Behavior and Repair of Deteriorated Reinforced Concrete Beams

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The results of a study on reinforced concrete beams subjected to deterioration effects such as those caused by the action of deicing salts are presented. Included are descriptions of the basic problems in the field, including the corrosion mechanism, and the experimental results. Beam tests are described that show the behavior of beams with specific conditions of interrupted cover and/or bond. Specific signs of distress useful in field inspection are emphasized. Also described is a series of tests of those same beams after repair. Specific attention is given to how much of the beam's original strength can be expected to be recovered after rehabilitation.

The purpose of the experimental and analytic study described in this paper is to provide quantitative information concerning the effect of loss of cover and flexural bond on the strength of reinforced concrete beams and to experimentally determine the response of the beam after repair. The overall study leads to effective repair for beams subject to these deficiencies as well as to an understanding of the behavior of these members.

To initiate this study, a total of 40 reduced-scale 5-in x 10-in x 9.5-ft (12.7-cm x 25.4-cm x 2.9-m) reinforced concrete beams were constructed with predetermined and varying amounts of cover removal and bond loss in the region of varying flexural stress (1-3). The beams were loaded until failure occurred. As load was applied, the progressive values of strain in the reinforcing steel bars, as well as load and deflection increments, were recorded. The presentation includes fabrication details, testing details, and conclusions on the beam's behavior when bond or cover does not exist and on the flexural capacity restored to the beam by the repair process (4,5).

The loss of significant amounts of cover and flexural bond in reinforced concrete beams has become a widespread problem. The problem is most commonly found in bridges and parking garages that are subjected to deicing salt. The salt releases chloride ions that enter the concrete and corrode the reinforcing steel. The corrosion products increase in volume, which results in cracking of the concrete cover and progressive loss of flexural bond. Insufficient research has been conducted to date to determine whether loss of cover and flexural bond significantly reduces the strength of reinforced concrete beams and to quantify that loss. Little or no information exists to document quantitatively the behavior of the beam after repair. This project addresses these subjects.

CORROSION PROCESS

This work deals with the reinforced concrete beam subjected to the detrimental effects of actions equal to or similar to those of deicing salt applications. In view of this, it is appropriate to briefly present a description of the corrosion process.

Chemically, concrete can be characterized as a saturated solution of calcium hydroxide and is very alkaline in nature. Good-quality, uncontaminated concrete has a pH of approximately 12.5. The reinforcing steel bar, chemically iron, is unstable in an oxygen environment. Initially, the bar's corrosion rate is high and forms a passive oxide film around the steel bar, which in turn acts as a barrier to moisture and oxygen. At this point, the corrosion rate decreases markedly and the reinforced concrete beam is stable. This passive oxide film is very stable in the high-pH environment; thus, the steel reinforces the concrete, and the alkaline nature of the concrete stabilizes the oxide film on the steel, preventing further corrosion.

The resultant effect of the application of deicing salts is to destroy this protective oxide film. Both calcium chloride and sodium chloride are largely chloride (64 and 61 percent, respectively). Both materials ionize rapidly in water, which leaves the chloride ion free to migrate into the concrete. The chloride ion (Cl^-) is chemically aggressive and will lower the pH of the concrete. The lower pH allows the passive oxide film to dissolve and exposes the reinforcing bar to additional moisture and oxygen.

The basic corrosion mechanism, then, is as follows: Salt is introduced into the reinforced concrete. The salt dissolves and the chloride ion penetrates the concrete through cracks, voids, and surface pores, lowering the pH of the concrete. The existing oxide film dissolves, separates from the steel bar, and exposes it to moisture and oxygen. The process continues with corrosion of the steel and separation of the oxides. The process builds up pressures on the order of 4000 psi (27.6 MPa) as a result of the expansion of the corrosion product. The surfaces above and below the reinforcing bar are separated and thus delaminated by the internal stresses. This is a continuing process unless specifically arrested by rehabilitation (6). The delamination breaks at the surface of the concrete and forms a spall that is in turn removed by freeze-thaw cycles and the general service performance of the beam. The amount of deterioration and its duration will depend on numerous factors, such as the ratio of cover to bar diameter, the general arrangement of bars within the section, and the porosity of the concrete (7;8, p. 79).

