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Publication of this paper sponsored by Committee on Concrete Bridges.

Strengthening of Bridge Beams and Similar Structures by Means of Epoxy-Resin-Bonded External Reinforcement

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This paper describes the technique of increasing beam strength by means of externally applied reinforcing steel plates, bonded by means of epoxy-resin adhesive. Test methods, application data, and some indication of preferred resin formulation are given. Steel plates are bonded to the beams by means of epoxy-resin paste adhesive. During curing of the resin the plates are held in place by clamps and/or bolts. This provides an inexpensive method of restoring or upgrading the strength of damaged or substrength concrete bridge and similar beams. To date, results have proved entirely satisfactory, but possible dangers of corrosion on the steel-concrete interface are being investigated. Further work is foreseen that will employ new materials and methods of application and enlarge the scope of the process.

The use of epoxy-resin adhesives in bridge construction and repair is well known (1,2). Spall repair compounds, rebar adhesives, new-to-old concrete bonding, and the bonding of precast units have been used for more than 20 years. In 1964, the accidental omission of reinforcing steelwork in an apartment complex in Durban, South Africa, resulted in the first recorded application of malleable steel plates bonded to the basement beams in order to replace the missing bars. Research and control operations were carried out by the University of Natal (3). Many further contracts have been carried out in South Africa, e.g., on bridges and in a damaged multistory parking garage.

According to current practice in South Africa, typical specifications call for the adhesive to be gray and have the consistency of a thick paste. The maximum vertical wet film is 15 mm, the thickness of the wet film is unlimited, the minimum pot life is 1 h, the curing time at 62°F is several days, and the minimum shelf life is 12 months; application is by trowel. The mechanical properties of the adhesive after full cure are a minimum compressive strength of 80 MPa, a minimum tensile strength of 11 MPa, and a minimum shear strength of 10 MPa.

The procedure is as follows:

1. The concrete is drilled at centers of approximately 12 in and threaded bolts of convenient size are set into these holes; a diameter of 0.625 in is convenient. The bars are grouted into place with a suitable epoxy adhesive, depending on whether the holes are horizontal, vertical, or inverted.
2. The concrete is scabbled (rough dressed) to expose the main aggregate, and then all dust is removed by an industrial air blower or vacuum cleaner.
3. A template is made to obtain the pattern of the bolts, and this is transferred to the steel plate. The plate is then drilled and any machine oil is removed before the grit blasting.
4. The adhesive is mixed and spread on the plate in a wedge shape with the apex along the center line of the plate. The concrete is primed with epoxy primer.
5. The steel plate is transferred into its final position. The bolts are then tightened to secure the plate to the concrete surface and to squeeze out surplus adhesive.
6. The steel plate may be subsequently treated with an epoxy enamel for corrosion protection or coated with a wet-to-dry concrete adhesive and then given a plaster treatment. Where fire protection is important, the thickness of cover should be a minimum of 1.5 in.

The following basic working principles have been established:

1. The technique represents an inexpensive means of replacing lost strength or of increasing strength to take additional loads.
2. The success of the operation depends on the successful adhesion of plate to concrete, i.e., careful preparation of both surfaces, correct clamping of the plate during the critical bonding phase, and, above all, the correct application of a suitably formulated epoxy adhesive. Initially the plates were bonded to the vertical faces of the beams, but subsequently the plates have been bonded to the top faces and undersides of the beams.

Similar work on a major telephone exchange in Switzerland in 1971 provided satisfactory results under both static and dynamic load conditions (4). The technique has also been extensively studied in France (5,6).

DESIGN

The basic design method follows the basic reinforced concrete theory in which, for calculation purposes, the additional steel plates are transformed into an equivalent concrete section and the modular ratio is adjusted for live or dead load.

Since, in practice, the new reinforcement can only be stressed under live load, it is necessary first to determine the dead-load stresses in the existing section and next to determine the live-load stresses on the rebuilt section. The stresses are then added algebraically; care must be taken that the original section is not overstressed.

