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Strengthening of Structures Using Epoxy Bonded Steel- or Fibre Reinforced Plastic Plates

Renforcement de structures par collage de plaques métalliques ou en plastique renforcé de fibres

Tragwerksverstärkung mittels geklebten Stahl- oder faserhaltigen Kunststoffplatten

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SUMMARY

This paper describes tests on concrete beams strengthened for shear using bonded plates of steel or GFRP and CFRP (Glass or Carbon Fibre Reinforced Plastic) materials as outer reinforcement. The results show that it is possible to achieve a good strengthening effect with either steel or CFRP plates. In the design stage, however, the actual strain in the concrete at fracture should be considered for best results.

RÉSUMÉ

L'auteur décrit divers essais effectués sur des poutres en béton, dont le comportement au cisaillement a été amélioré par le collage de plaques soit en acier, soit en matière plas-tique armée de fibres de verre ou de carbone, appliquées comme armature extérieure. Les résultats d'essais montrent qu'il est possible de parvenir à un excellent renforcement par utilisation de l'un de ces deux types de plaques. Au cours du dimensionnement, il faudrait toutefois prendre en considération les allongements à la rupture effectifs du béton.

ZUSAMMENFASSUNG

Es werden Versuche an Betonbalken beschrieben, die zur Verbesserung des Schubtragverhaltens mit geklebten Platten aus Stahl bzw. glas- oder kohlenstoffaserverstärktem Kunststoff (CFK) als äussere Bewehrung versehen wurden. Die Versuchsergebnisse zeigen, dass mit Stahl- oder CFK-Platten eine gute Verstärkungswirkung erreicht werden kann. Bei der Bemessung jedoch sollten die tatsächlichen Betonbruchdehnungen verwendet werden.

1. INTRODUCTION

1.1 General

As most of us know concrete is a building material with a high compressive strength but poor tensile strength. A beam without any form of reinforcement will crack and fail with a relative small load. In most cases the failure happens suddenly and in a disastrous manner. The most common way to reinforce concrete structures is therefore to cast steel reinforcing bars into the fresh concrete during the building stage. The reinforcement interacts with the concrete as though they were one part and together carry the load. This implies that the amount of reinforcement has to be determined before the structure is built. Since, in normal cases, a concrete structure has a very long life expectancy, it is not unusual that the demands placed upon the structure change with time. The structure may in the future be required to bear greater loads or to fulfil new standards in the future. When such a situation arises it should be determined whether it is more economical to strengthen the exisiting structure or to replace it. In comparison with the building of a new structure is it often more complicated to strengthening an existing one.

A repair method used world-wide to strengthen existing concrete structures is to attach steel plates in the form of additional reinforcement on the outside of the structure using a cold cured epoxy adhesive. However, before joining the plates and the concrete together, certain steps must be taken. First the latiance of the concrete and the rust mill of the steel plates have to be removed by sandblasting. The surfaces need to be cleaned very carefully. The steel plates are then treated with a primer to prevent corrosion. The last step is to join the two materials together and after about one week the hardening process for the adhesive is concluded.

So far, most of the structures in the world strengthened with this method have been strengthened for bending. However, this paper presents tests performed in Sweden, at Luleå University of Technology, with concrete structures strengthened for shear using steel or FRP (Fibre Reinforced Plastics) materials. Also, design formulas for engineering purposes are presented, both for bending and shear.

1.2 Background

The project to strengthen existing concrete structures started in Sweden in 1988. The Swedish Road Administration needed to increase the load bearing capacity of existing highway bridges in order to meet the standards in the EEC (European Economic Community). From an internal investigation of existing bridges it was found that almost 1 300 of 9 000 existing concrete bridges needed to be replaced or strengthened. The method of gluing outer reinforcement could then be applied. At Luleå University of Technology extensive reserch has been performed for this strengthening methodology; practical tests, theoretical derivations, and a full scale test have been performed. The method of increasing a concrete structure's bending capacity or its stiffness with steel plates bonded to its soffits is today quite well understood. Nevertheless, strengthening a concrete member for shear is more difficult. This paper will discuss some problems in strengthening concrete structures for shear and how it is possible to perform this type of strengthening. Before the results are presented, however, a short presentation of the engineering formulas for design are discussed.