In summary, then, it is the attack of the chloride ion (Cl^-) and the changing pH of the concrete that lead to the deterioration of the reinforced concrete. The time element of this deterioration is highly variable. It is severely affected by weather conditions, including the number of freeze-thaw cycles, the nature and quality of the concrete mix itself, the amount of protection (cover) afforded the steel bars, and the amount of deicing salts applied to the concrete surface, in addition to other variables. Severe deterioration has been observed in parking garages in as short a time as two years. This paper deals with the structural behavior and strength of these deteriorated reinforced concrete beams.

TEST BEAMS

A total of 40 beams were constructed. Each beam had a basic cross section 5 in (12.7 cm) wide and 10 in (25.4 cm) deep, and the reinforcement consisted of two no. 4 bars placed at an effective depth of 8.25 in (21 cm). Stirrups made of 0.187-in (4.8-mm) diameter smooth wire were placed at 4 in (10.2 cm) center to center in the approximate outer thirds of the beam to avoid a premature shear failure. Five control beams were cast without any interruption of cover and/or bond. The cross section is shown in Figure 1 and the steel placement in Figure 2.

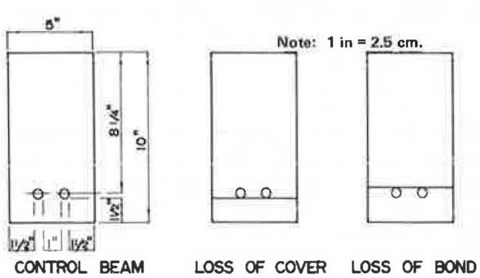
To achieve an effective loss of cover and loss of cover and bond in the laboratory beams, concrete was blocked out in the casting process in order to simulate deterioration. This was done for 35 beams. For a loss of the 1.5-in (3.8-cm) cover, concrete was blocked out from the bottom of the reinforcing bar to the bottom of the beam. For a loss of both cover and flexural bond, concrete was blocked out from the top of the reinforcing bar to the bottom of the beam in such a way that a piece of paper could be inserted in between the bar and the cast concrete. This pattern is shown in Figure 1. The concrete was vibrated briefly to ensure complete filling of the form work. Both cover and bond were removed for distances of 1-6 ft (0.3-1.8 m), in 1-ft increments, symmetrically about the beam's centerline. A total of 12 beams with both cover and bond removed from 1 to 3 ft (0.3-0.9 m) were repaired, after they were initially tested to failure. The repair was done in two ways. One group was repaired by using a 6000-psi (41.4-MPa) mortar mix without preapplication of a bonding agent, and one similar group was repaired by using the same mortar mix after application of a bonding agent. All 12 beams were then again tested to failure.

The beams were 10 ft (3 m) long and spanned 9.5 ft (2.9 m). The load was applied as one concentrated load on a 5-in (12.7-cm) square bearing pad located at a point just outside the zone of simulated deterioration. The bonding agent used was a Type II epoxy (5) and is described later in this paper.

MATERIALS

The materials were representative of those conventionally used in reinforced concrete construction such as parking garages and bridges. The reinforcing bars were no. 4 standard deformed bars with an average tensile yield point of 63.5 ksi (438 MPa) and a range of 62.0-65.0 ksi (427-448 MPa). Stirrups were fabricated of 0.187-in (4.8-mm) round smooth wire. The concrete used throughout the project was supplied by a local ready-mix plant. The 28-day compressive strength averaged 6350 psi (43.8 MPa) and ranged from 6100 to 6600 psi (42-45.5 MPa). The modulus of rupture averaged 700 psi (4.8 MPa). Failure models of the cylinders were consistent and typical of the anticipated conical splitting usually observed in well-made cylinders. Curing was done at a temperature range of 76°-78°F (24.4°-25.6°C), room temperature in the laboratory, and under taped plastic sheets for all beams and cylinders. The five control beams, those without any interruption of cover or bond, were tested over the two-year term of this project. The failure loading was an average of 11 ± 0.5 kips (48.9 ± 2.2 kN), and the testing error was estimated at less than ±0.2 kip (0.9 kN). The slump of the concrete, which used a bank-run gravel 0.375-in (9.5-mm) maximum-size coarse aggregate and portland cement, averaged 2.5 in (6.4 cm). It is the judgment of the investigators that the material properties and beam properties produced were sufficiently consistent as to have no practical or significant effect on the results and conclusions of the test program.

