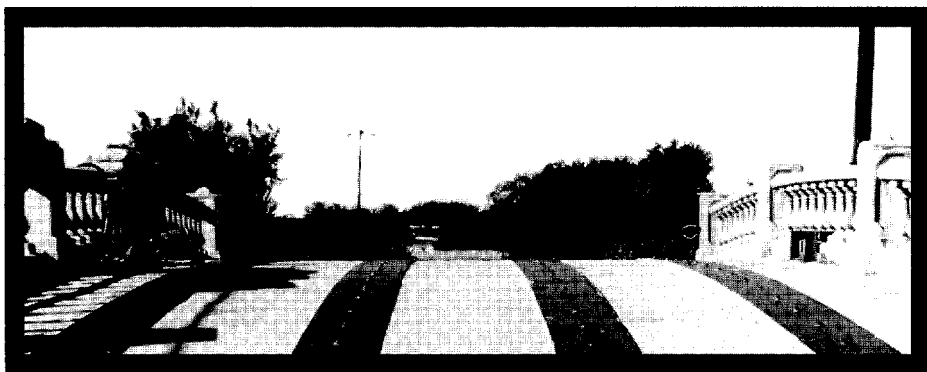


Repair and Strengthening of Concrete Members with Adhesive Bonded Plates



Editors

**R. Narayan Swami
Robert Gaul**



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PREFACE

The ACI Technical Session "Repair and Strengthening with Adhesive Bonded Plates" was held in Washington, D.C. on March 15-16, 1992. The papers presented at the technical session and the additional papers included in this special publication provide the first comprehensive publication of information on this subject in the United States. The authors are recognized experts from Europe, the United States, and Asia. The papers represent an excellent overview of the state of the art of this innovative technique for repair and strengthening.

The papers cover research, design, construction methods, and case histories. The research described includes not only theoretical analysis and short-term experimental programs but also test programs that involve long-term outdoor exposure. Research in the use of fiber reinforced plastic plates instead of the more commonly used steel plates is also described. The design papers illustrate practical engineering approaches and include details of anchoring plates, effect of plate dimensions, adhesive performance, and temperature limitations. Construction methods described include preparation of the plates, application of adhesive, and installation of the plates. The case histories provide a broad and historical view of the use of adhesive bonded plates to strengthen buildings and civil engineering structures.

Although strengthening with adhesive bonded plates is finding growing acceptance, its use is limited because many engineers are not aware that a body of knowledge exists that can allow engineers to use this method based on sound engineering principles. It was the purpose of this technical session to begin to disseminate that knowledge.

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Contribution of Externally Bonded Steel Plate Reinforcement to the Shear Resistance of Reinforced Concrete Beams

by R. Narayan Swamy, Roy Jones,
and Abdelhamid Charif

Synopsis: Although some test data are available on the behaviour of reinforced concrete beams with bonded plates in shear, there is still considerable lack of information on the effectiveness of such plates in resisting shear forces. This paper presents the results of the first stage of a comprehensive study to evaluate the role and mechanism of strengthening in shear when bonded steel plates are used either at the tension face or in the webs. The major variable in this study was the shear span-effective depth ratio which is the critical parameter for shear failures, and two different geometries of web plates were used. The effect of these bonded plates on deformations, cracking, failure loads and mode of failure are presented and discussed. Bonded plates at the tension face enhanced the flexural stiffness of the original beams which were weak in shear, but their influence in enhancing shear behaviour was minimal; bonded web plates, on the other hand, were very effective in upgrading the structural behaviour in shear. However, the latter were unable to reach their full load capacity due to shear cracking in the unreinforced part of the webs and the consequent debonding of the web plates.

Keywords: Adhesives; beams (supports); plates (structural members); reinforcing steels; renovating; shear properties; shear strength; tension; web reinforcement

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INTRODUCTION

Adhesive bonding is a versatile technology, and has been used in a wide variety of engineering applications, from every day domestic use to high performance applications in aerospace and automotive industries. More recently, bonding of structural plates using epoxy resins to the surfaces of structural members in buildings and bridges has also been adopted as a means of upgrading/strengthening/retrofitting and rehabilitating structurally inadequate or damaged structures (1-4). Both from technical and practical points of view, the plate bonding technology offers many advantages, but perhaps the most valid argument for its adoption is that it is based on sound engineering principles, it is cost-effective, it does not change existing structural stresses dramatically and, above all, when completed, the changes in the overall sizes of the structure are minimal and are only of the order of a few millimetres.

Comprehensive research data are now available on the engineering effects of bonding steel plates to enhance the flexural behaviour of reinforced concrete beams which are adequately reinforced in shear (5-7). The results of these extensive studies show that through correct design, both beam action and composite behaviour of the plated beams can be preserved until failure. The plating can substantially increase the beam flexural stiffness, reduce cracking and the resultant structural deformations at all load levels and, consequently, the serviceability loads are considerably enhanced by the stiffening action of the bonded plates.

One problem associated with the bonding of plates in the tension zone is that high local interface bond stresses and peeling forces are created at the ends of the plates when the composite beam is loaded in flexure. Providing adequate anchorage to the bonded plates is therefore an essential component of the design process, in the same way that conventional steel bar tension reinforcement needs to be checked for bond and anchorage. Tests show, however, that bonded anchor plates can overcome anchorage failure and enable the full theoretical flexural strength to be achieved, together with ductile behaviour. Design guide rules are available to prevent premature brittle shear/bond failures (5,6).

These studies also show that the plate bonding technique can be applied confidently to structurally damaged structural members. It has been shown that strengthening of significantly cracked beams is structurally efficient and that the plated beams can be restored to strength and stiffness values superior to those of the original unplated beams (7). The additional attraction of this technology is that the ultimate strength of plated beams failing in flexure can be very satisfactorily predicted by existing reinforced concrete beam theories (5-7).

Plates for Shear Resistance

Although plates have been added in practice to the sides of beams to supposedly enhance their shear capacity (1,8,9), there is only limited information on their effectiveness to carry shear forces. Bending shear creates resultant diagonal tensile and compressive stresses; and in reinforced concrete beams, the problem of shear is a complex phenomenon, and both the shear strength and the mode of failure are influenced by numerous factors (10-13). Nevertheless, the shear capacity of concrete members can be increased by externally bonded shear reinforcement in the form of intermittent vertical strips (8-9, 14-19). These tests show that plates bonded to the tension face alone or to the webs of the beam only cannot enhance the shear capacity of the beams to the values required to produce ductile, flexural failures. It is, of course, possible to bond plates over the full depth of the webs and over the full length of the beams, or restricted to the shear spans, and this can often enhance the load capacity of the beams to that of flexural failure; however, continuous web plates in the shear spans or over the full length of the beams are likely to create undesirable secondary failures and, in any case, continuous web plates do not appear to be the most attractive method of enhancing shear strength, whether metallic or non-metallic plates are used (19, 20). Anchoring the intermittent strips in the compression zone is a useful way of enhancing shear capacity and develop flexural failure, but even then undesirable shear cracking cannot be avoided as shown in tests on T-beams (18).

The aim of this study is to enhance our understanding of the role of plates bonded either to the tension face or to the webs of the beams in resisting shear forces. Since the ratio of shear span to effective depth is a major factor influencing the mode of shear failure, three such ratios were used in the tests.

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RESEARCH SIGNIFICANCE

Although some test data have been reported on the effectiveness of the plate bonding technique in enhancing the shear strength of reinforced concrete beams, the information currently available does not provide essential and basic guidance on three important factors. Firstly, what is the precise role and mechanism of strengthening when bonded plates are used either in the tension faces or in the webs alone of a beam with no or only partial shear reinforcement? Secondly, what is the most effective geometry and arrangement of bonded plates that will reduce shear cracking to a minimum, and produce ductile flexural failures? Finally, it is not yet clear if bonding plates is the most structurally efficient method of avoiding shear failures. Further, none of the studies reported so far have investigated the influence of the shear span - effective depth ratio on shear strength, which is the critical parameter influencing the mode of shear failures. This paper reports the results of the first stage of a comprehensive study to address the above issues, and shows that plates bonded to the tension face or webs alone cannot lead to ductile flexural failures.

EXPERIMENTAL PROGRAMME

Details of Test Beams

Eleven rectangular beams, in two series, were altogether tested. All the beams were identical in size, 155 x 255 x 2500 mm, reinforced with 3-20 mm diameter bars, at an effective depth of 220 mm. Series 1 consisted of three beams, which were adequately reinforced in shear and designed to fail in flexure. These three beams were the control beams, and their results were to be compared with those in Series 2. To avoid shear failures, these three beams were provided with 6mm diameter shear links in both shear spans at a spacing of 75mm centre to centre. The details of these three beams are as follows and are shown in Fig. 1 and Table 1.

- Beam FO1 - No adhesive and no plate
- Beam FO2 - 3 mm thick adhesive layer only
- Beam F11 - 125 x 1.5 mm steel plate bonded through
1.5 mm thick adhesive layer

These three beams were tested under four point loading at a shear span/effective depth (a/d) ratio of about 3.5.

Series 2 consisted of eight beams which were designed to examine the role and contribution of externally bonded plates to the shear resistance of a reinforced concrete beam when the plate was bonded either only on the tension face or on the vertical shear faces of the beam. All these eight beams were

identical to the beams of Series 1 in their geometry and internal tension steel reinforcement.

Six of these beams were designed to investigate the effect of bonding a plate on the tension face on shear resistance. These beams were expected to fail in shear, and were made without any shear links in the shear spans, except for three links at each support to prevent bearing failure as shown in Fig. 2. These beams were tested, in pairs, at a/d ratios of 1.5, 3.0 and 4.5 – one beam at each a/d ratio being unplated (beams S43, S41 and S42) and the other with a 125 x 1.5 mm steel plate bonded to the tension face through a 1.5 mm thick adhesive layer (beams S53, S51 and S52). Details of these six beams are also shown in Table 1. These six beams were expected to provide data on the effectiveness of plates bonded to the tension face in enhancing the shear strength of the beams.

The two remaining beams of Series 2 (S61 and S62) were tested to study the possible use of externally bonded web plates as shear reinforcement. These beams were tested at an a/d ratio of 3.0 which was chosen as a compromise between beams of low and high a/d ratios. In both beams the external shear reinforcement was bonded to one shear span only, whilst internal stirrups were provided in the other shear span as shown in Fig. 3. Two plate configurations were used for shear reinforcement – double strips in beam S61 and channel sections for beam S62 (Fig. 3). The two beams had thus the same internal tension and external shear reinforcement, except that in beam S62, by the nature of the channel strips, they also provided intermittent bonded sections of plates at the tension face.