If steel is accepted as the only practical repair material, the maximum allowable stress in the

Figure 1. Plates bonded to the underside of the side-span beams of a bridge at Swanley, Kent.

existing reinforcement will limit the stress in the added plates to about 50 percent of its permissible limit, unless the dead load can be eased by jacks or props.

In regard to plate area, an optimum area can be established. The greater the area of additional steel, the lower the neutral axis, which, in turn, lowers the extreme fiber stress until a point is reached at which there is no appreciable increase in the moment of resistance.

CASE HISTORIES

In 1975, four bridges on the M5 Motorway at Quinton Interchange, Worcestershire, England, were repaired by plating, and in 1977 two bridges on the M25-M20 Motorway Interchange at Swanley, Kent, were also treated successfully (see Figure 1).

At Quinton, cracks were discovered in the soffits of the side and main spans during a routine inspection of the bridges, which had been completed in 1966 but not opened to traffic until 1970. The bridges were all box reinforced concrete slabs, continuous over three spans of 55, 90, and 55 ft. Calculations suggested that inadequate tension reinforcement had originally been provided in parts of the side spans and in the edge beams of the main spans.

Two alternative repair methods were considered: prestressing and bonding additional reinforcement. Prestressing was discounted for three reasons: (a) although the deck construction was of box design, a very considerable force would be required to stress the concrete effectively; (b) it would be difficult to establish suitable anchorages in the slab soffits for the force required; and (c) headroom under the bridge would be adversely affected by the necessary concrete cover for the stressing cables.

The amount of surface preparation required to receive the reinforcing plates appeared to be minimal. Another advantage was that the bonding operation required less technical control than did prestressing.

The initial plan was to bond 0.5-in steel plates directly to the concrete and, during bonding of the epoxy resin, to hold the plates in place by means of fixing bolts, which could also act as fail-safe connectors in the event of bond failure. However, it proved impossible to place the bolts in the beams because of the presence of metal objects, e.g., bonding wire, near the surface. What had appeared to be a uniform surface was not so, because of deformation in the formwork during construction.

To avoid the expense of smoothing the concrete to accept a 0.5-in thickness of plate, a compromise was devised. First, the concrete surface was grit blasted. Tests indicated that similar pull-off results were obtained by using grit, needle-gun treatment, or scabbling. Next, major differences, e.g., local high spots, were reduced by scabbling. Even so, the problem of mating the 0.5-in steel plate to the concrete beam was such that high bending stresses in the plate and residual tensile stresses in the cured resin would have resulted.

Accordingly, the design was modified by using two layers of 0.25-in plate that overlapped each other. The fixing bolts were located to match with the lower parts, i.e., where the flue line might be at its thickest; in practice the bolts were at 4-ft centers. On-site tests confirmed that the use of a comparatively high-viscosity two-part resin system was desirable; low-viscosity epoxy adhesive tended to be squeezed out more easily under pressure but had the tendency to continue flowing. The high-viscosity system proved easier to handle and more economical. The prepared additive, designed by Ciba-Geigy, operated on a 24-h initial cure at 5°C and gave a shear strength of 2000 lbf/in² at seven days. Check tests revealed a bond area in excess of 90 percent, and further improvements were obtained by localized wedging.

SITE PRACTICE

The work program was scheduled over a four-month period for the four bridges; thus, full-scale closures were unacceptable. Reports that a bridge near Paris, France, had been repaired in a similar manner by using only lane closures was considered, but it was felt that lane closure was more hazardous to the repair crew than was allowing the traffic to flow freely. Accordingly, tests were carried out based on a design shear stress of 200 lbf/in² for the steel-resin-concrete interfaces. Although the tests were perfunctory because of time limitations, the results clearly indicated that, since there would be at worst a 30-percent reduction on an additive that had a lap strength of more than 1000

lbf/in^2, the weakest link would inevitably be the concrete itself, which had an assumed shear strength of 450 lbf/in^2 (compressive strength of 6500 lbf/in^2).