Figure 1. Beam details.



BONDING AGENT

In the beam-repair process, only one bonding agent was used: a clear, amber-colored amino-amine epoxy system that contained no mineral filler (5). It suited the procedures of the test project in the laboratory because of its convenient physical properties. With a pot life of slightly more than 1 h, it could be applied in temperatures of 60°F (15.6°C) and above, was tack free in less than 2 h, and was hard dry after one overnight curing period. It was a relatively thick syrup but was readily applied by brush. It should be noted that it was not the purpose of this project to test the various bonding agents available (4) but rather to ascertain their integral effect on the repair process of the beam.

TEST PROCEDURE

The beam tests were performed by using a self-containing testing-installation structural frame and the MTS hydraulic testing machine manufactured by MTS Systems Corporation of Minneapolis, Minnesota.

This allowed a consistent application of the load in 500-lb (227-kg) increments, at a rate of 20 lb/s (9 kg/s) at 3-min intervals. Deflection measurements and strain gage readings were recorded immediately before and after each 500-lb load increment was applied. Deflections were measured at midspan by using a Soiltest 0.001-in (0.025-mm) dial gage.

Twenty-six of the beams were tested with the location of the concentrated load at a point 3 ft (0.9 m) from the left-hand reaction. Others were tested with the load 1.5 ft (0.46 m) from that support due to the increased length of simulated deterioration. These locations are also shown in Figure 2. This created in the test beams a con-

Figure 2. Dimensions of test beam.

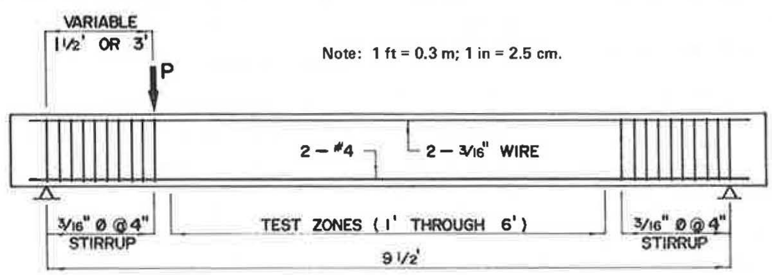


Figure 3. Structural mechanics.

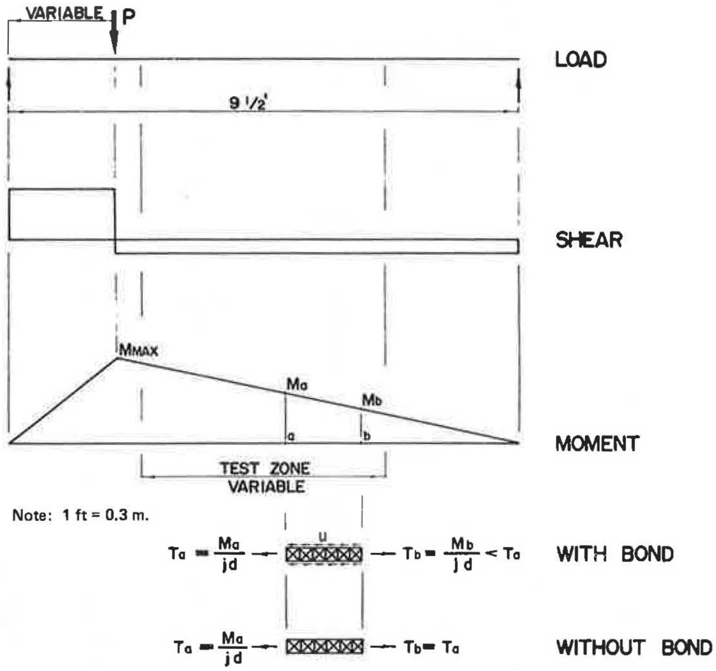
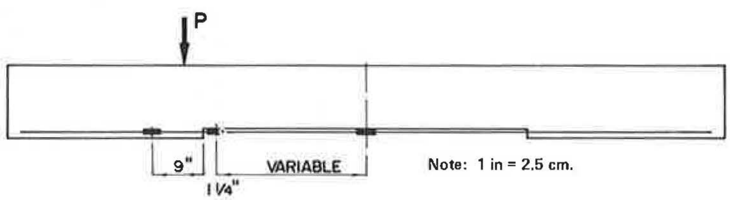


Figure 4. Strain-gage locations.



stantly changing moment gradient, which is necessary to induce variable tensile stresses in the reinforcing bar and the corresponding magnitudes of flexural bond. This is shown in Figure 3, where the fundamental behavior of the composite reinforced concrete beam is indicated.

