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New Approach for Strengthening Structural Concrete Bridge Girders

Conception nouvelle pour le renforcement de poutres de pont en béton armé

Neues Konzept für die Verstärkung von Stahlbeton-Brückenträgern

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SUMMARY

This paper describes a design model, including a new detailing approach for the stirrups, which has been used in the strengthening of reinforced concrete beams, such as bridge girders. In the proposed approach a new concrete top overlay containing confinement stirrups is cast on top of existing girders. The confinement stirrups are welded to the stirrups in the girders near the support region in order to prevent an inter-laminar shear failure. The results of tests have shown that each of the girders strengthened using this design approach has acted as a single unit, failed in a ductile manner and achieved its full flexural capacity compared with similar girders which have been cast monolithically.

RÉSUMÉ

L'article présente le principe d'une nouvelle conception pour la consolidation et le renforcement des poutres en béton armé. Le principe de cette méthode consiste à couler sur la poutre existante une nouvelle dalle de béton armé. Afin de réduire les risques de cisaillement, les armatures de cette nouvelle dalle sont fixées par des étriers soudés sur les armatures des poutres existantes. Des essais ont démontré que les poutres ainsi renforcées se conduisaient comme un ensemble homogène répondant aux mêmes critères de solidité et flexibilité que des poutres ayant été réalisés de façon monolithique.

ZUSAMMENFASSUNG

Diese Studie erklärt eine konstruktive und errechnete Methode für Bügel, die für die Unterzüge in Stahlbetonbauten benutzt wird. In dieser Methode wurde eine obere Schicht von Beton geschützt, mit zusätzlichen Bügeln auf die alten Bügel geschweisst, in der Nähe von den prägende Stellen, wegen den Querkräften. Durch die Ergebnisse wurde festgestellt, dass jeder zusätzlich verstärkte Unterzug als Einheit wirkte, duktil brach und dieselbe plastische Verformung erreichte, wie monolithisch hergestellte Träger.

1- INTRODUCTION

A large number of existing reinforced concrete structures, e.g. bridge girders, have been designed to carry loads which are well below present day requirements with respect to load carrying capacity and ductility. Also, there are many structures which have been weakened as a result of earthquakes, incorrect design and detailing, or poor construction practice. Such structures need to have their strength and ductility enhanced to enable them to carry more demanding loading levels. There is an obvious requirement for the development of methods to strengthen existing structures. In this context, several techniques for repairing and strengthening of existing structures have already been investigated, however, there is little information available on their effectiveness[1]. These approaches have included the use of ferrocement, epoxy injection, plate-steel bonding, a combination of the above, concrete overlays and underlays, and post tensioning. These techniques have proved that they may result in the restoration and maintenance of the original strength of a structure but are unlikely to result in a significant increase in the original structural capacity. Several of these approaches have also proved somewhat difficult to implement in practice. Moreover, the majority of research work in this area has been limited to laboratory tests to examine the comparative behavior of structures repaired using different techniques. The main problem encountered in using these techniques was the occurrence of a brittle shear failure in the strengthened beams. It is believed that there is a need for a rigorous design approach to be developed which could be applied generally to structural concrete members.

In this paper the use of a flexure-shear interaction design model with a new approach for detailing of stirrups, which has already been verified for the design of beams with different cross sections [2,3], is described for strengthening of structural concrete girders.

2- FLEXURE-SHEAR INTERACTION DESIGN MODEL FOR STRENGTHENING OF BEAMS

2.1 Outline of the model

Conventional design methods for structural concrete beams treat shear and flexural actions separately although they occur simultaneously in practice. The design approach for shear relies on the transverse reinforcement at the ultimate limit state to resist the shear stress v_s in excess of that assumed to be carried by the concrete section v_c. Such an approach assumes that two types of mechanisms act simultaneously in the beam structure, namely, truss and beam/arch actions corresponding to the shear stresses. The assumption of the presence of two mutually exclusive mechanisms thus leads to the confusion in the current methods for shear design. On the other hand, it is known that[4] the ability of a structure to carry additional load when subjected to large deformations due to additional load depends primary on confinement of the compression concrete. In flexural as well as in diagonal failures collapse of a beam occurs as a result of spalling of the compression concrete in either the inclined leg or the horizontal leg regions of the Compression Force Path (CFP) as a result of the development of transverse tensile stresses in the region of the path[5]. In order to prevent the collapse of beams, the compression concrete in the path regions must be provided with confining stirrups as shown in Fig. 1. The inclined leg regions are provided with conventional full length stirrups and the horizontal leg regions are provided with short stirrups. This detailing approach adopts the same mechanism for resisting moment and shear, irrespective of the level of loading. When the concrete capacity is exceeded, the transverse reinforcement is intended to enhance the strength of concrete in the compression zone, which is the main element in the beam structure which resists the imposed loading (axial, shear and bending moment). To elaborate further on the proposed unified behavioral mechanism, consider the case where the shear span to depth ratio a/d is within Kani's Valley. The capacity of each leg region M1, due to the development of transverse tensile stresses in the compression region, is less than the full flexural capacity of the beams Mf.

