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Rehabilitation of Beams with Externally Bonded Reinforcement

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Abstract

When the working space and the time of disturbance on the use of structural system become the constraints of the rehabilitation, externally bonded reinforcement may be considered as the most favourable rehabilitation technique. The experimental program reported here was carried out on five singly reinforced, rectangular beams having constant (100x150x2700mm) dimensions, without shear reinforcement. Concrete strength was in the range of 30-35 MPa. Loading was monotonically increasing point load applied at mid-span. Virgin specimens were first loaded to yield and unloaded. Then steel plates were bonded by epoxy on the tension side of the beams and re-loaded to failure. From the test results, it can be concluded that the application of externally bonded plate on the tension side of the reinforced concrete beams significantly increases the stiffness and yield strength of the section. The capacity calculations done by code formulations are valid for repaired beams. Although the width to thickness ratio of the steel plates were as low as 9, no spring type of failure was observed. The decrease in ductility in steel plate rehabilitated beams is not the characteristic of this particular repair technique, but it is because of the virtue of higher tension steel percentage (ρ/ρ_b) values reached at the end of rehabilitation.

Introduction

It is sometimes necessary to enhance the serviceability performance or ultimate load carrying capacity of certain structural members in existing concrete structures. Among the structural members, the flexural rehabilitation of reinforced concrete floor-beams and bridge-girders is the scope of the present study.

The need for rehabilitation of reinforced concrete structural elements may be due to the following reasons: Change in the use of structural system (e.g. from a residential building to a warehouse) or in the code specifications (e.g. higher load factors or lower material constants); Design deficiencies or construction errors threatening the integrity of the

structural system; Damage caused by high intensity external loadings such as earthquakes and wind loads.

There are several methods reported in the beam rehabilitation such as: removing the crushed and/or cracked concrete section and replacing it with a new concrete either same or higher strength; external post-tensioning on the tension side; adding a second layer of concrete and reinforcement through in place casting; epoxy injection into the cracks; and the addition of epoxy bonded steel plates to the tension flange of the structural element. The rehabilitation of floor-beams or bridge-girders via epoxy bonded external flexural reinforcement will be experimentally evaluated in this report.

The "external bonding of reinforcement" method, used in structural member rehabilitation was reported to be in use for over three decades. Also, it has been found to produce effective and economical solutions to particular problems. Although the technique was reported to be introduced in France in the late 1960's, where the first reported application was a bridge girder strengthening, the most striking number of application was reported to be in Japan, where up to 1975 well over 200 highway bridges were strengthened to accommodate the large increases in loading due to heavy goods traffic [1].

Data describing the post-yielding behaviour of rehabilitated beams is very limited. The common idea among the researchers [1,2,3] is that the application of external reinforcement on beams increases the stiffness and decreases the ductility, which is a completely expected behaviour due to the increase in the reinforcement ratio. The externally bonded reinforcement need not to be in the form of steel plates. Saadetmanesh et.al. [2] reported the test results of Glass Fiber Reinforced Plastics used instead of steel plates, pointing out the danger of corrosion at the epoxy steel plate interface. Basunbul et.al. [3] used four methods (epoxy injection, ferrocement layer, combination of these two, and steel plate bonding) in rehabilitation of 36 beams, and concluded that the level of ultimate load capacity increase was approximately same for all levels of initial damage (from cracking to yielding) in steel plate bonded specimens, but differs in other repair models.

Research Significance

"Epoxy bonded steel plate" type of rehabilitation of floor-beams or bridge-girders, which are damaged due to over-loadings, is thought to be a positive input to the construction economy. The rehabilitation of existing structures, in case of damage or for upgrading purposes, is quicker and sometimes cheaper than demolishing and reconstructing the structure. In the present study, the beams which are repaired by epoxy bonded steel plates are examined to clarify their load carrying capacity calculations, and ductility predictions. Time dependent deflection characteristics of the rehabilitated beams, behaviour of concrete to epoxy bond or steel to epoxy bond over time and/or under sub-zero or elevated temperatures, and the effect of chemical attacks to the epoxy bond is out of the scope of the present study.