Each beam that was eventually repaired went through two series of loadings. Before each beam was repaired, it was loaded to 3 kips (13.3 kN) in order to initiate cracking. The 3-kip load produced the kind of working stress to which a parking-garage beam would likely be exposed. The first set of beams tested was loaded with several cycles of this working-stress load, but cycles other than the first failed to increase the size of any cracks. For this reason, only one cycle of loading was used thereafter. After each beam had been loaded in this way, it was tested to failure. Catastrophic failure was avoided in all except three situations. Beams intended for repair were then removed, repaired, and retested to ascertain the maximum load-carrying capacity of the restored beam. Each beam was loaded until the concrete failed (usually in the compression zone). The beam was considered to have failed when it would accept no more load.

INSTRUMENTATION

To determine the effect of the different beam types (the nature and amount of simulated deterioration) on the reinforcing steel, strain gages were attached to one reinforcing bar on each beam in several locations. The locations of the gages were selected on the basis of data obtained from the report by Bryars (1). A minimum of two gages per altered beam was used in locations that were thought to be the most critical (see Figure 4).

The strain gage used was the Baldwin-Lima-Hamilton type AB-19 bakelite gage. This is not normally a desirable gage for strain measurements of reinforcing bars because of the involved application process, but an abundance of these gages was readily available. The application process was tested and proved to be satisfactory. After applying the gages to the reinforcing rods, the gages were waterproofed and padded to reduce the risk of damage during the concrete placement. The V/E-20A digital strain gage indicator, equipped with the V/E-25 scan controller, the V/E-21A switch, balance, and calibration module, and the V/E-22B printer, was used to read, scan, and print the numerous strain measurements. These instruments are manufactured by Vishay Instruments of Malvern, Pennsylvania.

RESULTS AND DISCUSSION

The testing procedures used over the two-year span are considered quite successful (1-3). Results were reproducible, and numerical values over the two years were quite consistent and well within the magnitudes of error inherent in any testing procedure. Of equal importance was the success of the techniques used to simulate the deteriorated beam. Again, test results were consistent and acceptable. All test results are summarized in Table 1, which contains both experimental results and relevant calculations based thereon.

There are two main areas of importance that must be presented in these results. The first, of course, is the set of numerical test results ob-

Table 1. Experimental results.