Details of Tests

All the beams were tested with the tension face at the top. This enabled the cracking on the tension and side faces to be carefully monitored. Beams tested at a/d ratios of 1.5, 3.0 (Series 2) and at 3.5 (Series 1) were tested under four-point loading; beams tested at a/d ratios of 4.5 (Series 2, beams S42 and S52) were loaded at mid-span without a constant bending moment region. All the beams were extensively instrumented to obtain deflections, concrete and steel (both rebar and plate) strains and rotations as shown in Figs 1 to 3; in addition, the strains in the bonded plate shear reinforcement in beams S61 and S62 were also monitored at the locations shown in Fig. 3. All these measurements were made at every load stage up to failure.

Details of Materials

The same concrete mixture was used in all the beams, and the concrete was proportioned to give 28 day cube compressive strength of about 50 MPa (about 40 MPa cylinder strength), consistent with that used in bridge structures. The mixture proportions were 1 : 1.75 : 2.61 : 0.54 (cement: sand: coarse aggregate: water-cement ratio). ASTM Type 1 portland cement, washed natural

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river sand, and 10 mm nominal size crushed gravel were used as concrete constituents. The mixes were very workable with a slump of about 100 mm. At 28 days, the cube compressive strength of the concrete averaged 50 MPa, flexural strength 4.5 MPa, and the elastic modulus 30.7 GPa. The shrinkage of the concrete was about 400×10^{-6} at 28 days and was about 700×10^{-6} at 1 year. The actual concrete strength properties obtained from control specimens cast with each beam are shown in Table 1.

Cold worked ribbed bars of 20 mm diameter were used for the tension steel of the beams, and mild steel for the links and top bars to support the links. The tension steel had a characteristic strength of 460 MPa, and an ultimate tensile strength of 585 MPa. The mild steel had a characteristic strength of 325 MPa, and an ultimate strength of 475 MPa. The plates for external reinforcement were also of mild steel, with a characteristic strength of 245 MPa and an ultimate tensile strength of 340 MPa.

The epoxy resin used to bond the plates in the tests was a two-part system. The resin had a compressive strength of about 45 MPa, an ultimate tensile strength of 6.5 to 13.5 MPa, and an elastic modulus of 1 to 2 GPa, depending on specimen thickness and loading rate.

Bonding Procedure

The beams and the cube control specimens were demolded at 24h and cured in an uncontrolled internal environment. The bonding procedure for the plates was the same as those adopted by the authors in their earlier tests. This consisted of grit blasting the plates on both faces, removing laitance and all other loose material from the concrete surface and applying the epoxy adhesive to both concrete and plate surfaces. The thickness of the adhesive was controlled carefully by hardened epoxy spacers. The epoxy resin was allowed to cure under pressure for 14 days in all cases prior to testing.

TEST RESULTS AND DISCUSSION

From a large amount of data obtained from the tests, only those relevant to this paper are discussed here. In particular, since the overall aim of this study was the development of a plate bonding arrangement that will avoid shear failures and produce ductile flexural failures, the results of the tests are analysed in terms of failure loads and failure modes rather than in terms of shear stresses.

Failure of Control Beams

The control beams FO1, FO2 and F11 in Series 1 all failed in flexure, as was expected, with concrete crushing in the compression zone after the tension bars had yielded in beams FO1 and FO2 and after the bars and plate had yielded

in beam F11. Beams FO1 and FO2 had nearly the same ultimate strength, with beam FO2 registering a marginal 2% increase in strength. Clearly, the adhesive layer itself contributed little to the strength of the beam, as indeed would be expected. Beam F11 showed about 14% increase in ultimate flexural strength compared to beam FO1, thus showing a modest enhancement in strength from the bonded plate. Since the results of these beams are intended to be the basis for comparison with shear failures and since flexural failures of plated beams are in general well understood, the results of these beams are not reproduced here; they are also not discussed any further in this paper. However, these results have been used to evaluate the data obtained from the beam tests in Series 2.

BEAM SHEAR FAILURES

Deformation Properties

Typical variations with load of deflection, concrete compressive strain, steel bar strain and steel plate strain are shown in Figs 4 to 7 for the unplated beams (beams S43, S41 and S42) and for beams with a plate at the tension face (beams S53, S51 and S52) and tested at a/d ratios of 1.5, 3.0 and 4.5 respectively (Series 2). The load-deflection curves show that plating of the tension zone enhances the stiffness of the beam even if it is weak in shear, and this is also confirmed by the load-rotation curves not reproduced here. This increase in stiffness reduced all the deformations of the unplated beams by amounts varying from about 10 per cent to about 40 per cent. The provision of the bonded plate at the tension face also increased slightly the load capacity of the corresponding unplated beams, and the plated beams were therefore able to sustain slightly higher values of deflection and strains as shown in Figs 4 to 6. However, all the figures show that there was little plastic deformation prior to failure except in the case of the concrete compressive strains which showed some non-linearity near the failure load. Figs 6 and 7 confirm this and show that neither the reinforcing bars nor the steel plates had reached yield stress at failure, although the strains in the plates were approaching values of 2000 microstrains. The plates also showed some relaxation of stress prior to failure, confirming that they had not fully yielded. Beams tested at a/d ratios of 4.5 and 1.5 showed maximum concrete compressive strains of 2700 to 3000 microstrains, approaching values normally obtained in flexural failures.

The deformation behaviour of beams with bonded shear reinforcement, beams S61 and S62, is shown in Figs 8 to 10. These figures show that the deflections, concrete strains and steel bar strains all exhibit a consistent pattern of behaviour and confirm that externally bonded shear reinforcement was also effective in enhancing beam stiffness, ultimate failure load and the ductility compared to the beams without internal shear reinforcement. Quantitatively, the reductions in the structural deformations varied from about 3 to 12 per cent.

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Although this was less than that achieved by the bonded plate at the tension face, as one would expect it to be, there was clear evidence that the combination of internal tension bar reinforcement and external bonded shear reinforcement resulted in some increase in flexural stiffness, although this was only marginal. With both types of shear plate configuration there was visible and progressive improvement in load capacity and non-linear behaviour; but, the channel configuration was clearly superior, although marginally, than the strip configuration. Obviously, the channel shape was able to give better anchorage, binding the webs and tension face, whereas the strips were without this advantage. The effectiveness of the bonded shear reinforcement is further confirmed by the load-steel bar strains shown in Fig. 10 – the steel reinforcement had yielded prior to failure of these beams. A comparison of steel bar strains in Fig. 10 and Fig. 6 also confirms that in a beam weak in shear, bonded web steel is far more effective in upgrading the structural behaviour than the bonded tension plate. The beams with bonded web steel developed steel bar strains in excess of the yield strains, beyond 3000 microstrains compared to beams with the bonded plate on the tension face.

Typical strain distributions on the bonded channel strip reinforcement in beam S62 are shown in Fig. 11. The strains on the vertical faces of the bonded strips in beam S61 were similar. The maximum strains recorded on the vertical faces of the bonded shear steel varied from about 60 to about 530 microstrains. Thus, there was shear transfer from the concrete to the bonded shear reinforcement, and although they carried some shear loads, the implication of Fig. 11 is that the shear force transmitted by each strip is different, and that this very much depends on the location of the diagonal crack, as has been observed in beams with internal link reinforcement (10-13). None of the bonded strips reached their yield strain and they were thus unable to reach their full load capacity.

Cracking Behaviour

The first cracks observed in all the tests occurred in the flexural zone or under the load point (for beams of $a/d = 4.5$) irrespective of the presence or absence of internal links or bonded plates. Beams with a bonded plate at the tension face exhibited a delay in the formation of the first flexural cracks, the plate acting as a crack arrester as has been reported earlier (5-7). This did not occur, as would be expected, in beams S61 and S62 which had only bonded shear reinforcement. However, in these beams flexural cracking was reduced in the shear span where the external strips or channels were bonded. The first flexural crack moments are shown in Table 2.

The formation of the diagonal cracks generally followed the pattern, reported in literature, of shear cracking in beams without web reinforcement (10-13). The longitudinal plates at the tension face did not delay the first diagonal cracking, as shown in Table 2, and there is indeed no reason why they

should do so. In these beams (beams S53, S51 and S52) and in the corresponding unplated beams (beams S43, S41 and S42), the formation of the first diagonal crack was a function of the a/d ratio. In beams tested at a/d ratio of 1.5, the web shear cracks occurred independently of flexural cracks, whereas in beams tested at $a/d = 4.5$, the diagonal crack was initiated as an extension of flexural cracking. In beams tested at $a/d = 3.0$, the diagonal cracking was a combination of flexural and shear cracking.

Since the provision of the bonded plate at the tension face increased the ultimate failure load slightly (as shown in Fig. 4 to 6 and discussed later), the proportion of the ultimate load at which the first shear cracks appeared tended to be slightly lower for the plated beams. Also, beams tested at $a/d = 4.5$ tended to fail soon after the appearance of the first diagonal cracks, whereas those tested at $a/d = 1.5$ and $a/d = 3.0$ sustained higher loading after the formation of the diagonal cracks, following generally what is reported in literature on the shear behaviour of reinforced concrete beams (10-13). However, taking all these phenomena together, it is fair to say that so far as the initiation of diagonal shear cracking is concerned, the plates bonded at the tension face alone did not give much help to enhance shear behaviour.

Beams with bonded shear reinforcement, beams S61 and S62, on the other hand, clearly enhanced the diagonal cracking load (Table 2), but there was no difference in this load capacity between beams with the two different types of bonded shear reinforcement. Although this increase in strength was only of the order of about 17 per cent, bonded shear reinforcement was effective in carrying shear loads, and the propagation of the diagonal cracks was particularly slow and steady at the early stages. In both beams S61 and S62, the visible diagonal cracks occurred between the vertical strips – near the load points, these cracks developed from vertical flexural cracks, whereas near the supports, where the effect of bending is much less, these diagonal cracks tended to start off as inclined cracks.