Problems Reported and Rehabilitation Steps

Although the principles of this strengthening technique are fairly simple, the following are some causes of the reported problems in the integral behaviour of the rehabilitated specimens: 1. presence of water penetration to the glue line, 2. presence of de-icing salts, 3. elevated temperatures (above 50-60 Celsius), 4. having not enough effective anchorage area, 5. thin steel plates ($t<3$ mm), 6. inappropriate w/t ratio ($40<w/t<60$ Given in Ref.1), 7. higher tension reinforcement ratios, 8. poor workmanship.

For applications involving the "external bonding of reinforcement", the design and rehabilitation steps are very important. The reinforcement ratio together with the bonded plate must be checked carefully to prevent a brittle beam failure. At the same time, the surface preparation during the rehabilitation is an important step and the following must be satisfied: 1. concrete surface that is bonded must be sound, 2. concrete and steel surfaces must be free of salts, 3. steel plate must be clean (no oil, mill-scale or rust), 4. concrete surface should be roughened, 5. surfaces should be dry and free from dust.

Description of Test Specimens

The test specimens were designed to represent a flexure dominated failure. In total, 5 specimens were tested in which the shear span to depth ratio was approximately constant at a value of 10. The beams had 100x150mm nominal x-sectional dimensions with two ϕ=9.2 mm longitudinal reinforcement located at tension side (Table-1). Neither compression nor shear reinforcement were used in these specimens.

Natural river sand and round river gravel were used as aggregate in concrete mix design. Sieve analysis tests were conducted on the ingredients and the final mixture with a fineness modulus of 4.35 was obtained. Mixture proportioning for concrete is given in Table-2.

The ribbed longitudinal reinforcement and epoxy bonded steel plates had definite yield plateau (Tables 3 and 4). The overall length of the specimens were 3.0m and the support-to-support length of the specimens were 2.7m. Longitudinal reinforcement ratios were around 20% to 40% and 40% to 65% of balance values for virgin and repaired specimens, respectively.

Testing of Specimens

The beam specimens were tested under a single point load applied at mid-span, with a 33 tons capacity Riehle Universal Testing Machine. The loading system of this testing machine was screw type, therefore deformation controlled type of loading was achieved to some extend. Concrete cylinders (150x300mm) were loaded directly to failure, while full stress-strain curves were obtained for the re-bars and plates.

Five dial gauges, three of which were electronic, were used to measure the deflections of the beams. Electronic dial gauges were used to measure the mid-span deflection and curvature at the mid-span of the specimens. Two eye-reading dial-gauges were used to

measure the support settlements at the two supports. The single point load, which was applied at the mid-span, was measured by using an electrical load cell.

The specimens were white-washed before the test in order to follow the cracking pattern easily. During the test the cracking pattern was marked and the specimens were photographed during and after the test.

Method of Repairing

Since the repair technique of the beams involve only epoxy bonded steel plate, no attempt was made to fill the flexural cracks with epoxy. The concrete surface (tension side of the beam) was sound enough and no chipping-off cracked concrete was done. The surface was cleaned by a steel brush before the application of epoxy layer. The steel plates with pre-determined width and thickness were cleaned and brushed to have an oil and dust free surface. Both of the surfaces were dry at the time of epoxy application.

The two components of the epoxy was hand mixed by rotating towards the same direction throughout the mixing process. This minimizes air trapping. The mixing and application of the epoxy were done within the specified pot-time and the joint-open-time. Epoxy layer was applied both on steel plate and concrete surface. After the application of epoxy layer, steel plates were placed onto the bottom of the beam specimens and fixed with grips in order to have a uniform pressure on the bond surface. The average thickness of the epoxy layer was around 1.5 mm. Specified compressive strength of the epoxy was 80 MPa with a unit weight of 1.34 g/cm^3.

Test Observations

The repair technique of the beams involved only the bonding of steel plates via epoxy. The experimental program reported here has got two phases; the first being the test of virgin beams and the second being the test of repaired specimens.

Specimens which had a nominal length of 3.0m were simply supported having a support-to-support length of 2.7m. One of the supports was a roller support, while the other was a hinge support. At the location of supports and point load, 50mm width, 20mm thick steel plates were used to protect the local concrete from crushing. Those plates were attached to the specimen by gypsum in order to satisfy a uniform stressing at the supports and at the location of single point loading. The point load was applied via a roller support to avoid any externally applied moment. The point load was measured by an electrical load cell. The curvature measurements were done on one face of the beams. Except the two dial gauges at the supports the rest of the data readings were done by an electronic data acquisition system. For all the specimens, except SOB-1, the post yielding portion of the Moment-Curvature diagram was carefully recorded. In specimen SOB-1 the curvaturemeter was disconnected from the specimen at the point of yielding of the section. This was due to the lack of experience on the integral behaviour of beams rehabilitated via epoxy bonded external flexural reinforcement.

The virgin specimens were loaded up right to the yielding of longitudinal reinforcement. This point was carefully controlled from the load-deformation graphics, since no strain

gauges were used. The loading was deformation controlled via a screw type loading system. All the virgin specimens bounced back when the load was taken off. The repair was made afterwards. The crack distribution was: a main crack at the maximum moment point, and some secondary flexural cracks evenly distributed around that point. No shear cracking was observed in any of the virgin specimens.

Exactly the same electronic data reading system was used for repaired specimens at the same locations. The length of the steel plate was nearly support-to-support (2.4m). Therefore, no bond failure was observed. On the other hand, for specimen SOB-1, after the ultimate load level, all the electronic equipment were disconnected and specimen was loaded well beyond the ultimate; as a result, after the test, the steel plate was being taken off rather easily compared to other specimens indicating a bond deterioration. During loading, the cracks which already exist due to the previous loading started to widen first, after that evenly spaced new hairy cracks occurred at the tension face of the specimen. In any of the five repaired specimens, which had an average steel plate width-to-thickness ratio (w/t) of 9-18, no spring type failure of the bonded reinforcement was observed. Since the steel plate was spanning nearly from support to support, there were no cracks in concrete concentrated at the end of the steel plate due to stress concentration.

Discussion of Test Results

Load-Midspan deflection and Moment-Curvature diagrams (Figures 1 and 2) clearly show that the externally bonded reinforcement causes a noticeable improvement in the mechanical behaviour of repaired beams over the virgin ones (Table 5). In those diagrams, although the concrete strength was ranging from 26 to 41 MPa in repaired specimens, no attempt was made to normalize the curves. Comparing the virgin and repaired specimens it is seen that the initial stiffness of the repaired beams are much higher than that of the virgin ones. Moreover, this rehabilitation technique carries the yield load to a higher value.

Bonded plate width/thickness (w/t) values were approximately 9 and 18 for the test beams. Although w/t>40 is given in the literature [1] in order to prevent a spring type failure, no such type of failure was observed with these very low w/t values.

In the published research[1,2,3] in this field, it is agreed that the application of externally bonded reinforcement rapidly decreases the ductility of the structural elements; and most of the times this conclusion is given as if it is not expected. If we analyze Figure 3 [4] which includes the test results of many different researchers, it is seen that the ductility of a section is closely related to the ρ/ρ_b value and the specimens of the current research is in the same trend of the previously reported non-rehabilitated specimens. This indicates that the decrease in ductility is from the virtue of an increase in ρ/ρ_b and not only of the existence of externally bonded plate.

Another aspect controlled during the analysis of the test results was the applicability of the current design equations to the beams repaired via externally bonded reinforcement. The calculated capacities for virgin and repaired specimens are in good agreement with the experimental values. Capacity calculations were done by the conventional ultimate strength design method given in TS-500 [5], and ACI 318-89 [6].