2. THEORY

2.1 Truss model

Assuming that we have full interaction between concrete, steel, and adhesive, that plane sections remain plane during loading (Bernoulli-Navier-hypothesis) and that we have a known relation between strains and stresses, then a truss model for beams will give us design criteria for plate bonded structures in the elastic domain. Let us study a beam strengthened for both bending and shear with inclined stirrups and steel plates as shown in figure 1.

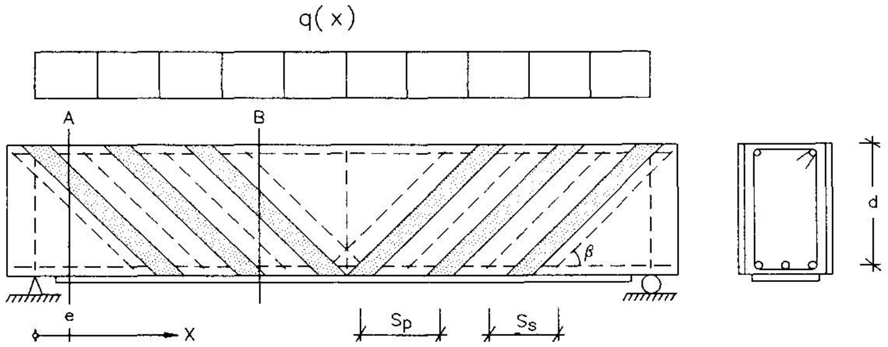

Fig. 1 Strengthened concrete beam with a distributed load q(x)

In the Swedish code for concrete structures, BBK 79, the following expressions are used for the load V_c carried by the concrete:

$$V_c = b_c d f_v \tag{1}$$

where

b_c = width
d = effective depth
f_v = formal concrete shear stress

$$f_v = \xi(1 + 50\rho)0.30 f_{ct} \tag{2}$$

where

$$\begin{aligned} \xi &= 1.4 && \text{when the depth} && d \le 0.2\ \text{m} \\ \xi &= 1.6 - d && && 0.2\ \text{m} < d \le 0.5\ \text{m} \\ \xi &= 1.3 - 0.4d && && 0.5\ \text{m} < d \le 1.0\ \text{m} \\ \xi &= 0.9 && && 1.0\ \text{m} < d \end{aligned} \tag{3}$$

$$\rho = \frac{A_b}{b_c d} \tag{4}$$

where A_b = bending reinforcement area in the tensile zone

If we cut out a section between point A and point B as in figure 2, simple equilibrium equations can be derived which can be stated in a very clear and descriptive way, see also [1]. From the equilibrium equations and using the notations in BBK 79, it is possible to write the following two equations for the design of strengthening with bonded plates. In the formulas below, f_{yd} denotes the yield strength, s the spacing of the shear reinforcement, and the notations s and p as exponents denote stirrups and plates, respectively.

$$\frac{A^p_{sv}}{s_p} = \frac{1}{f^p_{yd}\,0.9d}\left[V_A - b_c d f_v - \frac{f^s_{yd} A^s_{sv}\,0.9d}{s_s}\right] \tag{5}$$

$$A_p = \frac{1}{f_{ydp}}\left[\frac{M_A}{0.9d} + \frac{1}{2}V_A + \frac{1}{2}b_c d f_v - f_{ydb} A_b\right] \tag{6}$$

The first equation gives the cross section area, A^p_{sv}, for shear strengthening and the second gives the necessary cross section area, A_p, for bending strengthening. In these formulas, however, it is assumed that the load, the quality of the steel, and the distance between the plates are decided or known before strengthening takes place or else iterative calculations are needed. Nevertheless, when the concrete starts to fracture more accurate methods are needed, i.e NLFM (Non Linear Fracture Mechanics) methods may be suitable. However, these methods are not treated in this paper, but can be found in [1].