Loss of Cover (ft)	Loss of Bond (ft)	Repaired: Without Bonding Agent	Repaired: With Bonding Agent	Test Results: Maximum Load (kip)	Test Results: Maximum Moment (kip-ft)	Test Results: f'_c (ksi)	Test Results: f_y (ksi)	Coefficient of Strength[a]: Loss	Coefficient of Strength[a]: Recovery
0	0	-	-	10.5	21.55	7.0	64	-	-
0	0	-	-	11.5	23.60	6.2	65	-	-
0	0	-	-	11.0	22.58	6.2	65	-	-
0	0	-	-	18.0	22.74	6.8	63	-	-
0	0	-	-	17.5	22.10	6.8	63	-	-
1	0	-	-	10.9	22.34	7.0	64	0.991	-
1	0	Yes	-	9.5	19.50	7.0	64	-	0.864
1	0	-	Yes	11.0	22.58	7.0	64	-	1.000
1	1	-	-	11.0	22.58	6.2	65	1.000	-
1	1	-	-	10.8	22.17	7.0	64	0.982	-
1	1	Yes	-	9.5	19.50	7.0	64	-	0.864
1	1	-	Yes	11.0	22.58	7.0	64	-	1.000
1	1	-	-	11.0	22.58	6.2	65	1.000	-
2	0	-	-	10.4	21.35	7.0	64	0.945	-
2	0	Yes	-	9.5	19.50	7.0	64	-	0.864
2	0	-	Yes	11.0	22.58	7.0	64	-	1.000
2	2	-	-	11.0	22.58	6.2	65	1.000	-
2	2	-	-	11.0	22.58	6.2	65	1.000	-
2	2	-	-	10.0	20.52	7.0	64	0.909	-
2	2	Yes	-	10.0	20.52	7.0	64	-	0.909
2	2	-	Yes	11.0	22.58	7.0	64	-	1.000
3	0	-	-	9.5	19.50	7.0	64	0.864	-
3	0	Yes	-	9.5	19.50	7.0	64	-	0.864
3	0	-	Yes	11.0	22.58	7.0	64	-	1.000
3	3	-	-	11.0	22.58	6.2	65	1.000	-
3	3	-	-	9.9	20.30	7.0	64	0.900	-
3	3	Yes	-	9.5	19.50	7.0	64	-	0.864
3	3	-	Yes	10.5	21.55	7.0	64	-	0.955
4	0	-	-	18.0	22.73	6.8	63	1.000	-
4	0	-	-	17.5	22.10	6.8	63	0.986	-
5	0	-	-	19.0	24.00	6.8	63	1.000	-
5	0	-	-	19.0	24.00	6.8	63	1.000	-
6	0	-	-	18.0	22.74	6.8	63	1.000	-
6	0	-	-	19.5	24.63	6.8	63	1.000	-
4	4	-	-	17.0	21.47	6.8	63	0.958	-
4	4	-	-	16.0	20.21	6.8	63	0.901	-
5	5	-	-	15.5	19.58	6.8	63	0.873	-
5	5	-	-	14.5	18.32	6.8	63	0.817	-
6	6	-	-	14.0	17.68	6.8	63	0.789	-
6	6	-	-	15.5	19.58	6.8	63	0.873	-

Note: 1 kip = 4.45 kN; 1 kip-ft = 1.356 kN·m; 1 ksi = 6.89 MPa.

[a]Altered Pu/original Pu.

tained. One of the main objectives of this project has been achieved in that quantitative results have been obtained that describe the percentage of strength lost or gained by simulated deterioration and subsequent repair. The second area of importance is that of the behavior of the beams as indicated by the crack patterns on the beam. The crack pattern produced by a particular beam has proved to be predictable, reliable, and directly related to the amount and type of deterioration as well as to the nature of the repair used. Figures 5-7 show the typical crack patterns.

To describe the relative strength of these 40 beams, we have selected the ratio of the experimental load value, for various situations, to the load capacity of the control beams. This ratio is referred to as the coefficient of strength (CS). A CS value of unity indicates perfect agreement between the tested beam and the control beam--in other words, no loss of strength or full recovery of strength as the case may be.

Table 1 gives the loss of strength for deteriorated beams as well as the recovery after subsequent repair. It is significant to note that the greatest loss of strength occurred for the beam with no cover and no bond over a distance of 6 ft (1.8 m). This was not unanticipated. The CS value was 0.789. In other words, even when the cover and bond were prevented for as much as 63 percent of the span, only 21 percent of the member's strength was lost! It is also of significance to note that 99 percent

Figure 5. Basic crack patterns.

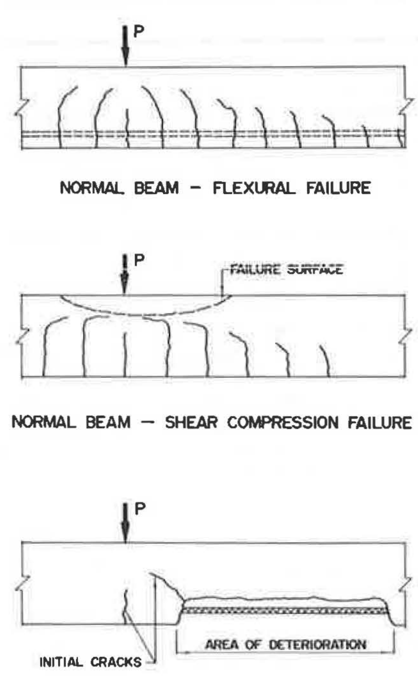


Figure 6. Crack patterns of deteriorated beams.