If a factor of safety of 2 with only 75 percent actual bond in the adhesive layer is assumed, a maximum design stress of $(450 \times 75) \div (100 \times 2) = 168$ lbf/in^2 is obtained. In practice, when 10-in-wide plates are used, it was calculated that the actual design stress would not exceed 35 lbf/in^2.

By using these stresses as a basis, the decision was reached to use the double 0.25-in plate bonding technique on the middle span only, since single plates were considered sufficient for the side spans. As a safety precaution, the edge beams of the main spans were reinforced with three layers of plate, all of which were shot blasted prior to delivery on site, then flash blasted immediately before use. This procedure eliminated the need for priming coats of resin or other protective treatments. During the operation, the resin quality was controlled by daily lap and pull-off tests, and the resin and hardener components were checked by master spectrographs.

Load tests were carried out on the structure before and after plating by using a 40-ton axle, which induced a stress of 1100 lbf/in^2 on the plates. Since this is approximately the lower limit of strain-gauge sensitivity, the tests were not regarded as conclusive. Furthermore, since temperatures and weather conditions were variable, any figures obtained were further suspect.

The strengthening operation proved uneventful; the only failure was a crack in the adhesive layer at the free end of one plate. It was decided that the local wedges had been removed too early or that one located further in had been overtightened and had caused tensile stress. Accordingly, plate ends were clamped in position. The operation was completed on schedule at a cost of $197 800. The price breakdown was as follows:

Item	Cost ($)
Steel plates (50 t)	36 550
Epoxy resin	38 700
Scaffolding and plant	40 850
Labor	81 700

Full-scale reconstruction of the bridges by conventional methods, however, even ignoring the inconvenience of traffic disruption, was estimated at $1 700 000.

A similar technique was employed on the M25-M20 Interchange in 1977, following discovery of cracks in the soffit of one side span immediately after the interchange had been opened to traffic (7). In this instance, approximately 450 plates 19.5 ft long, 9.5 in wide, and 0.25 in thick, each weighing 1650 lb, were placed in one month. A trowel-grade adhesive Ciba-Geigy XD800 two-part epoxy resin that had a pot life of 45 min at 60°F was used. Plates were applied on both the top surface and the underside of the beams. For the soffit work a running falsework on which the plate could be placed and then wedged against the soffit was used. For the top surface, the plates were weighted down in place with sandbags.

PERFORMANCE

To date, both these sites are performing in an entirely satisfactory manner. However, the following practical considerations are posed in regard to the long-term success of this type of treatment:

1. Where there are joints in the plates, e.g., butt or lap joints, the possibility of high localized stress occurs. A possible solution to this is welding the plates at butt joints; this has been carried out in South Africa, but the heat generated in welding will destroy the epoxy resin to a greater or lesser extent, depending on the skill of the welder.
2. Plate thickness may be affected by the soffit section. The possibility of partial bonding, e.g., at the ends of arched beams only, has been investigated by the University of Natal.
3. Under increased loading, cracking along the plane of the resin-concrete interface may be initiated from a crack bridged by the plate reinforcement.

In regard to the materials themselves, the ingress of water or corrosive chemicals along the edges of a shot-blasted steel plate could set up oxidization beneath the adhesive layer. Doubt has also been cast on the creep properties of epoxy resin in general. In addition, the cyclic fluctuations, traffic, temperature, etc., during curing that were referred to in connection with the Quinton repairs require further testing. For these reasons, the U.K. Standing Committee on Structural Safety report for the year ended March 31, 1978, called for research and monitoring on a long-term basis before the technique is recommended for general use.

In a study by the U.K. Transport and Road Research Laboratory (8), tests were carried out on plate-reinforced beams to study the effects of a change of adhesive, a joint in the plate, variation in plate thickness, and load cycles. The conclusions reached included the following:

1. The failure plane occurs in the concrete adjacent to the steel plate, commencing at the free end.
2. Extensive cracking of beams occurred well before the separation of the plate commenced. The report indicates that this separation occurred suddenly, although the cracking and other factors gave prior warning of failure. It is my contention that the period between initial cracking and failure can be considerably modified by the type of resin formulation employed; resilient formulations have been based on epoxy-polysulfide blends.
3. Depending on the type of epoxy mix tested and the plates themselves, the postcracking stiffness was increased between 35 and 105 percent.
4. The four variables tested had little effect on the load required to cause separation.
5. The avoidance of high local stresses, especially at plate joints, is of importance, and the second plate layer may well separate before the first plate has completely failed.