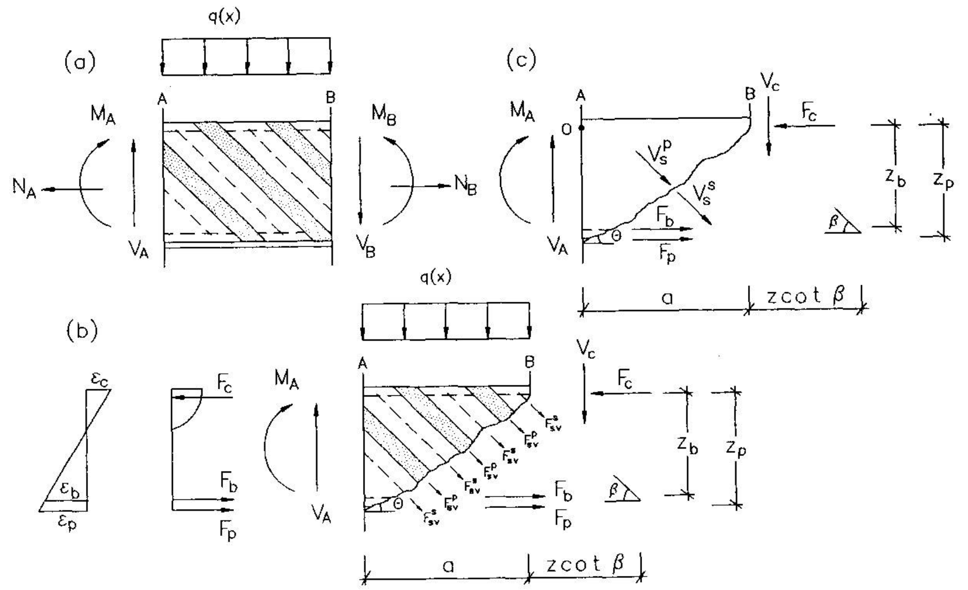

Fig. 2 Section A-B and corresponding forces acting in the studied section

3. BEAMS TESTED IN SHEAR

3.1 Preparation

The beams tested in shear, denoted series F, consisted of 6 concrete beams with the dimensions shown in figure 3. One beam, F1, was left unplated to act as a reference beam; however, even this beam was strengthened after failure. The other beams were all plated, most of them with steel plates, but also with GFRP (Glass Fibre Reinforced Plastics) and CFRP (Carbon Fibre Reinforced Plastics) plates. The sides of the beams were sandblasted and cleaned very thoroughly before the plates were attached. The beams that were strengthened with steel plates had a uniform glue layer with a thickness of 2.0 mm, whereas the beams with GFRP and CFRP plates had a glue layer that was estimated to be 0.5 mm thick.

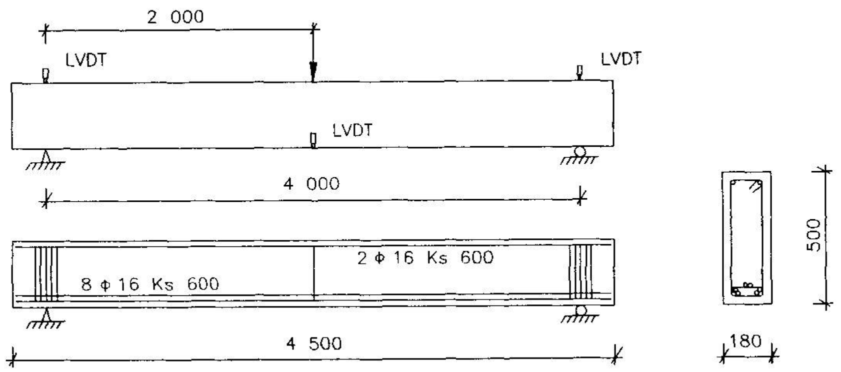

Fig. 3 Test specimens and test arrangements for series F

All of the beams were tested in three point bending, and measurements were made of the force, the mid span deflection, and the settling over the supports, which is shown in figure 3. The crack propagation during loading was also recorded. The loading was deformation controlled with a deflection rate of 0.3 mm/min. In figure 4 all of the beams are shown before loading. It should also be mentioned that in addition of beam F1, beam F2 was used in two of the tests. In table 1 the material parameters are summarised together with data for the concrete and the epoxy bonded plates. All the notations in table 1 can be found in figure 4 where E is the Young's modulus of the strengthening plates.

The low yield strength of the steel plates is readily apparent in contrast to the high yield strengths of the GFRP and especially CFRP plates.

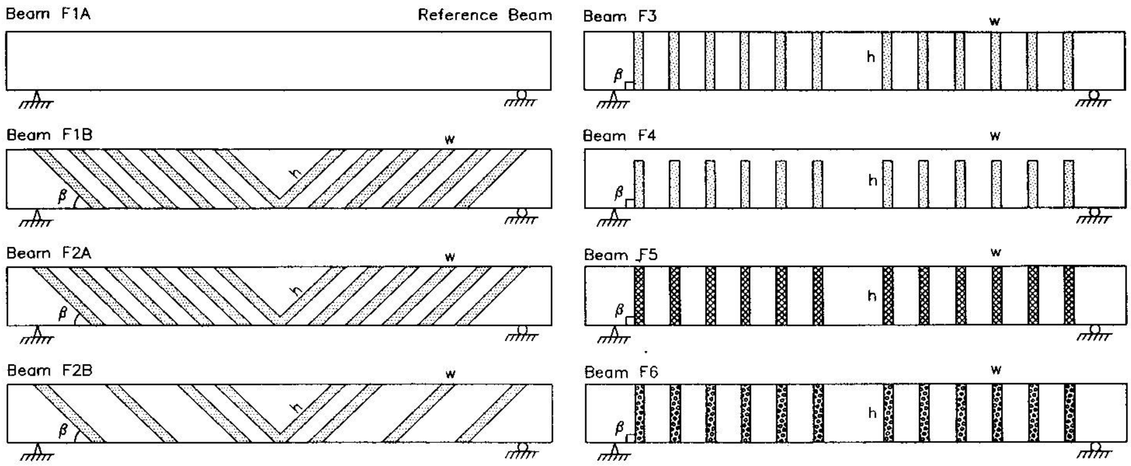

Fig. 4 Beams used in test series F, before loading

Beam	Plates type	h [mm]	w [mm]	t [mm]	s [mm]	β	f_{cc} [MPa]	f_{ct} [MPa]	f_{yd} [MPa]	f_{ud} [MPa]	E [GPa]
F1A	---	---	---	---	---	---	32.8	3.7	---	---	200
F1B	steel	$500\sqrt{2}$	80	2.9	300	45°	32.8	3.7	170	289	200
F2A	steel	$500\sqrt{2}$	80	2.9	300	45°	43.2	3.7	171	289	200
F2B	steel	$500\sqrt{2}$	80	2.9	600	45°	43.2	3.7	171	289	200
F3	steel	500	80	2.9	320	90°	56.3	4.2	161	289	200
F4	steel	400	80	2.9	320	90°	58.4	4.0	161	289	200
F5	GFRP	500	80	0.9	300	90°	48.6	4.0	564	612	22
F6	CFRP	500	50	1.2	300	90°	60.0	4.2	2497	2497	167

Table 1 Material parameters and dimensions of epoxy bonded plates in series F

Beam F1A is the reference beam and F1B the reference beam after strengthening. Beam F2A is strengthened in the same manner as beam F1B, however, the beam was not precracked before it was strengthened. Furthermore, beam F2B is the same beam with every other plate removed. Beams F3 and F4 were also strengthened with steel plates bonded to their sides. The difference between these beams and the ones already discussed was that the plates were attached vertically. However, the distance between the plates was 320 mm. On one side of beam F3, strain gauges were glued to the steel plates, see figure 5. Furthermore, the heigth of the steel plates for beam F4 was just 80 % of the plates used for F3. The aim of this test was to investigate the need for anchorage in the compressive zone. Beams F5 and F6 was strengthened with vertical GFRP and CFRP plates, respectively.

3.2 Results

In table 2 are the results from the tests summarised. Nevertheless, before the results are presented, a short explanation of the table is needed. Beam $F1B_1$ is the strengthened reference beam and beam $F1B_2$ is the same beam but with every other plate removed. Beam F5A and F6A are the beams strengthened with GFRP and CFRP, respectively, where the theoretical yield limit for the plates has been used as a design criterion. Beam F5B and F6B are the same beams, but here the actual stess level in the plates at failure has been considered. Let us now study the results from the tests. A comparison of the beams F1B and F2 reveals that despite that beam F1B was precracked, it could support the same load as beam F2. This means that it may be possible to strengthen an existing cracked structure so that it attains the same bearing capacity as an uncracked one. Beam F3, strengthened with vertical plates, almost reached the theoretical design level. If we study the curve of the measured strains, see figure 5, we can see where the steel plates start to contribute to the load bearing capacity. If we follow the strain curves over the time of loading, we see that, for up to almost 70 % of the failure load, the strains on both sides of the midsection of the beam are quite symmetrical.

Beam	V_s [kN]	V_c [kN]	$V_s + V_c$ [kN]	V_{exp} [kN]	$\frac{V_{exp}}{V_c + V_s}$	$\frac{V_{exp}}{V_{ref}}$	Type of failure
F1A	---	122.5	122.5	122.5	1.00	1.0	shear
$F1B_1$	141.5	122.5	264.0	215.0	no comparison made, bending failure		
$F1B_2$	70.8	122.5	193.3	205.3	1.06	1.8	shear
F2A	142.2	122.5	264.7	212.5	no comparison made, bending failure		
F2B	71.1	122.5	193.6	202.5	1.05	1.7	shear
F3	94.3	122.5	216.8	200.0	0.92	1.6	shear
F4	75.8	122.5	198.3	160.0	0.81	1.3	shear
F5A	108.6	122.5	231.1	150.0	0.65	1.2	shear
F5B	5.0	122.5	127.5	150.0	0.85	1.2	shear
F6A	409.0	122.5	531.5	155.0	0.30	1.3	shear
F6B	27.5	122.5	150.0	155.0	0.98	1.3	shear

Table 2 Theoretically calculated and experimentally obtained values for series F

Nevertheless, the divergence at the end of the loading curves is large for the most extended gauges. At failure, gauge 8 exceeded 800 µs; this is equivalent to a stress of 165 MPa which is above the yield strength, f_{yd}, of the steel used.

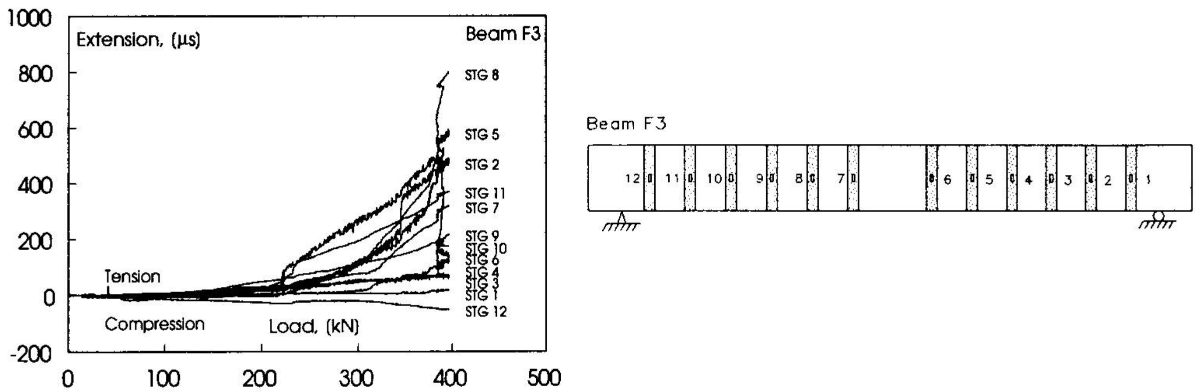

Fig.5 Placement of gauges and load-extension curves for measured strains for beam F3

The height of the steel plates for beam F4 was just 80 % of the plates used for F3. However, the load reached for beam F4 was 80 % of that reached for beam F3, which could imply that it may be possible to use, in a restricted way, a simplified model for strengthening, even if the plates can not be anchored in the compressive zone, see [1]. It is known that the concrete starts to fracture at a specific strain level; this level is in most dimensioning cases the critical design variable. Table 2 also shows that if the stress level at failure is used as a design criterion the theoretical shear level corresponds quite well to the one obtained from the tests on beams F5B and F6B. However, to keep the strain level in the strengthening plate at a moderate level two design criteria have to be fulfilled. First, the Young's modulus in the plate must be sufficently large (the larger the better) and second the cross section of the plate must have a sufficently large area. Let us study table 2. The small modulus is the reason for the limited strengthening effect of the GFRP plates. Since Young's modulus is a material parameter it can not be changed. For the CFRP plates the situation is different. Carbon fibre has a quite large Young's modulus which implies it is suitable for strengthening. However, in this particular case the cross sectional area is too small, but if the area is increased it is most likely that the shear capacity would also be increased, see also [1].

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