Obviously, with no internal shear links, the diagonal cracks would have formed somewhere in the shear span within the body of the beam. The bonded strips can only control their formation and appearance at the beam surfaces and where they are located, but cannot prevent, like the links, their formation inside the concrete. In the unreinforced web areas between the strips, therefore, diagonal cracks become initially visible, and perhaps much more active, as the shear is progressively transferred to these unreinforced areas. On increasing the loading, these diagonal cracks reappeared between other strips further along the beam, thereby implying that they had formed and continued to propagate behind the vertical strips. It was thus clear that bonded shear reinforcement was effective, unlike bonded tension face plates, in contributing to shear, but that this contribution was limited for the reasons explained above.

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Modes of Failure

All the eight beams failed in shear before reaching their ultimate flexural strength. Fig. 12 shows the unplated beams (beams S43, S41 and S42) and the corresponding plated beams (beams S53, S51 and S52) after failure. Descriptive details of the mode of failure are given in Table 2. Beams tested at $a/d = 4.5$ (beams S42 and S52) failed by diagonal tension soon after the appearance of the inclined cracks. Failure was characterized by tensile splitting along the internal tensile steel and subsequent instability of the compression zone. Beams S41, S51 and S43, S53 tested at a/d ratios of 3.0 and 1.5 respectively failed by a combination of diagonal tensile splitting in the webs, splitting along the tension steel and some concrete crushing at the compression edge of the crack. In all cases, the plated beams showed much higher intensity of cracking and failure, with wider cracks and more well-defined fracture planes, whereas these were much less accentuated in the companion unplated beams as shown in Fig. 12.

The two types of bonded web steel produced slightly different types of failures. In beam S61, the critical diagonal crack formed between the web strips near the load point progressed towards the tension face, and the beam failed by splitting of the concrete cover, and the crack extended up to the support through the tension face uncovered by the strips (Fig. 13). Beam S62 with external channels failed by diagonal splitting resulting from the debonding of the second channel from the load point (Fig. 13 and Fig. 3). Some crushing of the compression zone occurred in beam S62. In both beams there was some tendency for vertical splitting either in the web or at the edge of the bonded web steel, which contributed to its debonding from the concrete surface.

It was clear that the channel section was a better geometry of web steel than the strip bonded only to the sides of the beam in controlling the failure mode. The section bonded to the tension face helped to prevent the splitting of the tension cover zone which was observed in beam S61 with bonded strips.

Failure Loads

Since the ultimate aim of this study is to develop a plate bonding system that will avoid shear failures and produce ductile flexural failures, the modes of failure of the tested beams are discussed in terms of failure loads rather than in terms of shear stresses. Table 3 shows the failure loads of the beams and the ratio of the ultimate failure moment to the theoretical ultimate flexural moment calculated on the basis of conventional reinforced concrete theory and equilibrium and compatibility conditions.

It is seen that beams with a plate bonded to the tension face carried slightly enhanced failure loads compared to the unplated beams, and this increase varied from about 10 to 16 per cent. Beams with bonded shear

reinforcement, on the other hand, carried additional loads of 33% to 40% compared to the control beam. The beam with bonded channel sections (S62) performed marginally better than the beam with strips (S61).

It is clear that bonded shear reinforcement is effective in contributing to the shear resistance of the RC beams with no internal shear links, and that they are more effective in carrying shear than plates bonded to the tension face alone. The beams with a plate bonded to the tension face carried about 60 to 70 per cent of the loads required to make the beams fail in flexure, depending on the a/d ratio at which they were tested, whereas the beams with bonded shear reinforcement carried about 80 per cent of the loads required to produce flexure failure. The strips and channels thus did not perform as well as an equivalent cross-sectional area of internal stirrups. The shear failures were caused by the inability of the chosen configuration of the bonded plates, whether at the tension face or in the webs, to prevent the propagation of the diagonal tension cracks in the tension zone near the support, in the web and near the compression zone. Since the tests reported here cover the broad range of a/d ratios that cause shear failure, closer attention to the detailing of the plates should be able to produce flexural failures.

CONCLUSIONS

From the results obtained in this study, the following conclusions can be drawn.

1. Plating the tension zone of a beam enhances its flexural stiffness even if it is weak in shear. For the beams tested in this study, this increase in stiffness reduced all the deformations of the unplated beams by amounts varying from 10 to about 40 per cent depending on the shear span - effective depth ratio.
2. There was clear and consistent evidence that externally bonded web steel also contributed to enhanced stiffness, higher failure loads and increased ductility compared to beams without internal shear links. The channel configuration was marginally superior in this respect than the strip configuration.
3. In a beam weak in shear, bonded web steel is far more effective in upgrading the structural behaviour than the bonded tension plate.
4. There was clearly shear transfer from the concrete to the bonded shear reinforcement, but such reinforcement was unable to reach their full load capacity due to formation of diagonal cracks in the unreinforced parts of the web or tension face and the resulting debonding of the web steel or cracking in the tension zone.

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5. Taking all aspects of structural behaviour into consideration, it is fair to say that so far as diagonal shear cracking is concerned, the plates bonded at the tension face did not give much help to enhance shear behaviour. The bonded shear reinforcement, on the other hand, clearly enhanced the diagonal cracking load.

6. Beams weak in shear but with a plate bonded to the tension face carried slightly enhanced failure loads varying from 10 to 16 percent. Beams with bonded shear reinforcement carried additional loads of 33 to 40 percent compared to the control beam. Beams with bonded channel sections performed, on the whole, better than beams with bonded strips.

7. Bonded shear reinforcement is far more effective in contributing to the shear resistance of a reinforced concrete beam with no internal shear links than plates bonded to the tension face. However, neither, when used alone, is likely to produce ductile flexural failures. Whatever be the geometry of the bonded web steel, diagonal cracks will always appear in the unprotected parts of the web and behind the bonded web steel, and are likely to lead to premature shear failures.

ACKNOWLEDGEMENT

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TABLE 1 — DETAILS OF TEST BEAMS

Beam	Shear Span to depth ratio a/d	Internal steel links	Longitudinal steel plate	External web steel	Cube strength MPa	Modulus of rupture Mpa	Indirect tensile strength MPa	Remarks
<u>SERIES 1</u>								
F01	3.5	Full	None	None	52.4	4.93	3.93	—
F02	3.5	Full	None	None	52.0	4.56	3.37	1.5mm adhesive layer
F11	3.5	Full	1.5mm	None	53.3	4.93	3.52	1.5mm adhesive layer
<u>SERIES 2</u>								
S43	1.5	None	None	None	52.4	4.98	4.03	3 stirrups at supports
S41	3.0	"	"	"	55.6	4.93	3.37	" " "
S42	4.5	"	"	"	53.1	4.87	3.58	" " "
S53	1.5	"	1.5mm plate	"	52.0	4.41	3.44	" " "
S51	3.0	"	"	"	55.4	4.88	3.78	" " "
S52	4.5	"	"	"	55.2	4.46	3.37	" " "
S61	3.0	side A	none	side B	54.5	4.60	3.33	stirrups in one shear span and 1.5mm strips bonded to the other
S62	3.0	"	"	"	52.7	4.74	3.56	stirrups in one shear span and 1.5mm channels bonded to the other

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TABLE 2 — STRENGTH RESULTS AND MODE OF FAILURE

Beam	Ratio a/d	First crack moment, kNm		Mode of shear failure
		Flex. crack	shear crack	
S43	1.5	23.0	46.0	Diagonal tension, bond splitting and concrete crushing
S53	1.5	30.7	46.0	Diagonal splitting and bond cracking
S41	3.0	23.0	46.0	Diagonal tension, bond cracking and concrete crushing
S51	3.0	30.7	46.0	Diagonal tension and bond splitting
S42	4.5	28.8	46.0	Diagonal tension and bond cracking
S52	4.5	34.5	46.0	Diagonal tension and bond splitting
S61	3.0	23.0	53.7	Diagonal tension and tension face splitting
S62	3.0	23.0	53.7	Diagonal tension, concrete crushing and plate debonding

TABLE 3 — BEAM ULTIMATE LOADS

Beam	a/d ratio	Failure load kN	Failure moment kNm	Theoretical ultimate flexural moment kNm	Failure moment
					Flexural moment
S43	1.5	305.0	50.3	77.6	0.65
S53	1.5	335.0	55.3	87.3	0.63
S41	3.0	138.0	45.4	77.6	0.59
S51	3.0	160.0	52.8	87.3	0.61
S42	4.5	109.0	54.0	77.6	0.70
S52	4.5	120.0	59.4	87.3	0.68
S61	3.0	184.0	60.7	77.6	0.78
S62	3.0	195.0	64.4	77.6	0.83