These tensile stresses reduce the concrete compressive strength by an amount equivalent to Δf_c which will lead to a corresponding reduction in the flexural capacity of each leg of the beam by ΔM where ΔM=Mf-Ml. The full flexural capacity of the beam Mf can be restored by offsetting the reduction in the concrete compressive strength Δf_c. In this case this is achieved by utilizing the effect of confinement to enhance the concrete compressive strength by an equal amount i.e. Δf_c. After determining Δf_c, the confinement requirements and thus the amount of stirrups required to achieve this added strength is determined using Eqs. 1 to 3. The complete development and the experimental validation of the flexure-shear interaction design model for beams, including the determination of the leg capacity M1, have been reported elsewhere[3].

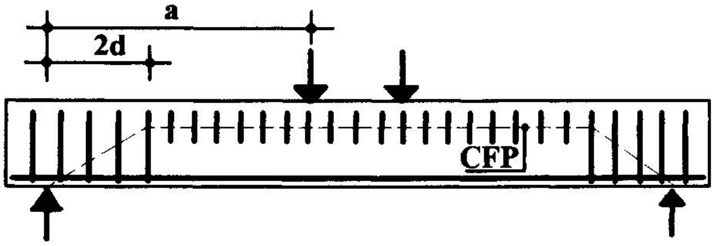

Fig. 1 Detailing arrangements for beams.

By definition

$$\Delta f_c = (K_s - 1) f_c \tag{1}$$

where K_s is the confinement enhancement factor[4]. Based on flexural theory

$$\Delta f_c / f_c = (Mf - M1)/Mf = \Delta M / Mf \tag{2}$$

The combination of Eqs. 1 and 2 can be expressed as follows:

$$K_s = \Delta M / Mf + 1 \tag{3}$$

The amount of confinement stirrups corresponding to K_s is found using a modified form[3] of the confinement model originally developed by Sheikh and Yeh[4].

2.2 The application of the model for strengthening of girders

The flexure-shear interaction design model can readily be adopted for the strengthening of girders. The detailing approach for the stirrups has a significant advantage over traditional approaches to detailing when applied to the maintenance and strengthening of existing structural concrete members. In the case of girders, a new concrete layer can be cast on top of the girders to act as a compression zone within the girder structure. The enhancement of the major parts of the girders, i.e. the horizontal leg region of the CFP, can easily be achieved since the top cast layers are only required to be provided with short stirrups which do not need to be extended into the existing part of the girders. The inclined leg regions do, however, require more careful consideration. The stirrups in the new concrete layer should be welded to the existing stirrups in the inclined leg regions thus avoiding the requirements to treat the overall depth of the section. Welding of the stirrups in the inclined leg regions would also prevent an inter-laminar shear failure.

3- TEST PROGRAM

3.1 Test beams

Twenty under-reinforced simply supported beams with different shear span to depth ratios a/d were included in a test program conducted at Birzeit University[6] for the verification of the proposed design approach for the strengthening of existing girders. All the test beams had a cross section of 150mm X 240mm, overall length of 2000mm, longitudinal reinforcement ratio ρ of 0.0075, and design concrete compressive strength f'_c of 20MPa. The test beams were divided into two types based on the design approach adopted as shown in Table 1 and Fig. 2. Traditional beams of Type T (beams T1, T2, T3, T4, and T5) were designed based on the ACI code of practice[7]. Strengthened beams of Type S (beams S1, S2, S3, S4, and S5) were cast in two stages. Beams of half of the overall depth were designed either based on the ACI code of practice to simulate existing beams in need of strengthening, or based on the proposed approach, whichever is critical. In this case, the upper horizontal leg of the stirrups were left exposed to facilitate welding. The remaining part of the beams i.e. the top cast overlay, was designed based on the proposed approach taking into consideration the whole section i.e. as if the beam had been cast monolithically. The stirrups in the top cast overlay and in the original beams were welded together over a distance equal to 2d from the support. This was to ensure the full interaction behavior between the upper and lower parts of the beams. The weld provided was sufficient to prevent an inter-laminar shear failure. It should be noted that two nominally identical beams were cast for each beam Type. High-strength deformed steel bars of 10mm diameter with f_y=420MPa were used for longitudinal reinforcement. The stirrups were fabricated from mild steel bars of 6mm diameter with f_y=280MPa.