Conclusions

1- The failure for the repaired specimens were started with the widening of the existing initial flexural cracks occurred in the test of the virgin specimens.
2- All the specimens underwent a ductile failure; the ultimate failure was because of crushing of compression concrete due to increasing curvatures and decreasing compression zone depth.
3- No spring type failure occurred in the bonded plates although the w/t was decreased down as low as 9.
4- Application of externally bonded reinforcement increased the stiffness and yield moment as expected.
5- The ductility decrease in repaired specimens is from the virtue of increase in ρ/ρ_b; not only of the existence of externally bonded plate.
6- Current design codes are good enough to calculate the capacities of the beams which are repaired for flexural upgrading purposes.

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TABLE 1 - Specimen Characteristics

Specimen	f_c',v MPa	f_c',r MPa	b mm	h mm	d mm
SOB-1	33.0	41.0	104	153	139
SOB-2	19.0	26.0	109	151	137
SOB-3	30.5	35.0	101	146	132
SOB-4	31.0	35.0	102	151	137
SOB-5	30.5	35.0	99	150	136

TABLE 2 - Mix Design Proportions

Ingredient	Amount
River Gravel	1031 kg
River Sand	898 kg
Water	180 kg
Cement	300 kg
water/cement	0.6
Unit Weight	2450 kg/m^3

TABLE 3 - Longitudinal Reinforcement Characteristics

Specimen	ϕ mm	ε_{sy}	f_{sy} MPa	ε_{ssh}	f_{su} MPa	ε_{su}
SOB-1	2x9.2	0.0019	362	0.0305	450	0.0889
SOB-2	2x9.2	0.0021	362	0.0409	450	0.0921
SOB-3	2x9.2	0.0021	373	0.0302	490	0.0902
SOB-4	2x9.2	0.0023	391	0.0302	504	0.0904
SOB-5	2x9.2	0.0021	373	0.0350	484	0.0913

TABLE 4 - Repair Plate Characteristics

Specimen	L mm	t mm	w mm	ε_{py}	f_{py} MPa	ε_{psh}	f_{pu} MPa	ε_{pu}
SOB-1	2400	4.3	70.0	0.0021	276	0.0210	350	0.121
SOB-2			70.0					
SOB-3			34.7					
SOB-4		3.8	70.0	0.0025	250	0.0100	355	0.130
SOB-5			33.8					

TABLE 5 - Experimental Yield Moment Values

Specimen	SOB-1	SOB-2	SOB-3	SOB-4	SOB-5
My,v (kN.m)	5.29	6.25	6.97	6.43	6.62
My,v (exp/calc)	0.84	1.05	1.14	0.96	1.06
My,r (kN.m)	18.32	18.32	10.88	16.98	12.61
My,r (exp/calc)	1.09	1.21	1.01	1.16	1.18