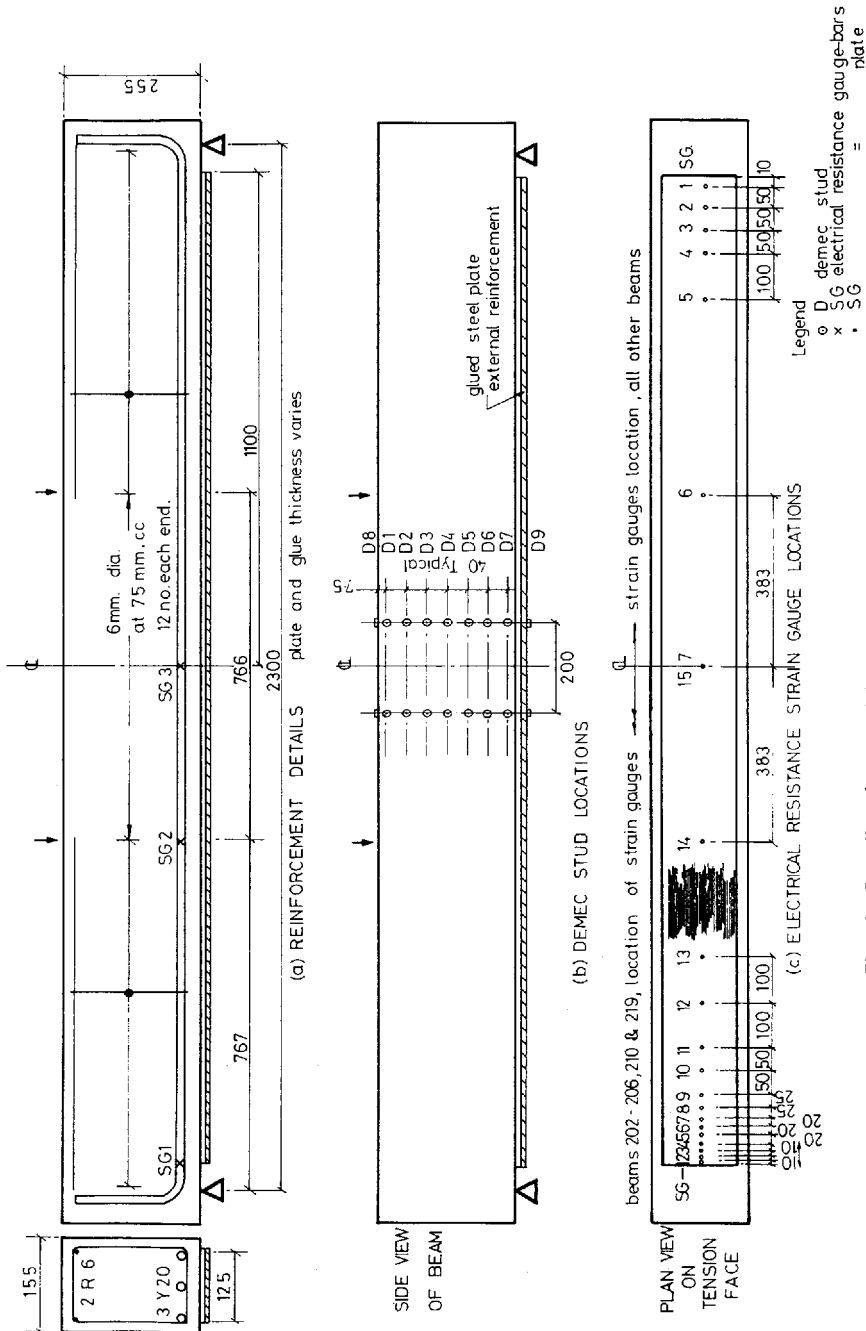


Fig. 1—Details of control beams failing in flexure

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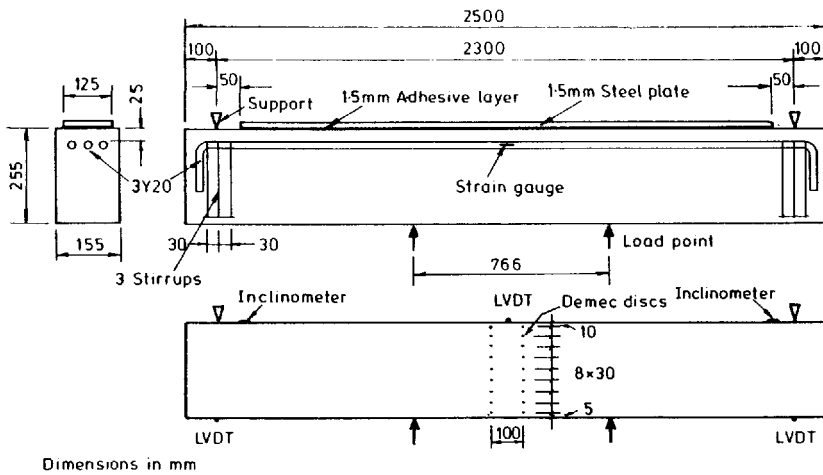


Fig. 2—Details of beams for shear resistance

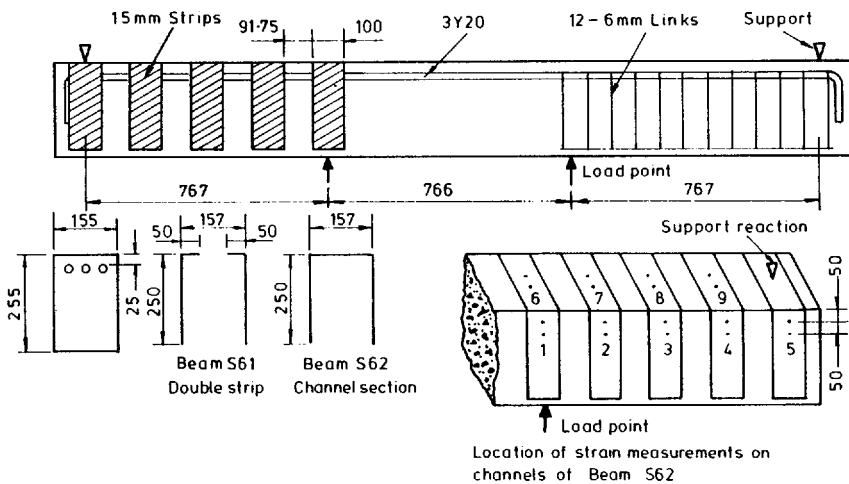


Fig. 3—Details of beams with external web steel

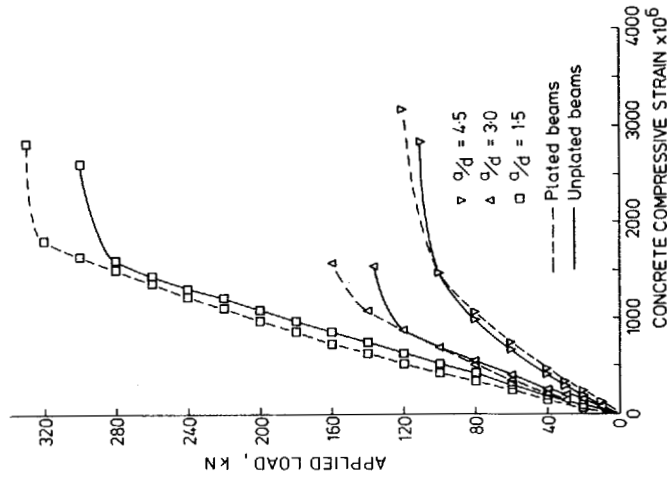


Fig. 5—Maximum concrete compressive strains in beams without and with plate bonded to tension face

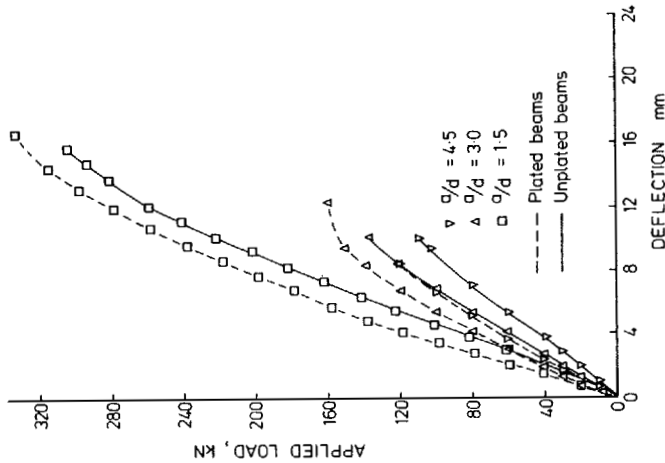


Fig. 4—Load-deflection behavior of beams without and with plate bonded to tension face

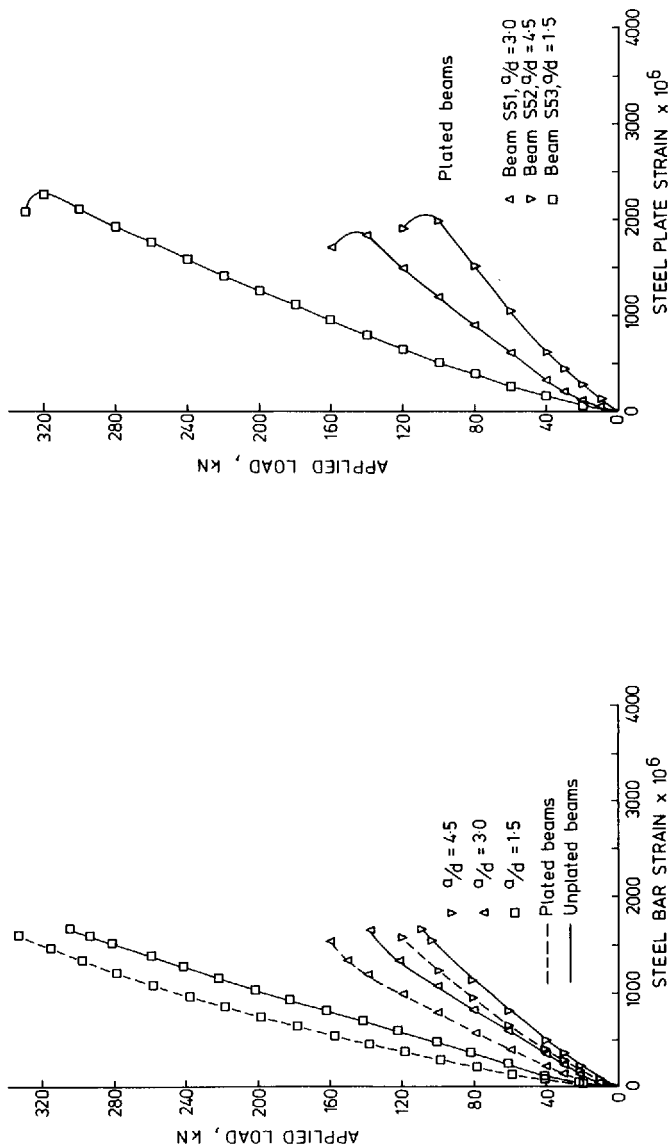


Fig. 6—Load-steel bar strain for beams without and with plate bonded to tension face

