing water was added at the nozzle) shotcreting equipment was used. Shooting equipment consisted of four Jetcreters supplemented by one Micon and one Maynedier gun. The guns required frequent maintenance mainly because of wear. An ample supply of compressed air was provided with pressures as high as 110 psi at the gun. Air pressures at the nozzle were estimated at about 80 psi for most applications.

The shooting of test panels was required before shotcrete was applied to the structure to establish the mix proportions and qualify the nozzlemen. Although all of the nozzlemen had had prior experience, each was required to demonstrate his ability to apply shotcrete of the required quality. The test panels were 3 ft square at the back with sides flared out at a 60-degree angle. To simulate actual project conditions, the thickness of the panels and the location and size of reinforcing bars and wire mesh were the same as in the structure.

A 28-day strength of 4,200 psi on cores taken from the preconstruction panels was required--20 percent more than the 3,500 psi required in control test panels taken during construction. The panels were also required to be substantially free from lenses and sand pockets and have good bond of the shotcrete to the reinforcement. Because the reinforcing bars were as large as No. 11 and as closely spaced as 8 in. (5 in. at laps) some difficulty in obtaining sound shotcrete was anticipated.

The first series of panels was rejected because of low strength, attributable to insufficient cement resulting from improper batching. Several nozzlemen were disqualified because of the presence of sand lenses and pockets behind the reinforcing bars of their test panels. It was determined that a mix somewhat wetter than that used in ordinary shotcrete construction was necessary for the material to flow around the reinforcing bars without forming sand pockets. The mix proportions had to be adjusted to obtain the needed strength and to compensate for the small amount of added water. Because the mix was on the wet side, it was necessary to use a design of 1 part cement to 3.5 parts sand by weight. This generally gave strengths of about 5,000 psi. The high strength level was considered desirable because shotcrete is usually more variable than ordinary concrete. This mix design also provided good freeze-thaw resistance. Nozzlemen who failed to qualify on their first attempt were given the opportunity to gun another panel.

Field control testing was done on panels having the same size and features as those in the preconstruction tests. They were gunned on the same day as that portion of the structure that they represented, to provide an indication of the quality of shotcrete in the structure. Cores taken from the control test panels were required to have a 28-day strength of 3,500 psi. Seven-day strengths were also determined to compute the strength ratio for the two ages. This permitted an early estimate of the acceptability of the shotcrete.

A generous number of control test panels were made early in the work, but the number was reduced after the number of cores taken from the structure was sufficient to indicate a satisfactory correlation. In all, 42 control test panels were made. With few exceptions there were no problems in obtaining shotcrete of the required quality. The shotcrete was applied in two layers placed several weeks apart. The first layer was about 4.5 in. thick and encompassed the existing reinforcing bars. The second was about 2 in. thick and encompassed the newly installed wire mesh. The bottoms of the piers were usually shot first, and the remainder was done by working from the top down.

Because of the height of the piers (some 100 feet) the gunning on each pier extended over a 3- to 4-day period. This resulted in bonding problems. When working from the top down overspray and waste shotcrete diluted by water from the nozzle ran down over the piers and created a coating that could not be removed by water blast. In the early stages brooming of the surface of the first shotcrete layer was too light or was done too early (before bleeding was complete) to produce a good bonding surface. Also, a coating of the type described previously tended to develop on this surface when the second layer of shotcrete was applied.

Because these conditions could lead to poor bond, they were closely observed by the inspectors. Considerable sandblasting or material blasting (using the regular shotcrete mix with no water added at the nozzle) was required before these areas were covered with shotcrete.

After the shotcrete had hardened, each layer was sounded with a hammer to check for drummy areas. Particular attention was paid to locations where drumminess was thought to be most likely. Ten piers were found to have drummy areas, most of which were small. The total area involved was about 500 square feet, only 0.4 percent of the area of the shotcrete. Most of the drummy areas were found in the lower half of the piers where the coating problem was greatest. Virtually all drummy areas occurred between the two shotcrete layers. All drummy areas were chipped out and reshot.

The acceptability of the shotcrete was determined mainly from cores taken from the structure. These were usually taken through the two layers comprising the 6.5-in. thickness. Following the rationale in ACI Shotcrete Specification 506.2-77, it was required that the average of three 28-day tests from any day's work should not be less than 3,000 psi and no single test should be below 2,600 psi. In all about 120 cores were taken from the structure. The overall average strength exceeded 5,500 psi. The lowest average of three tests was 3,500 psi, and one core tested below the 2,600 psi requirement.