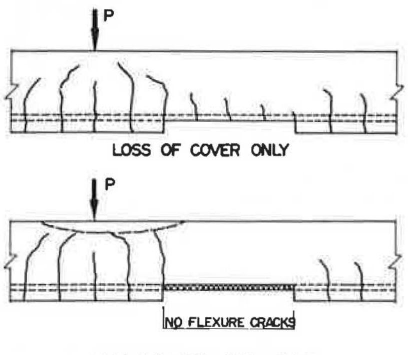
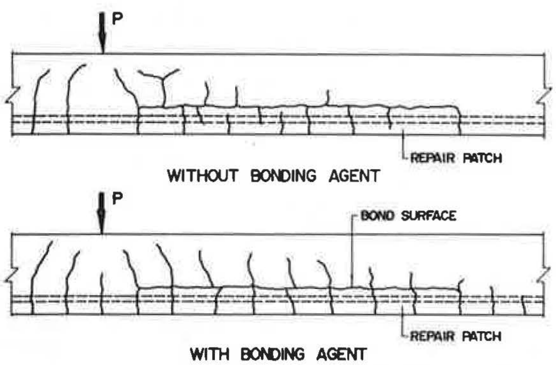


Figure 7. Crack patterns of repaired beams.



(CS = 0.991) of the strength was recovered when beams subjected to losses up to 3 ft (0.9 m) (31.5 percent of the span) were repaired. In the short term, then, the beam loses little strength and can be completely repaired.

At the point where flexural bond loss begins, all of the tensile stress is instantaneously directed into the reinforcing steel, which results in stress concentration. Stress concentration is demonstrated by the locations of initial yielding of steel and initial cracking of concrete at the reentrant corner (see Figure 5). When bond between the concrete and steel is lost, there is nothing to reduce the tensile stress in the steel. Theoretically, the stress in the reinforcing steel is constant in the region of bond loss; however, strains in the test beams indicate that stress in the region of bond loss is not constant. This apparently results from the mechanical bond that remains between the concrete and the unbonded steel. The magnitude of tensile stress is increased in the region of bond loss, and it cannot be redistributed through bond stress. This results in premature yielding of the reinforcing steel in the region of bond loss.

Figures 5-7 describe the behavior of the beams under action of loading. The crack patterns are indicative of the distress being felt by the particular beam.

The behavior of beams with loss of cover and/or bond was radically different from that of the control beams. Figure 6 shows the crack pattern of a beam with loss of cover only. It behaves essentially as a control beam and has the same crack pattern as the control beam. Even though further deterioration would certainly occur in the field under service conditions, it is apparent that there is sufficient bond on the bar, when only cover is "removed", for the beam to behave normally for short-term loading. Values of CS were consistently above 0.864. However, for the beam without cover and without bond, there is a radical change in behavior and a greater decrease in strength.

For such beams without bond, the CS value varied from a high of 1.000 to a low of 0.789 for bond loss up to 6 ft (1.8 m) (63 percent of the span). Along with this loss of strength, a radical crack pattern was apparent. The first crack appeared at about 50 percent of service loading, which corresponded to a concentrated loading of 1.5 kips (6.7 kN) for the beams loaded 1.5 ft (0.46 m) from the left support. This crack was quite wide at its origin, was singular, and extended to within 2 in (5 cm) of the top of the beam. The crack width was dramatic in every test. At higher load levels, additional flexural cracks formed under the load, as shown in Figures 5 and 6. As the load approached maximum for the beam, an additional horizontal crack opened at the top of the original crack and at right angles to it. This crack propagated toward the load and toward the center of the beam. This "tree-shaped" crack always occurred at the onset of failure.

Because of the lack of composite action between the concrete and the steel in beams with no bond, there were no flexural cracks in the region of simulated deterioration. The mechanics of this behavior are shown in Figure 3. The absence of cracking also indicated that the concrete was generally in compression. Although it is not the purpose of this paper to perform an in-depth analysis of the structural mechanics of the deteriorated beam, it is of interest to note that there is a combination of effects present here. They could be described as a combination of tied-arch behavior with bending. Certainly, the actual behavior is also influenced by the nature of the loading. In this case, there was no loading over the region of no bond. If such loading existed, the crack pattern would certainly reflect this change of internal stresses.