Fig. 7—Effect of shear span-to-effective depth ratio on plate strain

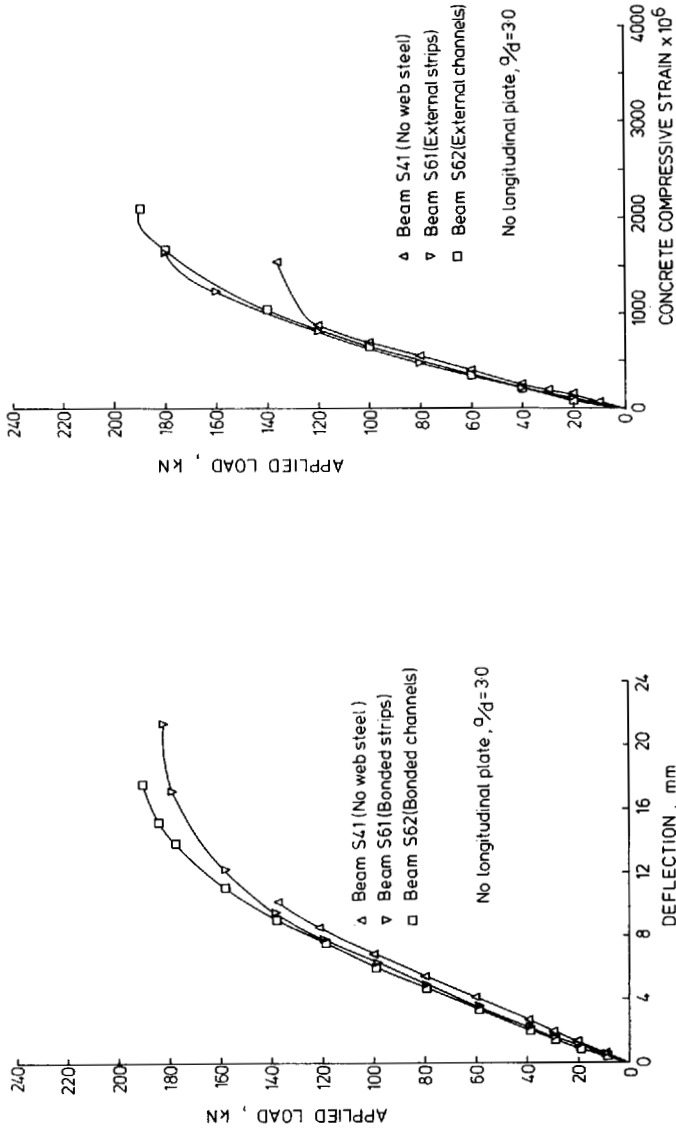


Fig. 8—Load-deflection behavior of beams without and with bonded web steel

Fig. 9—Concrete compressive strains in beams without and with bonded web steel

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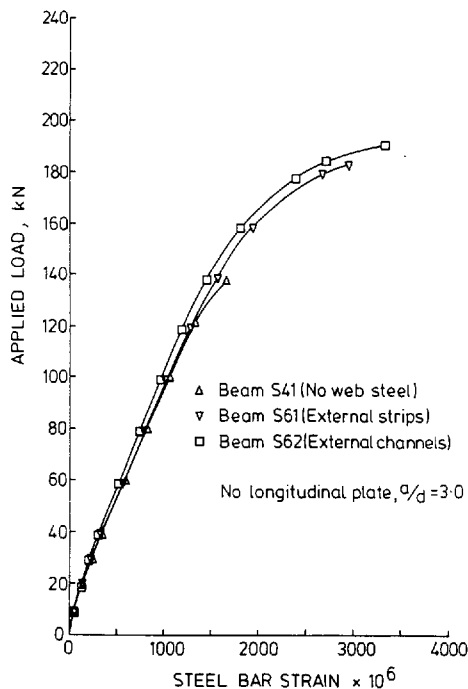


Fig. 10—Effect of bonded web steel on steel bar strains

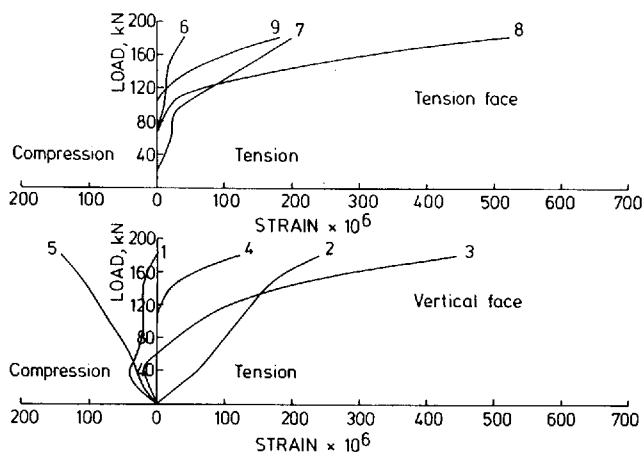
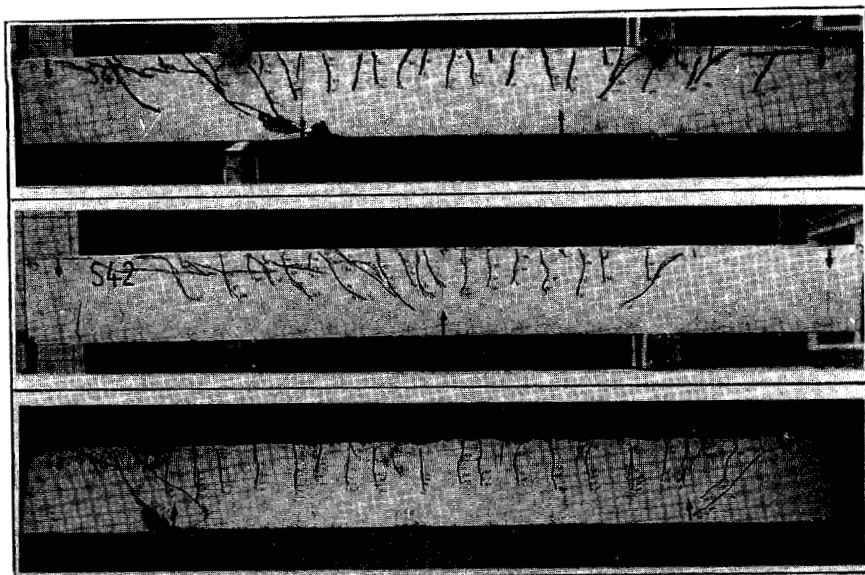
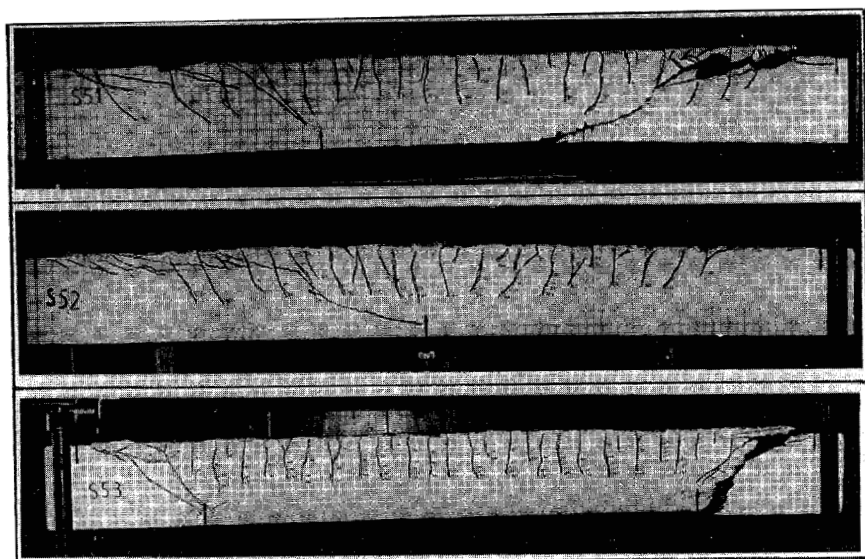


Fig. 11—Strain distribution in bonded channel sections of Beam S62



(a)



(b)

Fig. 12—Mode of failure of beams without and with bonded tension plate



(a)



(b)

Fig. 13—Mode of failure of beams with bonded web steel

SP 165-2

Special Design Aspects of Adhesive Bonding Plates

by D. Van Gemert

Research on the design of adhesive bonding plates for strengthening reinforced concrete structural elements is presented in this paper. Different design aspects are covered: increase of bending strength, calculation of anchoring length of epoxy bonded steel plates, temperature resistance, dowel action, strengthening of two-way slabs taking into account orthotropic behaviour of strengthened slabs. A brief overview of theoretical principles is given. The results of a large experimental program are evaluated according to the theoretical principles, and are translated into practical engineering design procedures. The design procedure for strengthening two-way slabs is explained in a practical example.

Keywords: Adhesives; bonding; durability; plates (structural members); reinforced concrete; structural design; thermal properties

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INTRODUCTION

Design methods have been developed for the calculation of the cross-sectional area of adhesively bonded steel plates, used for strengthening reinforced concrete beams or slabs against bending loads (1,2). The bending design is important, and it is very suitable to be treated with general formulas and design graphs (3). However, it is not the most difficult design aspect of strengthening reinforced concrete structural elements. The calculation of the anchoring length of the steel plates is an important problem, because the ending of the strengthening steel plates causes stress discontinuities, which in turn may lead to accelerated cracking of the concrete and premature failure of the strengthened structural element. Crack propagation can be stopped by placing dowels, to connect the external steel plate with the interior concrete behind the reinforcement rods. However, one must take advantage of the glued connection, to limit the number of bolts and make the technique economical. Even a limited number of dowels has a favourable influence on the shear stress distribution in the anchoring zone, which may result in a strength increase of more than 50 percent (4). The resistance of the epoxy bonded steel-concrete connection against thermal loads in the normal temperature range from -20°C to 90°C corresponds to the strength of the epoxy glue itself: nearly no change up to the glass transition temperature, but an important drop in resistance at higher temperatures (5). Procedures are presented to calculate the anchoring length of the steel strips, and to take into account the influence of dowels in the anchoring zone. They are based on theoretical and experimental research, carried out at the Reyntjens Laboratory of K.U.Leuven.

Strengthening of two-way reinforced concrete slabs is a frequently occurring problem. In two-way slabs the forces are transferred in the two load bearing directions. If strengthening is needed, it will be needed in both directions. However, crossing of epoxy bonded external steel plates is difficult to execute. If strengthening strips are only applied in one direction, the slab becomes orthotropic. By adapting orthotropy it will be possible to obtain the strengthening of two-way slabs by means of epoxy bonded steel strips in only one direction. The design method is based on orthotropic plate theory (3).

DIMENSIONING OF ADHESIVE BONDING STEEL PLATES IN BENDING

Adhesive bonding steel plates can be used in bending either to strengthen the structural element to increase its resistance to bending loads, or to stiffen the element to decrease deflections. Experiments have shown that the bond between external steel strip and concrete may be considered as perfect in the normal temperature range (1, 2). This makes it possible to calculate the necessary cross-sectional area of the bonded strip on the basis of normal reinforced concrete principles: assumption of linear strains over the cross-section. The only problem is the shifting of the neutral axis into the cracked zone when the strips are glued on a partly loaded element. This leads to a non-linear calculation. However, this calculation was easily programmed on computer, and design graphs were produced from which the necessary cross-sectional area can be read as a function of geometry, quality of internal and external steel, load situation at the moment of gluing, and required strengthening ratio. These graphs are presented in (3), and are available on request.