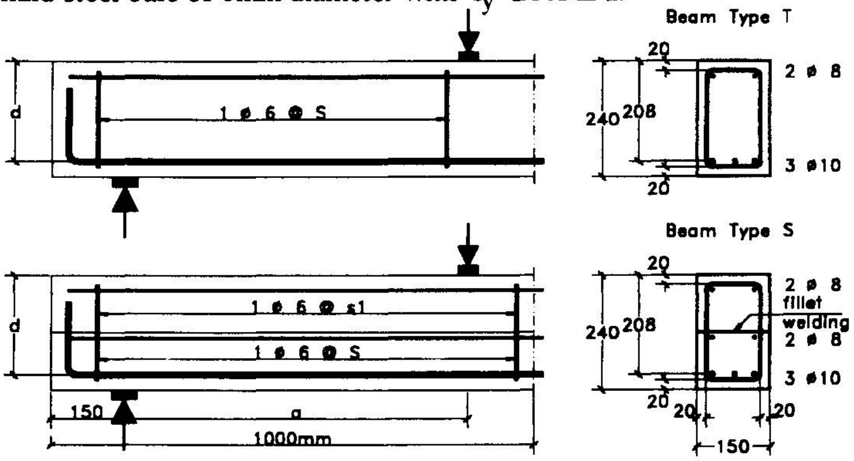

Fig. 2 Details of the test beams.

3.2 Test results

The measured and the calculated load carrying capacities of the beams are shown in Table 1. Fig. 3 shows typical load mid-span deflection curves for beams of Types T1, S1, T3, S3, T5, and S5. Fig. 4 shows a typical crack pattern and a typical strain diagram obtained from demec readings taken on the concrete surface for the strengthened beam Type S3 .

Beam type	a/d ratio	s mm	s1 mm	P1 kN	P2 kN	P1/P2
T1	3	50	-	37.5	30.2	1.24
S1	3	50	50	42.6	34.1	1.25
T2	3.25	75	-	34.1	27.9	1.22
S2	3.25	50	75	39.3	31.5	1.25
T3	3.5	75	-	29.7	25.9	1.15
S3	3.5	50	75	35.7	29.3	1.22
T4	3.75	75	-	28.9	24.2	1.19
S4	3.75	50	75	33.4	27.3	1.22
T5	4.0	100	-	28.2	22.4	1.26
S5	4.0	75	100	31.5	25.3	1.25

Table 1 Details and results of the test beams.

3.3 Mode of failure

All beams, regardless of the design approach used, failed in a flexural mode by spalling of the compression concrete in the region subjected to maximum bending moment as shown typically in Fig. 4.a for beam Type S3. Diagonal cracks developed in the shear span as an extension of the flexural cracks. All cracks proliferated and widened under increasing loads. In beams of Type S the short stirrups succeeded in preventing diagonal cracks from extending into the compression zone in the top overlay thus preventing diagonal failure in all strengthened beams. However, in beams of Type T the diagonal cracks bypassed the loading points and extended into the mid-span region, where stirrups had not been provided, thus entering into the compression concrete which spalled at the flexural failure. Very small horizontal cracks between the overlays and the original beams at a distance equal 2d from the support were found in the final crack pattern for some of the beams of Type S which had a/d ratios up to 3. Irrespective of these cracks, it is concluded that full interaction did develop between the overlays and the original beams. The concrete strain measurements, which were recorded on the sides of the beams, confirmed this conclusion. Typical strain measurements for the strengthened beam Type S3 shown in Fig. 4.b indicated that plane sections remained plane after bending. Furthermore, the crack widths and mid-span deflection measured at working load levels, which have been assumed at 65% of the ultimate load, satisfied the requirements of the serviceability limit state in the ACI code of practice.

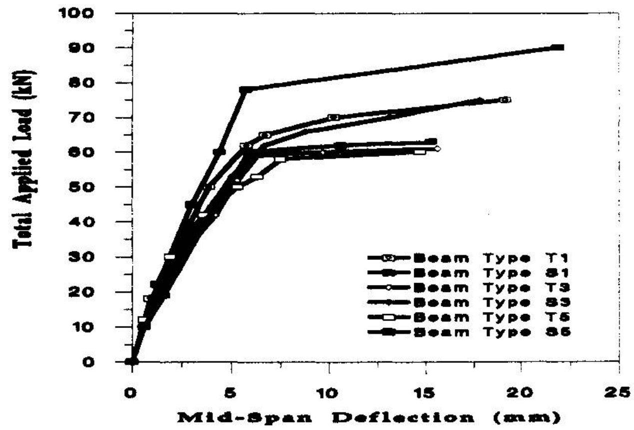

Fig. 3 Load mid-span deflection curves.

3.4 Load carrying capacity and ductility

All beams achieved their full flexural capacity and failed in a ductile manner which is a characteristic of under-reinforced beams as shown in Fig. 3. The load carrying capacity of the strengthened beams of Type S, regardless of the a/d ratios, were larger than the corresponding measured capacities of the traditionally detailed and cast beams of Types T as shown in Table 1. The increased strengths found in the strengthened beams were due to the effect of the reinforcing bars in the compression concrete in the original beams which behaved as tension reinforcement after the overlays had been added. The ratios of measured (P1) and calculated (P2) load carrying capacities (P1/P2) ranged from 1.22 to 1.25 and from 1.15 to 1.26 for beams of Types S and T respectively. The relatively large variation in the P1/P2 ratios (11%) for beams of Type T was due to the confinement being ignored in the design of the flexural capacity of the sections using the ACI code of practice. On the other hand, the variation in the P1/P2 ratios for beams of Type S was only 3% since they were designed using the flexure-shear interaction model. It is interesting to note that an increase in the load carrying capacity of up to 173% was achieved as a result of the strengthening approach adopted for these beams compared with the capacity of the original beams without addition of the confined overlays.

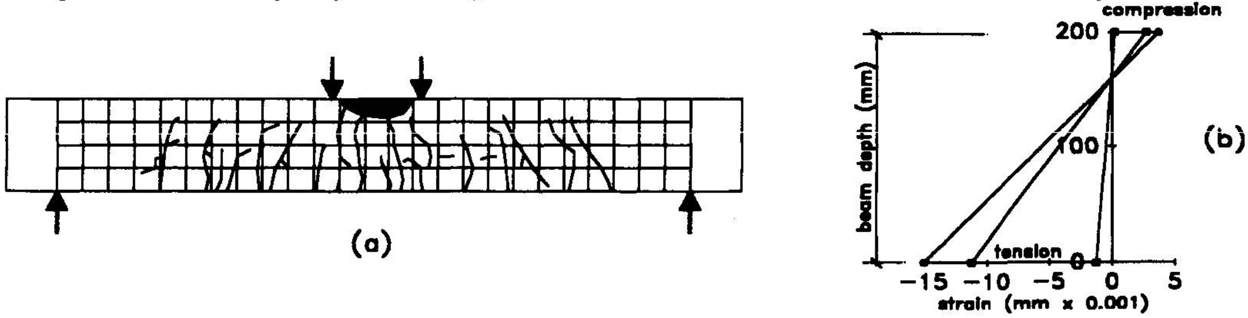

Fig. 4 Behavior of beam Type S3. a- Crack pattern. b- Strain diagram.

4- CONCLUSION

A flexure-shear interaction model has been used for the strengthening of existing beams. The beams were strengthened by casting confined concrete overlays on their upper surfaces. The inter laminar shear failure was prevented by welding the horizontal legs of the stirrups in the original beams to the adjacent legs of the stirrups in the overlays in the support regions. All of the strengthened beams failed in a ductile manner and achieved their full flexural capacities compared to similar beams which have been cast monolithically. An increase up to 173% of the capacity of the original beams was achieved using the proposed design approach. The variation between the calculated and measured capacities of the strengthened beams was 3%. The flexure-shear interaction design model has been confirmed to be suitable for use in the strengthening of existing reinforced concrete girders.

5- ACKNOWLEDGMENT

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