In some of the beams where there was loss of cover and bond, horizontal cracks formed at the level of the reinforcing steel at the far end of the test zone. This type of crack usually indicates a loss of bond between the concrete and steel as the steel begins to pull out of the concrete. The localized bond failure was due to a stress concentration at the point where the steel bars reentered the concrete. This stress concentration is a result of the lack of composite action between the concrete and steel in the test zone. As Figure 3 indicates, in a normal beam flexural bond stress acts to reduce the tensile stress in the steel by transferring stress to the concrete (9). When bond between the concrete and steel is lost, no bond stress exists to reduce the tensile stress in the steel. Theoretically, the tensile force in the exposed reinforcing steel is constant. This neglects a possible frictional effect. Although the strains measured in the exposed bars were not constant, they were higher than those measured in the control beams and beams with loss of cover only. Thus, at the near end of the test zone, the exposed portion of the steel was more highly stressed than the portion that was bonded to the concrete; a stress concentration existed at this point. Figure 5 shows typical crack patterns observed in beams with loss of bond.

In all the beams with loss of cover and bond, the strain gage near the reentrant corner was the first to reach yield. This was undoubtedly due to a large stress concentration at this point. Just inside the reentrant corner, where flexural bond is provided between the concrete and steel, a portion of the tensile stress in the steel is transferred to the concrete. At the point where flexural bond loss begins, all of the tensile stress is instantaneously directed into the reinforcing steel so that there is a severe stress concentration at this point. Figures 5 and 6 indicate the resultant crack pattern.

One of the most dramatic aspects of the behavior of the beams with loss of cover and bond was the ultimate failure. The failure was a very sudden shear-compression failure of the concrete under load along the surface defined by the horizontal crack described earlier (Figures 5 and 6). The deflections of the beams with loss of bond were linear and nearly identical to those of the control beam up to the failure load. At failure, the beams with loss of bond showed only a slight increase in deflection. The slow, ductile failure exhibited by the control beams was not shown by those with loss of bond. One beam showed a particularly dramatic failure. As the last increment of load was applied, the beam literally exploded as the concrete in the compression zone flew off the beam. This sudden release of energy split the beam longitudinally for almost the entire length of the test zone.

CONCLUSIONS

The following summary conclusions may be made as a result of the project described in this paper:

1. Regardless of the length involved, the loss of no more than the cover to the reinforcing steel has little short-term effect on the behavior and strength of the beam. As long as substantial bond remained between the concrete and that part of the bar surrounded by concrete, the beams tested showed no practical reduction in strength. It should be noted that, in actual service conditions, it is unlikely that the loss of cover would occur without some loss of bond also. It is also important to note that this type of deterioration is a continuing process unless specifically arrested and repaired; therefore, there is a time effect on exposed steel and concrete that has not yet been included in this project.
2. The loss of both cover and flexural bond significantly alters the behavior of the reinforced beam and substantially lowers its strength. This reduction in strength increases as the length of bond loss increases. The decrease in strength, as well as the change in behavior, is due to the loss of composite action inherent in conventional reinforced concrete beams. The type of failure and the crack pattern preceding that failure change; such failures are sudden and of the shear-compression type. The behavior over the region of the beam that had no cover and no flexural bond is similar to that of a tied arch with secondary effects.
3. When beams that have lost cover are repaired, the rehabilitation is complete. In these laboratory tests, without the use of the bonding agent, more than 86 percent of the original strength was recovered. When the bonding agent was used, the strength recovery was 100 percent in all beams tested. These conclusions hold for simulated deterioration 1, 2, and 3 ft (0.3, 0.6, and 0.9 m) in length. Repairs have not yet been tested for deterioration greater than 3 ft, which is 31.6 percent of the span.
4. When beams previously subjected to a loss of both cover and flexural bond are repaired, the strength recovery and behavior under load depend on the use of a bonding agent. The strength recovery of beams without a bonding agent averaged 88 percent of the original beam, and the minimum value was 86 percent. The cracking pattern under increasing load showed a lack of adhesion between the patch and the original concrete. When a bonding agent was used, the recovery was nearly complete: The average was 99 percent of the original beam, and the minimum test result was 96 percent. The cracking pattern in this case was similar to that in the normal beams (Figures 5 and 7).

RECOMMENDATIONS

The investigation described in this paper is ongoing. In brief, the following features and variables must be incorporated into the overall project:

1. Partial loss of flexural bond with respect to the diameter of the reinforcing bar must be included in the testing phases.
2. The loading system should be altered so as to more closely represent uniform loading and/or truck loading, as is more typical of service conditions.
3. Reinforcing steel must be altered to represent loss of shape and/or area, as would occur by means of actual corrosion over an extended period of time.
4. Alternative methods of repair, with and without a bonding agent, should be considered and tested. In such a procedure, beams must be repaired without physically inverting the beam, as was done for convenience in the laboratory procedures of this project.
5. If possible, the conclusions derived and the results measured should be verified by some amount of field testing on real structures.

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Abridgment

Repair of Torsionally Inadequate Concrete Beams by Use of Adhesively Bonded Plates

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The results of a study conducted to determine the feasibility of repairing torsionally inadequate reinforced concrete beams by using externally bonded steel plates are presented. The test program consisted of three sets of beams. Set A consisted of beams that had failed under torsional loading. Set B consisted of the same beams that failed as set A but were repaired by using epoxy adhesive to bond plates to the sides of the beams in the exterior thirds of the beam span. For set C, the plates were bonded to the sides of the beams before any testing. In the repaired beams, the external plates not only restored the integrity of the failed members but also increased the load-carrying capacity by 20 percent. For equal loads, the repaired beams had deflections approximately 6 percent less than those of the original beams. Set C, the control set, had 20 percent more carrying capacity than the concrete beams and also had deflections 20 percent lower. However, the plate configuration used did not force the members to fail in a flexural mode. The repair method was shown to be feasible, but more research is recommended to determine the best plate configuration and plate size.

The use of externally bonded steel plates to strengthen reinforced concrete beams is not new. Fleming and King (1) in South Africa have used bonded plates in the tension zone of concrete beams. Irwin (2) in the United Kingdom has done similar work. In the United States, at least one bridge in California has been strengthened by Byrd, Tallamy, MacDonald, and Lewis. However, all of these applications have been directed toward increasing the flexural capacity of the members.

Until this time, no work has been done on increasing the torsional capacity of reinforced concrete members. One principal type of structure that exhibits torsionally inadequate spandrel beams is the parking garage. Typically, the facade of a garage with several bays will be surrounded by spandrel beams, and very heavy loads, such as automobiles, will be parked at some distance from the spandrel beam. This loading produces a torsional moment, which in turn produces torsional stresses in the spandrel beam. Depending on the location in the member, these torsional stresses can add to the shear stresses.

Most prior testing of torsionally loaded members has been for the analysis and design of new members, but very little information is available on the repair of torsionally inadequate members.

OBJECTIVE OF THE PROGRAM

The objective of this study was to propose a method of repair for torsionally inadequate reinforced concrete spandrel beams. The proposed solution was to adhesively bond steel plates to the vertical faces of the beams in the high-shear and high-torsion regions by using an epoxy adhesive. The intent was to entirely control the torsional cracking of the member and shift the mode of failure from sudden torsional failure to a more conventional flexural failure with a forewarning of collapse.

DESIGN OF THE EXPERIMENT

The reinforced concrete members were designed in accordance with the American Concrete Institute (ACI) code and commentary (3,4). The relation between the test sections and the actual building framing is shown in Figure 1. The test members were modeled to represent a typical spandrel beam located between two adjacent columns and supporting a floor beam at each of the third points. In an actual building, a monolithically cast slab would complete the system. However, in order to better study the pattern of crack formation, the slab was omitted from the members in the testing program. The beams were specifically designed so that the mode of failure would be in torsion. For the experiment, the model was mounted so that it performed as a pinned connection with respect to flexure and normal shear and as fixed with respect to torsion.

The spandrel beams tested were 4x8 in (102x203 mm) in cross section and 9.5 ft (2.9 m) long. Four no. 6 bars of grade 60 steel were used as longitudinal reinforcement. The stirrups were 0.187 in (5 mm) of grade 40 steel. The steel plates, which

Figure 1. Test specimen.