A much more difficult problem is the design of the anchoring zone. The proposed design method is based on the results of experiments on special shear test specimens, and the translation of these results to the bending shear problem. The test piece is an unreinforced concrete prism with dimensions 150 mm x 150 mm x 300 mm. Two pairs of steel plates are glued on the opposite sides of the prism, to enable a pure shear loading, as explained in ref. 6. The test specimen and the testing apparatus are shown in Fig. 1. Typical results of the measured stress distributions are given in Fig. 2 and 3 for a test piece showing an ultimate load, P , of 87 kN per plate.

In the region of low tensile forces (Fig. 2, curve 1), there is an exponential drop of the resultant force in the plate and of the shearing stresses in the adhesive. In the first zone, shearing stresses reach a maximum of about 2.5 N/mm^2 , while at a greater distance and at higher loads, they can reach 5.5 to 6 N/mm^2 .

At higher loads (curves 2 and 3 in Fig. 2), the stresses in the initial zone of the anchorage do not grow equally with the applied force, but they are reduced by cracking of the concrete. In the initial zone of the joint, the shearing stress increases linearly with the applied force P until it reaches a value of about 1.5 N/mm^2 . This value corresponds to the concrete tensile strength at the surface, measured by the pull-off method (1). The tensile strength at the surface is much smaller than the direct tensile strength, measured by a tensile test on a cylinder or prism. No cracking of the concrete has been observed up to this value of the shearing stress. From this it may be concluded that for a safe pure shear design, the shearing stress in the initial zone may not exceed the surface tensile strength.

Under ultimate load conditions, the shearing stress distribution is given in Fig. 3. Stresses now reach values of 5 to 6 N/mm^2 , which is the direct tensile strength of the concrete.

On the basis of these experimental data as presented in Fig. 2 and 3, we apply the following design rules in pure shear.

- under service load P_s , a triangular stress distribution with a maximum at A is assumed, Fig. 4a. The maximum value will be limited to the characteristic surface tensile strength $R_{sk} = R_{sm} - 1,64 s$. R_{sm} and R_{sk} are the mean and characteristic values of the concrete surface tensile strength, measured by means of the pull-off test. s is the standard deviation calculated for the test results. The value L_1 of the anchoring length can now be calculated from the equilibrium of longitudinal forces.
- At ultimate load P_u , we assume a triangular distribution with a maximum in the middle, Fig. 4b. This distribution nearly approximates the measured stress distribution of Fig. 3 in a cracked specimen. The maximum value is now taken to be equal to the characteristic value R_{tk} of the direct tensile strength of the concrete, measured by a direct tensile test on concrete prisms or by a splitting test. The direct tensile strength of concrete is normally 2 to 4 times the surface tensile strength. With these assumptions, a second value L_2 for the anchoring length can be calculated, and the greater value of L_1 and L_2 will be chosen.

In bending problems, the shearing stresses in the adhesive layer between the external reinforcement and the concrete are partly due to the variation of the bending moments and partly to the introduction of forces in the anchoring zones, Fig. 5.

A typical distribution of the shearing stresses in the anchoring zone is given in Fig. 6. The design of the anchorage is now reduced to the pure shear case in the following way. The critical zone is situated near the free end A of the plate. The force $\Delta P = \sigma_A \cdot A_n$ is calculated, where σ_A is the theoretical tensile stress in the external plate at the end A, and A_n is the cross sectional area of the external steel. This force ΔP is the compressive force which must be superposed on the theoretical forces in the external plate following the scheme of Fig. 5 to obtain the real force distribution, with zero force at the free end A. Under service conditions, the joint behaves elastically and we assume that the compressive force ΔP causes a triangular shearing stress distribution with a maximum τ_a at a. These shearing stresses are added to the theoretical beam shearing stress τ_{th} . The resulting value ($\tau_{th} + \tau_a$) may not exceed the characteristic value R_{sk} of the surface tensile strength of the concrete. With ΔP and the value ($\tau_{th} + \tau_a$), the anchoring length L can be calculated. If this length or the shearing stresses become too large, the thickness of the plate must be reduced and the width of the plate must be enlarged. At the free end A, the re-entering edge causes stress concentrations and accelerates cracking in the concrete layer between the external and internal reinforcement. To avoid this premature cracking, a dowel can be placed to anchor the free end of the plate to the interior concrete of the beam or slab.

If the beam had to be strengthened against bending loads, it is also likely that strengthening against shear forces will be needed, because bending moments as well as shear forces depend on the magnitude of the applied loads. A procedure

for the design of external stirrups is given in (6). There is no general analytical model available for the shear behaviour of reinforced concrete beams. Therefore, the procedure given in (6) is empirical and is based on large scale experiments on beams.

INFLUENCE OF DOWELS IN ANCHORING ZONE

If the bonding operation is well executed, the bending failure of a plated beam will be of the type, shown in Fig. 7.

This type of failure is caused by the stress discontinuity at the plate ends. The crack occurs in the concrete cover, where the concrete quality is the lowest. Crack propagation occurs easily along the longitudinal reinforcement, which presents an internal plane of discontinuity.

One can easily imagine that a dowel, placed at the end of the plates, will stop such a kind of failure phenomenon. It is also clear that the function of the dowel is not to take up the shear stresses between the external steel plate and the concrete. It only must prevent the bending of the plate end, so that the crack can not widen and the concrete itself can transfer all shear stresses through aggregate interlock. To calculate the number and cross-sectional area of the dowels, one can refer to the calculation of the anchoring length. The dowels are designed to take up the shear stresses in the anchoring zone, which exceed the value of the surface tensile strength R_{sk} :

$$n \times f_{dv} \times A_d = (R_{bk} - R_{sk}) \frac{L_2}{2} \quad (1)$$

where : n = number of dowels

A_d = cross-sectional area of dowel

f_{dv} = design shear strength of dowel

To observe the dowel effect, a laboratory test program was conducted where small beams (225 mm x 125 mm x 1700 mm, span 1500 mm, externally bonded steel plate 3 mm x 125 mm x 1300 mm) were loaded in a four point bending test. One dowel with diameter 14 mm was placed at a distance of 40 mm from the plate end. It could be observed that in this case the load bearing capacity of the plated beams increased by about 60 percent. The shear stresses in the anchoring zone are more uniformly distributed, except at low loads, where the dowel action is not yet generated. Experiments have shown that design equation (1) gives safe results. Further research could be done to optimize the design procedure.

The externally bonded steel plates increase the bending stiffness and the magnitude of the compressed cross-sectional area. Bending crack width is also reduced. Consequently the vertical shear capacity is also improved.

Supplementary strengthening can be provided by means of external stirrups, as explained in (6).

RESISTANCE TO HIGH AND LOW TEMPERATURES

Like all cold hardening adhesives, the cold hardening epoxy glue has a poor thermal resistance. The translation of the above statement into practice was done by full-scale temperature loading tests in the temperature range from -20°C to $+90^{\circ}\text{C}$. The epoxy adhesive EPICOL U (Resiplast N.V.) was used in the tests. Table 1 summarises the results of these tests. It appears that for temperatures below the glass transition temperature there is no decrease of the ultimate load. At higher temperatures, however, the situation is different. From a temperature of about 65°C , the glue becomes weaker and more deformable. This is clearly illustrated by the different crack pattern observed in the tests. At lower temperatures, the crack always starts at the end of the plate and moves on into the concrete (Fig. 8). At higher temperatures, the epoxy joint is not able to transfer the shearing stresses from the steel plate to the concrete, and a crack propagates through the epoxy joint, starting at the plate end (Fig. 9).

The safety of the repair is strongly reduced at high temperatures. In most practical cases, a dowel is placed at the end of the steel plates, which is anchored in the concrete. This supplementary safety element will prevent a fast and sudden debonding of the steel plate reinforcement. The application of an insulating cover such as a bituminous felt, or other fire protective measures will be necessary if fire resistance is required.

STRENGTHENING OF REINFORCED CONCRETE SLABS

Strengthening or stiffening of reinforced concrete slabs by means of externally bonded steel strips will increase the amount of steel area in the direction of the strips. Consequently the stiffness in the strip direction will strongly differ from the stiffness in the transverse direction. This orthotropy in the stiffness characteristics has a great influence on the distribution of forces in the plate. The stiffer the slab in one direction, the greater the relative part of the load which will be taken by that direction and also the greater the moments that will be carried in that direction. Orthotropic plate theory can take into account all these modifications.

An analogous problem arises with regular grillages of parallel beams which are placed sufficiently near to each other to allow for the grillage to be considered as a continuous plate. Such elements are called "form-orthotropic". In grillages the available stiffness is virtually spread over the length and the width in order to

obtain a fictitious plate, characterised by an orthotropic stiffness distribution, Fig. 10.

The fundamental equilibrium equation for this type of plate is (3)

$$K_x \frac{\partial^4 w}{\partial x^4} + 2H \frac{\partial^4 w}{\partial x^2 \partial y^2} + K_y \frac{\partial^4 w}{\partial y^4} = p(x,y) \quad (2)$$

where w : deflection of the plate
 $p(x,y)$: distributed load on plate
 $K_x, K_y, 2H$: plate stiffnesses

In this case of asymmetrically stiffened plates, the expressions for the stiffnesses become :

$$K_x = \frac{E h^3}{12 (1 - \nu^2)} + \frac{E h \cdot e_x^2}{1 - \nu^2} + \frac{(EI)_1}{b_x}$$

$$K_y = \frac{E h^3}{12 (1 - \nu^2)} + \frac{E h \cdot e_y^2}{1 - \nu^2} + \frac{(EI)_2}{b_y} \quad (3)$$

$$2H = 2 \times \frac{E h^3}{12 (1 - \nu^2)} + G \times \frac{\beta_1 h_1 b_1^3}{b_x} k_1 + G \times \frac{\beta_2 h_2 b_2^3}{b_y} k_2$$

The notations refer to the geometrical characteristics, as shown in Fig. 11.

We consider a fictitious middle plane which has an eccentricity e_x against the middle plane of the deck plate in the cross section $x = \text{constant}$, and an eccentricity e_y , in the cross section $y = \text{constant}$. I_1 and I_2 are the moments of inertia of the beams ($b_1 h_1$ and $b_2 h_2$) against a rotation axis with eccentricity e_x and e_y respectively. β_1 and β_2 are the numerical constants for the calculation of the torsional moment of inertia of rectangular cross sections. ν is Poisson's ratio of slab.

With the reduction parameters k_1 and k_2 , the decreasing torsional stiffness of reinforced concrete elements after cracking can be taken into account. By cracking the torsional stiffness may drop to one tenth of the original uncracked value. The specific values of k_1 and k_2 are further functions of the reinforcement ratios, of the relation b_1/h_1 and of the relation between the acting bending and torsion moments.

For reinforced concrete slabs, strengthened by external steel strips (Fig. 12), the calculation of the stiffnesses in the uncracked state can be made in the same way as for ribbed slabs. Stiffnesses are calculated with the formulas (4).

$$\begin{aligned}
 K_x &= \frac{E_b h^3}{12 (1 - \nu_b^2)} + \frac{E_b h \cdot e_x^2}{1 - \nu_b^2} + \frac{(EI)_1}{b_x} \\
 K_y &= \frac{E_b h^3}{12 (1 - \nu_b^2)} + \frac{E_b h \cdot e_y^2}{1 - \nu_b^2} + \frac{(EI)_2}{b_y} \\
 2H &= 2 \times \frac{E_b h^3}{12 (1 - \nu_b^2)}
 \end{aligned} \tag{4}$$

Here $(EI)_1$ is the bending stiffness of the reinforcement bars in the x-direction (A_x) against the neutral axis of the cross section $x = \text{constant}$. $(EI)_2$ is the bending stiffness of the reinforcement bars (A_y) and external strips (A_n) against the neutral axis of the cross section $y = \text{constant}$. ν_b is Poisson's ratio of concrete.

In the cracked state, which mostly arises at strengthening projects, it is advisable that neither the transverse influence of the concrete plate nor its torsional stiffness should be taken into account. So the stiffnesses can be simplified as

$$\begin{aligned}
 K_x &= \frac{E_b I_x}{b_x} \\
 K_y &= \frac{E_b I_y}{b_y} \\
 2H &= 0
 \end{aligned} \tag{5}$$

Using this plate model, rectangular, orthotropic slabs under uniformly distributed loads were numerically analysed. The edge conditions and the relation of the stiffnesses were taken as variables. The results of these calculations are presented in graphs (3).

Fig. 13 shows the design graphs for simply supported square plates. The graphs give the extreme values of the bending moments m_x and m_y as functions of the ratio of the stiffnesses K_y/K_x , the ratio B of the torsional stiffness to the bending stiffness K_x ($B = H/K_x$), and the ratio of the spans L_x/L_y . Poisson's ratio was taken equal to zero in the calculations, and $m = E_s/E_c = 15$.

Example

A reinforced concrete slab, all edges simply supported, is loaded by a uniformly distributed load p . The slab characteristics are

$$\begin{aligned}
 L_x &= L_y = 6 \text{ m} \\
 p &= 7 \text{ kN/m}^2 \\
 h &= 170 \text{ mm}
 \end{aligned}$$

The slab can be calculated using the graphs in Fig. 13

$$\begin{aligned} L_x/L_y &= 1 \\ K_y/K_x &\approx 1 \\ \beta &= 1 \\ m_x &= 0.037 p \times L_y^2 = 0.037 (7 + 4.08) \times 36 = 14.8 \text{ kNm/m} \\ m_y &= 0.037 p \times L_y^2 = 0.037 (7 + 4.08) \times 36 = 14.8 \text{ kNm/m} \end{aligned}$$

The amounts of steel A_x and A_y (quality BE 40) needed are

$$A_x = \frac{14.8 \times 10^6}{0.9 \times 140 \times 240} = 489 \text{ mm}^2/\text{m} = A_y$$

Thus an area of nearly 500 mm²/m is required to increase the live load on the plate by 30 percent. We assumed that the plate is cracked in such a way that it is not allowed anymore to take into account the total torsional stiffness of the plate. Since the torsional stiffness never becomes zero in plates, we put the torsional parameter $B = 0.5$.

The strengthening is realized by means of strips parallel to the x-direction.

The distributed load now becomes

$$p_1 = (4.08 + 1.3 \times 7) = 13.2 \text{ kN/m}^2$$

The area of external steel is calculated by means of successive iterations. To start the iteration we take one steel strip with area 3 mm x 150 mm per meter slab width (450 mm²/m). For the concrete in the cracked state we calculate

$$\begin{aligned} I_x &= 197.5 \times 10^6 \text{ mm}^4/\text{m} \\ I_y &= 94.6 \times 10^6 \text{ mm}^4/\text{m} \\ \beta &= 0.5 \\ \frac{K_y}{K_x} &= \frac{94.6}{197.5} = 0.48 \end{aligned}$$

From the graphs we read for this ratio $K_y/K_x = 0.48$

$$m_x = 0.060 \times 13.2 \times 36 = 28.5 \text{ kNm/m}$$

$$m_y = 0.025 \times 13.2 \times 36 = 11.9 \text{ kNm/m}$$

The bending moment m_y is now even smaller than before the load increase because the load is being carried by the stronger direction. The available reinforcement A_y will therefore certainly be sufficient. The bending moment m_x increases from 14.8 to 28.5 kNm/m. If we allow a stress of 160 N/mm² in the external steel plates, the necessary total steel area A_t becomes

$$A_t = \frac{28.5 \times 10^6}{0.9 \times 154 \times 160} = 1285 \text{ mm}^2/\text{m}$$

The total amount of available steel area equals $(489 + 450) = 939 \text{ mm}^2$, which is smaller than the 1285 mm² needed.

In a next iteration we apply an area $A_n = 4 \text{ mm} \times 200 \text{ mm} = 800 \text{ mm}^2$.

One calculates

$$\begin{aligned} I_x &= 264.0 \times 10^6 \text{ mm}^4/\text{m} \\ I_y &= 94.6 \times 10^6 \text{ mm}^4/\text{m} \\ K_y/K_x &= 0.36 \\ m_x &= 0.063 \times 13.2 \times 36 = 29.9 \text{ kNm/m} \\ m_y &= 0.021 \times 13.2 \times 36 = 10.0 \text{ kNm/m} \end{aligned}$$

The bending moment m_y further decreases. In the x-direction a steel amount A_s is needed

$$A_s = \frac{29.9 \times 10^6}{0.9 \times 159 \times 160} = 1305 \text{ mm}^2/\text{m}$$

The available area is $489 + 800 = 1289 \approx 1305 \text{ mm}^2/\text{m}$.

The calculation of the external steel plate cross-section is made in the assumption that the concrete slab is completely unloaded. The calculation can also be made by taking into account the stress situation at the instant of gluing the steel plates. This generally leads to a smaller amount of steel to be glued on (ref. 3).

CONCLUSIONS

The design of adhesive bonding steel plates can be accomplished by simple methods, based on experimental research and numerical modelling. The bending problem is extensively studied, and reliable methods are presented to calculate the cross-sectional areas and the anchorage lengths. The influence of dowels in the anchoring zone is addressed and it was found that only a very limited number of dowels is necessary. As could be expected, the behaviour of the glued connection at elevated temperatures above the epoxy glass transition temperature is poor. However, suitable techniques are available to provide protection to the adhesive bonding plates from excessive temperatures, such as fire.

Using orthotropic plate theory it is possible to design adhesive bonding steel plate strengthening of rectangular reinforced concrete slabs, using steel plates in one direction only. The design graphs present the tool by which advantage can be taken from the stiffness changes, caused by the application of the steel plates in only one direction.

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TABLE 1 — ULTIMATE LOADS AT DIFFERENT TEMPERATURES

Beam nr.	Temperature (°C)	Total load at failure (kN)
4	-20	75
5	20	80
2	50	80
3	80	50
6	90	40

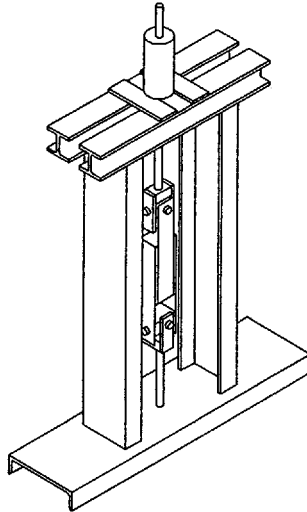


Fig. 1—Pure shear test setup

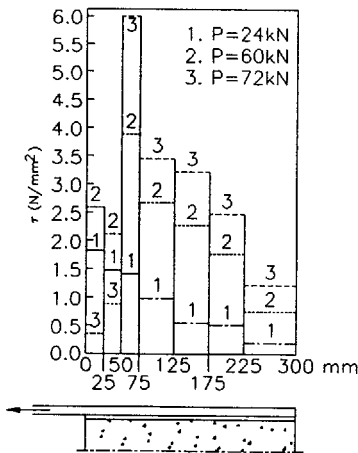


Fig. 2—Average shearing stresses at first loading

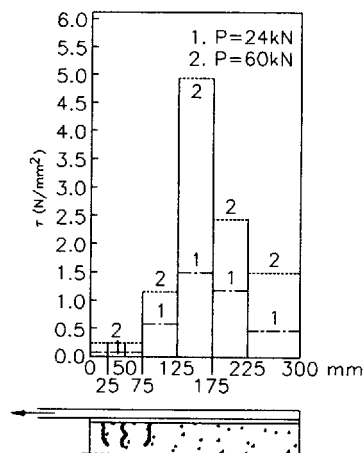


Fig. 3—Average shearing stresses in cracked specimen

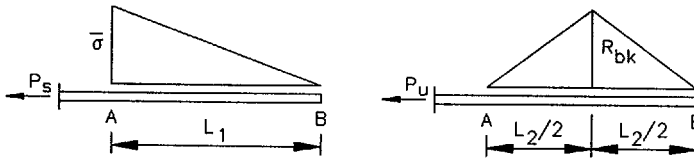


Fig. 4—Assumed stress distribution: (a) at service load; (b) at ultimate load

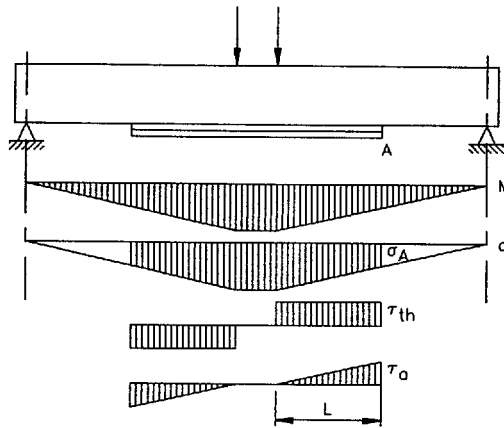


Fig. 5—Moments and stresses in bending

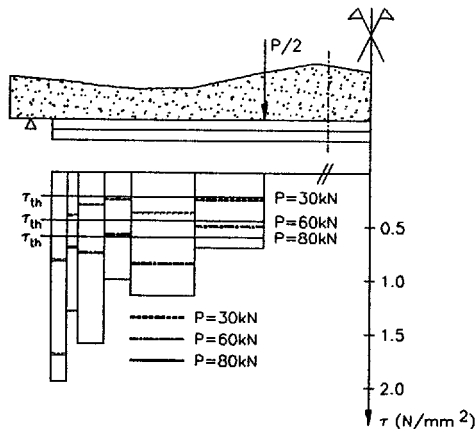


Fig. 6—Typical shear stress distribution in anchoring zone at four-point bending test

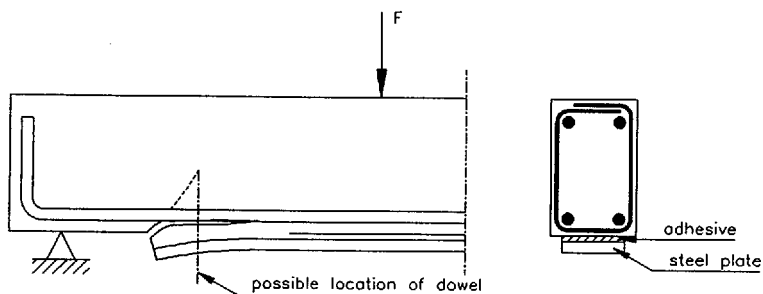


Fig. 7—Ripping off failure in concrete cover along internal reinforcement

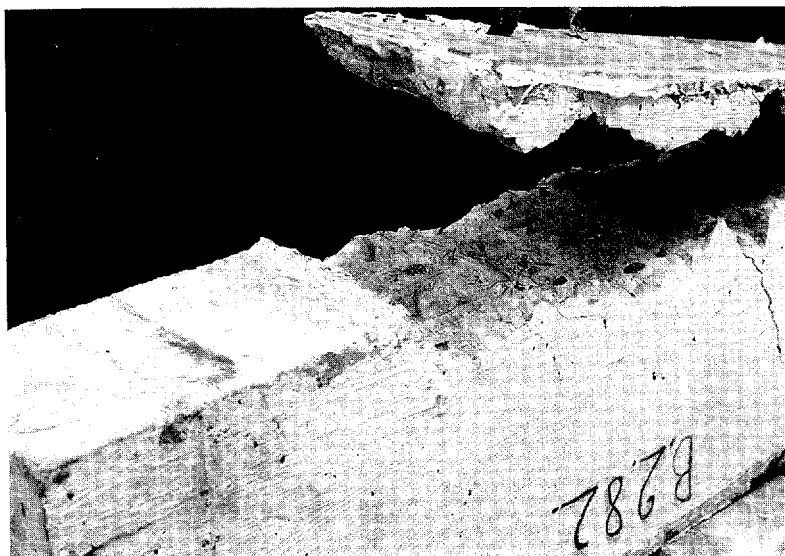


Fig. 8—Crack pattern at normal temperature

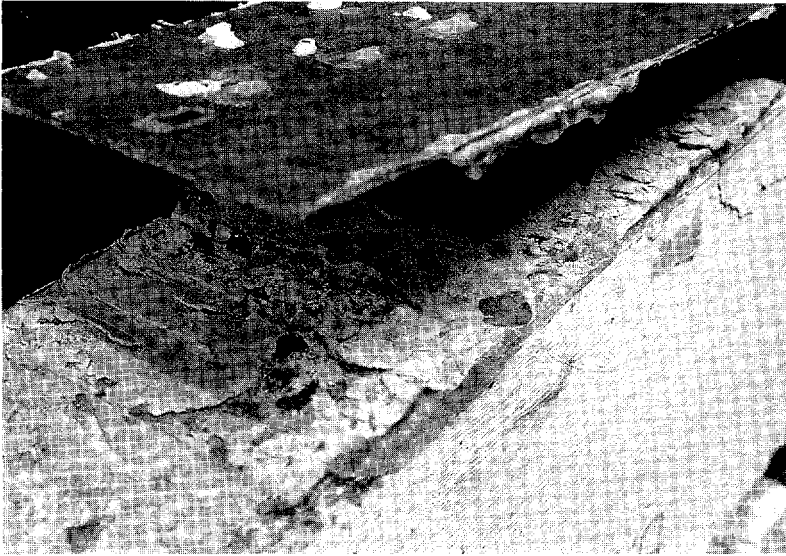


Fig. 9—Crack pattern at higher temperature

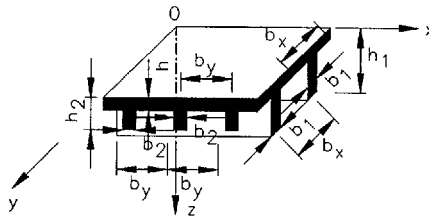


Fig. 10—Reinforced concrete rib slab

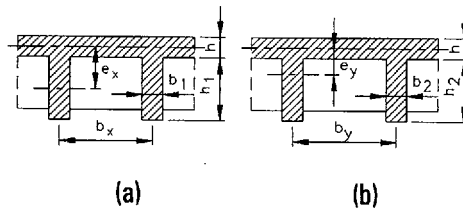


Fig. 11

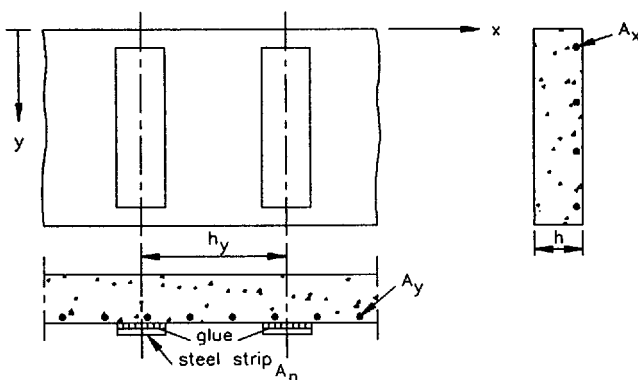


Fig. 12—Reinforced concrete slab, strengthened by means of externally bonded steel strips

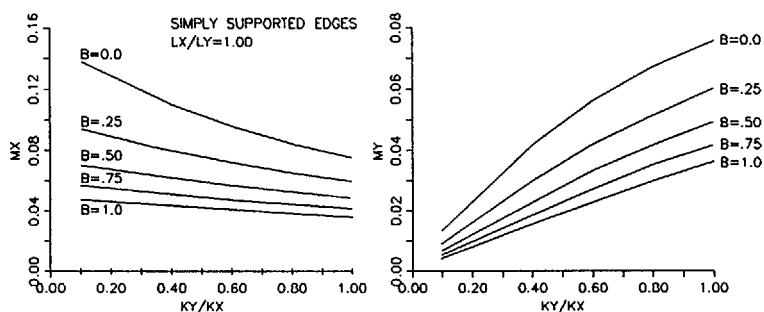


Fig. 13—Design graphs for square slabs with simply supported edges

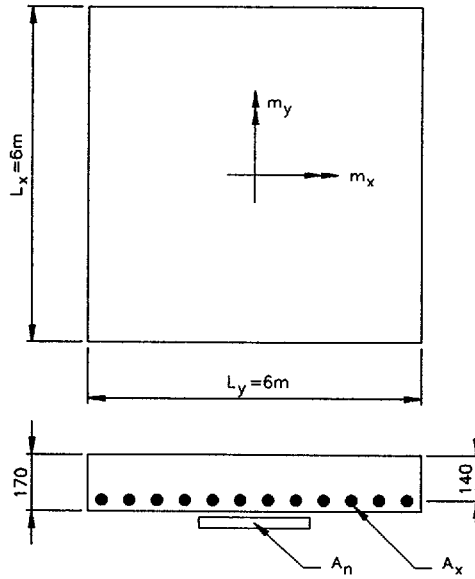


Fig. 14

SP 165-3

Composite Behavior of Partially Plated Beams in the Linear Elastic Range

by Marco Arduini and Antonio Di Leo

Synopsis: In this paper a closed-form solution is obtained for partially plated beams. Linear elastic behavior is assumed for beam, plate and adhesive. The behavior of the adhesive is idealised using the Roberts and Haji-Kazemi model. A system of differential equations is obtained and finally solved by detecting the distribution of shearing stresses through the adhesive layer. All other beam resultants are obtained via equations of equilibrium. The approach has suggested a simple experimental device that enable us to evaluate the stiffness-parameters of the adhesive layer. Finally, the response of the model is compared with numerical solutions by Finite Element Model and the experimental results obtained for flexural tests on partially plated beams. A good agreement is observed.

Keywords: Adhesives; beams (supports); flexural tests; plates (structural members); shear stress; stiffness

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INTRODUCTION

This paper is addressed to the technique of strengthening concrete beams in use, by means of bonding steel plates in the tension face with epoxy resin. This technique was proposed in 1967 by l'Hermite and Bresson [1]. The interest in this proposal is evidenced by the numerous reports published in scientific literature, see for example [2-9]. These last reports refer to experimental results that demonstrate significant improvements in the structural performance. In particular, the recent paper by Oehlers and Moran [10] shows that for assigned stiffness conditions of concrete beam and steel plate, the composite behavior is significantly linear up to the peak load.

From the above, the importance becomes evident of modeling the behavior of partially plated flexural beams (Fig. 1) in the linear elastic range.

THEORETICAL SOLUTION IN THE LINEAR ELASTIC FIELD

In this paper the stress and displacement distributions of the plated beam (Fig. 1) are calculated with a uniformly distributed load. The analysis is obtained in accordance with Eulero-Bernoulli hypotheses, applied to the unplated-beam and to the steel plate. Finally, the behavior of the adhesive layer is assumed in accordance to Roberts and Haji-Kazemi theory [11].

The composite infinitesimal element of the plated beam consists in the sub-elements shown in Fig.2. The material denoted by c represents the beam to be reinforced, whereas the material denoted by a denotes the adhesive layer and, finally, s denotes the steel plate. In Fig. 2 the positive directions of stress resultants and displacement components are shown for