The low core had a 28-day strength of 2,215 psi. A proximate core had a 7-day strength of 2,100 psi. These cores were taken from the first layer of the top drop (the top 6 ft of the pier) of pier 8S on the south end where the quality of shotcrete was suspect because of trouble in the batching operation. The shotcrete in the cores appeared sound to the eye. The apparent cause of the low strength was inadequate cement content resulting from faulty operation of the Concrete Mobile. The layer was sounded and no drummy areas were found. The matter was discussed with structural engineers who thought that this slight strength deficiency could be tolerated in this area where loading conditions are insignificant. The batching machine was adjusted, and cores taken from the next drop were very good. It was therefore decided to accept this drop and allow the contractor to proceed with the second shotcrete layer. The appearance of the shotcrete in the cores from the structures was very good; there were a minimum of lenses and sand pockets. Because of the favorable results obtained, the number of cores was reduced for the later piers.

HNTB is of the opinion that the aesthetics achieved by the contractor were much better than anticipated. The specifications called for a flashcoat finish, but at the suggestion of the contractor a sample with a finish struck off with a trowel was administered to a section of a pier and compared to a sample of the flashcoat. The finish with the trowel was selected. Wire guides were used on every corner and at about 3 or 4 foot centers on flat surfaces. The combination of trowel finish and wire controls produced very sharp lines much like those of a formed surface, except that there were no form marks.

In order to protect the repaired piers from new salt penetration they were treated with Chem-Trete BSM40 weatherproofing after the proper cure time had elapsed.

HNTB believes that the well-researched, clear, and strictly adhered to specifications will achieve the desired 50-year life expectancy of this major shotcrete repair project. A second rehabilitation contract for the bridges has been let. This contract will include a new deck, about 10 ft wider than the existing deck, with sealed expansion joints and a closed drainage system. The westbound bridge was rehabilitated in 1983, and the eastbound bridge is scheduled for completion in 1984.

Publication of this paper sponsored by Committee on Structures Maintenance.

Rivet Replacement Criteria

R.N. FAZIO and A.E. FAZIO

ABSTRACT

The New Jersey Transit Corporation (NJ Transit) is currently implementing a major capital improvements program to upgrade its physical plant. The rehabilitation of existing bridges is a major element of this work. The adoption of rivet replacement criteria for the various bridges programmed for rehabilitation is discussed in this paper. The rivet replacement criteria have been developed for use as a guideline by the engineer during inspection, design, and construction of the various bridges programmed for rehabilitation. The criteria developed are simple, reliable, and reproducible and provide a uniform evaluation scheme for the 600 railroad bridges found within NJ Transit's physical plant. In this paper the importance of loading conditions, type of connection, grip length, and cost as parameters to be considered in assessing if a rivet should be replaced is discussed.

The New Jersey Transit Corporation (NJ Transit) was created by the state legislature in 1979 and has been chartered to run all commuter passenger trains in the state of New Jersey. NJ Transit is the third largest commuter rail system in the nation and includes 490 route miles of track, 600 undergrade bridges, 75 locomotives, 968 passenger cars, and 142 stations. As a result of years of deferred maintenance, NJ Transit is in the process of implementing a major capital improvements program to upgrade its physical plant. The rehabilitation and replacement of various bridges within the rail system is a major element of this program. NJ Transit bridges vary in length from 5 ft to 2,926 ft and were found to have deficiencies that ranged from minor paint loss to major structural deterioration.

In this paper the adoption of uniform rivet replacement criteria for the various bridges that are programmed for rehabilitation is discussed. The criteria are developed to meet the following goals: (a) provide standard rivet replacement criteria that are simple, reliable, and reproducible for the 600 railroad bridges within NJ Transit's physical plant; (b) provide the various consulting firms, construction contractors, and in-house staff standard criteria to be used for the many bridges programmed for rehabilitation; (c) give guidance to the engineer during the inspection, design, construction, and quality control phases in selecting which rivets should be replaced; and (d) allow the development of more accurate rivet replacement costs for the bridges programmed for rehabilitation.

PROBLEM FORMULATION

Any structure consists of individual members that must be fastened together to create a structural system that is compatible with its intended service. If the connections are inadequate the structural