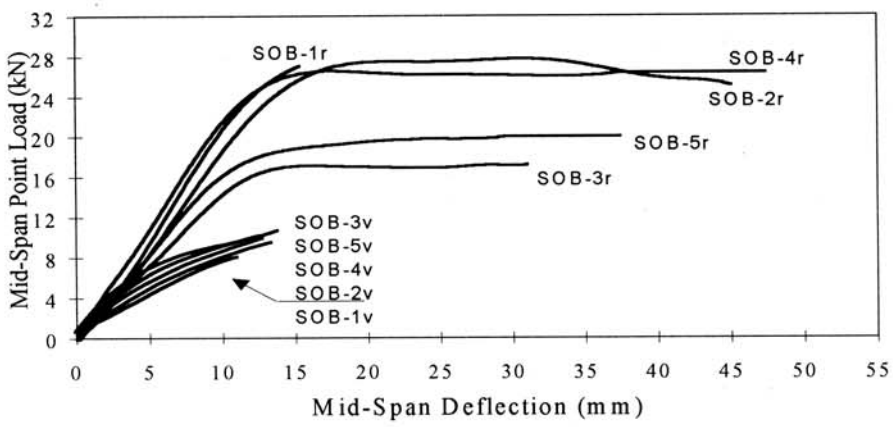

Figure 1 - Experimental Load-Deflection Behaviour at Mid-Span

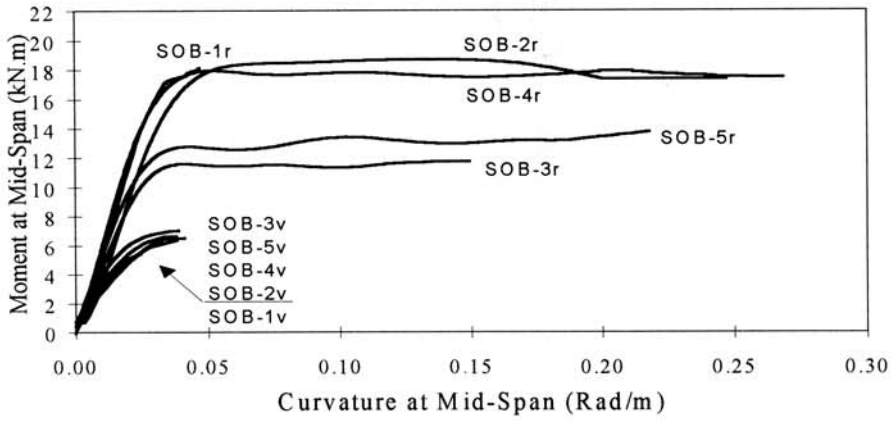

Figure 2 - Experimental Moment-Curvature Behaviour at Mid-Span

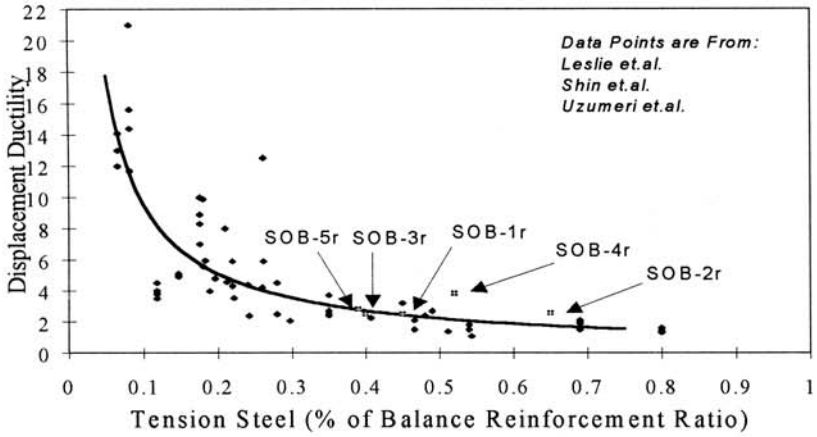

Figure 3 - Effect of Amount of Tension Steel on the Displacement Ductility [4]

Nomenclature:

ρ	: tension reinforcement ratio
ρ_b	: balance reinforcement ratio
f_c',v	: 150x300mm concrete cylinder compressive strength of virgin specimens
f_c',r	: 150x300mm concrete cylinder compressive strength of repaired specimens
b	: web width of the beam
h	: height of the beam x-section
d	: depth of the original tension reinforcement
ϕ	: diameter of tension steel
ε_{sy}	: yield strain of tension steel
f_{sy}	: yield stress of tension steel
ε_{ssh}	: strain at which strain-hardening of tension steel starts
f_{su}	: ultimate stress capacity of tension steel
ε_{su}	: ultimate strain capacity of tension steel
L	: length of steel plate
t	: thickness of steel plate
w	: width of steel plate
ε_{py}	: yield strain of steel plate
f_{py}	: yield stress of steel plate
ε_{psh}	: strain at which strain-hardening of steel plate starts
f_{pu}	: ultimate stress capacity of steel plate
ε_{pu}	: ultimate strain capacity of steel plate
My,v	: experimental yield strength of the virgin specimens
My,r	: experimental yield strength of the repaired specimens

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