POSSIBLE DEVELOPMENTS

There are a number of other developments that might be evaluated, some of a practical and some of a theoretical nature.

1. Rather than by application of an epoxy paste adhesive, the plate might be set up "dry" and shims used to give a predetermined resin thickness. The edges might then be sealed and the epoxy adhesive introduced by either vacuum or pressure, enabling the plates to pick up full live load.
2. Other reinforcing materials that have a high modulus, e.g., carbon fibers, might be considered.
3. Other allied operations such as strengthening

masonry, brick arches, or even steel structures themselves might be considered.

ACKNOWLEDGMENT

Much of the information in this review has been made available to me by R. F. Mander of the U.K. Department of Transport, to whom grateful acknowledgement is made. Data and photographs have also been kindly supplied by the U.K. Transport and Road Research Laboratory; Peter Davies of Ciba-Geigy; W. Patterson of Africa Bitumen Emulsions (Pty) Ltd., Johannesburg, South Africa; and Prostruct (Pty) Ltd., Durban, South Africa.

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Publication of this paper sponsored by Committee on Structures Maintenance.

Latex-Modified Concrete Bridge Deck Overlays: Field Performance Analysis

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Data on field performance of 132 bridge decks overlaid with latex mortar or concrete in Ohio, Michigan, Kentucky, and West Virginia, as well as some data from Minnesota and Vermont, were collected. Common durability distress features and the factors that might influence them were identified. Statistical analysis was performed to determine the set of variables that best explains the variation in the surface distress features among the different overlay projects investigated and to quantify the relationship between overlay condition and the pertinent variables through the formulation of regression models. Further assessment of available performance data was conducted to connect the obtained relationships with the limited data on effectiveness of latex overlays in providing corrosion protection to the deck rebars and to develop hypotheses on the formation and development of the durability deficiency features of latex overlays. The results obtained and the conclusions drawn explain, quantify, and delineate the interrelationship among such factors as years of service, average daily traffic, trafficked versus untrafficked decks during placement, continuous versus simply supported decks, thickness of overlay, and skid number on the durability and corrosion-protection capability of the overlay.

This study was initiated to provide administrators and bridge designers in the state of Ohio with an analysis of available data on field performance of latex-modified concrete and mortar overlays. Since, at the time the study started, only a score of such overlaid bridges were in service in Ohio, it was decided to extend the survey to the neighboring states of Michigan, Kentucky, and West Virginia, where a larger number of such decks existed. As the study progressed, some data from Vermont and Minnesota also became available.

The total number of latex overlay projects investigated was 132; 47 were in Ohio, 57 in Michigan, 17 in Kentucky, and 11 in West Virginia. Most of the latex overlays used before 1972 were of the mortar type, 19-25 mm (0.75-1.0 in) thick. Since 1972, the majority of the latex overlays have been of the concrete type, 25-44 mm (1.0-1.75 in) thick. The actual thickness, however, may be larger in rehabilitation jobs in which the scarified concrete thickness causes the above-mentioned values to be increased. Only 21 of the projects investigated were new two-course construction; the remaining 111 were rehabilitation projects on old decks. Twelve of the new construction projects had mortar overlays, and 9 had concrete overlays. Twenty-one of the rehabilitation projects had mortar overlays, and 90 had concrete overlays. Almost all of the latex overlays investigated used Dow latex modifier A. However, 20 used Dow modifier B.

Performance of latex-modified concrete bridge deck overlays relates to their durability, antiskid quality, and effectiveness in resisting chloride ingress and preventing corrosion of the rebars. Deficiencies in durability are usually manifested by the existence of one or more of the following physical expressions.

DEFICIENCIES IN DURABILITY

Overlay Distress

Overlay distress can be characterized by one or more of the following features: