THE SHEAR STRENGTHENING OF RECTANGULAR REINFORCED CONCRETE BEAMS USING BONDED EXTERNAL REINFORCEMENT

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ABSTRACT

This paper describes part of a programme of research aimed at investigating the potential for strengthening reinforced concrete members in shear by means of externally bonded web plates. This may be a useful strengthening technique following the assessment of older bridge and building structures designed to outdated codes of practice. An experimental programme has been undertaken to investigate the influence of shear-span to depth ratio and beam size on the service and ultimate behaviour of shear plated concrete beams. Overall, the study has shown that the provision of adhesively bonded external shear plates has the potential to significantly increase shear capacity. However, the provision of adequate plate anchorage is necessary for the successful application of the method.

KEYWORDS

Concrete, steel, shear, adhesives, strengthening

INTRODUCTION

Existing concrete structures may require strengthening or stiffening in order to increase their ultimate flexural or shear capacity, or to control deflections and cracking. One method for providing this enhanced capacity is to adhesively bond steel plates to the concrete surface. The technique of using externally bonded steel plates has been used successfully worldwide for over 30 years. Advantages of external reinforcement over other methods include minimum effect on headroom, low cost, ease of maintenance and the ability to strengthen part of the structure whilst still in use. Since the early 1990s, a notable expansion of interest in the technique by clients, consultants and contractors has occurred, partly as a result of the demand for stronger highway bridges to cope with heavier vehicles.

The authors' contributions to the development of steel plate bonding have been concerned with identifying the engineering property requirements for structural adhesives (Mays and Hutchinson, 1988), in particular the fatigue and creep performance of epoxy resin adhesive joints under adverse environmental conditions (Mays, 1990). They have also recently been involved in a programme of structural composites research which has demonstrated the viability of using carbon fibre reinforced plastic pultrusions in place of steel for flexural strengthening. Other work has focused on methods for evaluating candidate adhesives and research is currently in progress concerning the effect of traffic vibration during cure on the integrity of the adhesive bond.

To date most of the plate bonding applications in both buildings and bridges have been to enhance the flexural capacity of the beam by bonding plates to the tension face. This is a well documented method and guidance for designing such schemes is available in many countries. However, structures designed prior to the introduction of limit state design codes in the 1970s are likely to be deficient in shear reinforcement due to the inadequacy of the shear design equations in codes of practice before that time. Some plate bonding works have included steel plates bonded to the side faces of the beam in order to enhance their shear capacity, an empirical approach being taken for the design of the steel plates.

Mays and Peh (1989) undertook a study to assess the potential of four separate shear strengthening methods for beams. These were: continuous steel sheets bonded to the webs over the length of the shear spans; individual steel strips bonded to the webs; grouted steel bars inserted from the top flange over the full effective depth of the beam; 'C-shaped' links inserted from the web faces. The results showed that the shear capacity of rectangular reinforced concrete beams may be significantly enhanced by the use of continuous steel sheets bonded to the web but it was recommended that these be extended over the full span length of a beam but not necessarily over the full effective depth.

Sharif et al (1995) investigated different arrangements of bonded steel plates with the aim of eliminating shear failure in rectangular beams which had been pre-cracked in shear. The strength of all repaired beams was increased and the degraded stiffness of the beams was restored. However, brittle failures occurred where the external reinforcement consisted of vertical steel strips or steel sheets bonded to the webs over the length of the shear spans. Shear failures were only eliminated when the steel sheets fully encased the shear zone in the form of a bonded 'U' shaped steel jacket.

The research described in this paper has been undertaken in response to the earlier recommendations, by including the variables of shear span to depth ratio and the size of rectangular beams. Work in progress is investigating the shear strengthening of 'T' beams.

INFLUENCE OF SHEAR-SPAN TO DEPTH RATIO

It is well established in both British and American design practice (Evans and Kong, 1967) (ACI-ASCE Committee 426, 1973) that the failure mode of rectangular reinforced concrete beams without shear reinforcement is strongly dependent on the shear-span/depth ratio, a/d:

(a) for $a/d > 6$, failure usually occurs in bending;
(b) for $6 > a/d > 2.5$, the development of a flexural crack into an inclined flexure-shear crack results in diagonal tension failure;
(c) for $2.5 > a/d > 1$, a diagonal crack forms independently but the beam remains stable until shear-compression failure occurs;
(d) for $a/d < 1$, the behaviour approaches that of deep beams

In addition to the shear-span to depth ratio, the contribution of the concrete to the shear strength, V_c, is dependent on a number of other factors including the concrete strength (f_c) the main tension reinforcement ratio (ρ) and the beam size (b,d). These factors are represented in both the ACI and BSI design formulae for V_c.

The shear strength of reinforced concrete beams may be substantially increased by the provision of suitable shear reinforcement, usually in the form of stirrups or links, which serve to intercept the diagonal shear crack. Thus, the external shear force, V, is resisted partly by the concrete, V_c, and partly by the shear reinforcement, V_s, such that

$$V = V_c + V_s$$

Experimental Programme

An experimental programme was conducted which involved the shear strengthening and testing to failure of 15 rectangular concrete beams of cross-section 250mm deep by 155mm wide. To account for the different potential types of shear failure (diagonal tension, shear compression and deep beam failure) three alternative shear span to depth ratios were used (4.0, 1.8 and 0.8, respectively). In each case, control beams (one without and one with conventional shear reinforcement) together with three beams strengthened with external shear plates were tested.

The 3mm thick external mild steel plates covered both vertical concrete surfaces over the complete depth and the entire span of the beams. Following cleaning and grit blasting of both steel and concrete surfaces, the plates were bonded in position using Sikadur 31 epoxy adhesive, 1mm thick. No mechanical fastenings were used. After an appropriate adhesive curing period, the beams were load tested to failure in four point bending. Concrete and plate strains were monitored using electrical resistance rosette strain gauges.

A summary of the beam details and testing arrangements is provided in Table 1.

Table 1. Beam details and testing arrangements

Beam type	Shear reinforcement	Main tension reinforcement	Overall span(m)	Shear span(m)	Shear-span to depth ratio
1a 1b 1c	none 3mm thick plates 8mm links at 100mm centres	2T25 & 2T16 "	2.0 " "	0.675 " "	4.0 " "
2a 2b 2c	none 3mm thick plates 10mm links at 150mm centres	4T12 " "	0.9 " "	0.3 " "	1.8 " "
3a 3b 3c	none 3mm thick plates 10mm links at 150mm centres	4T12 " "	0.45 " "	0.15 " "	0.8 " "

Results and discussion

A summary of the results from this series of tests in terms of ultimate load capacity is provided in Table 2.

Table 2. Rectangular Beam Results

Beam type	Shear-span to depth ratio	Shear reinforcement type	Predicted load capacity (kN) and failure mode	Average failure load (kN) and failure mode	Percentage increase in ultimate capacity compared with control
1	4.0	none links plates	77 (shear) 186 (shear) 305 (flexure)	75 (shear) 240 (shear) 265 (shear)*	- 220 253
2	1.8	none links plates	74(shear) 196 (shear) 365 (flexure)	150 (shear) 200 (shear) 367 (flexure)*	- 33 145
3	0.8	none links plates	395 (shear) 401 (shear) 622 (shear)	500 (shear) 500 (flexure) 650 (flex/sh)*	- nil 30

*= mean of 3

The predicted load capacities were determined from the lesser of:

(a) the flexural capacity based upon conventional plastic theory for reinforced concrete assuming an ultimate compression strain of 0.0035, including the contribution of the web plates in both the flexural tension and compression zones;

and

(b) the shear capacity based upon:

(i) for beams with conventional link shear reinforcement:

- for a/d = 4.0, the truss analogy method of BS8110(1985) but with the material partial safety factors removed;
- for a/d = 1.8, truss analogy but with enhancement for loading close to the support in accordance with BS8110;
- for a/d = 0.8, deep beam theory according to Kong and Evans (1987);

(ii) for beams with bonded plate reinforcement, the design approach proposed by Bresson (1971) modified by using stress transfer curves which have been updated to account for modern adhesives and preparation techniques.

In all cases, the addition of external shear plate reinforcement was capable of providing at least as good an enhancement in shear capacity as that provided by conventional links. The failure loads of plated beams were in excess of predicted values, with the exception of beam type 1 where the theory overpredicted by 15%. Strain measurements provided an indication of the magnitude and orientation of the principal strains in the shear spans of plated beams as illustrated in Figure 1. The loading arrangement and failure mechanism of a plated rectangular beam of type 1 is shown in Figure 2.

INFLUENCE OF BEAM SIZE

It has been shown by Kani (1967) and Taylor (1972) that larger beams are proportionally weaker in shear than smaller beams; that is, the ultimate shear stress reduces with beam depth. It is believed that this is because the aggregate interlock contribution to shear strength, V_c, does not increase in the same proportion as the beam size. Design shear stress values in BS8110 allow for the influence of the effective depth, d. The proportion of the strength that the bonded shear plates contributes is also likely to change.

Experimental programme

The relative effectiveness of bonded plates in strengthening beams of a larger effective depth, but with shear-span to depth ratios of 1.3 and 0.8, was also assessed experimentally. Reinforced concrete beams 2370mm long by 400mm deep by 175mm wide and reinforced with 2 no. T32 bars in the tension zone were employed. All six beams contained 6mm diameter mild steel links at 80mm centres in the shear spans as shown in Figure 3. Two beams were used as unplated controls, one at each of the shear-span to depth ratios.

Two beams were strengthened using 4mm thick mild steel plates over both vertical surfaces over the complete depth and the entire span of the beams. 6mm thick mild steel plates were applied to the remaining two beams. In all cases the steel was degreased and grit blasted, and the concrete gritblasted, prior to bonding with Sikadur 31 epoxy adhesive, 1mm thick. The adhesive was allowed to cure for 7 days at room temperature prior to load testing. Electrical resistance strain gauges were attached to the internal reinforcing steel and the external plates. Linear transducers were used to monitor deflections at midspan and at the load points.

Results and discussion

A summary of the results from this series of tests in terms of ultimate load capacity is given in Table 3.

Table 3. Large Rectangular Beam Results

Beam type	Shear-span to depth ratio	Shear reinforcement type	Average failure load (kN) and failure mode	Percentage increase in ultimate capacity compared with control
2	1.3	nominal links	765 (shear)	-
		4mm plates	1314 (shear)*	72
		6mm plates	1452 (shear)	90
3	0.8	nominal links	1422 (shear)	-
		6mm plates	1393 (shear)	-2

*= mean of 2

All three plated beams type 2 failed in shear. Failure was caused by tensile splitting of the concrete cover beneath the plate leading to plate debonding. The addition of 4 mm thick plates generated an increase in shear capacity of approximately 70% as compared with an increase of between 85% and 145% with 230mm beams (see Table 2), depending on whether the plated beam results are compared with the controls with no links or the beams with conventional shear links. The use of 6mm thick plates in beam type 3 resulted in premature plate debonding, thought to be due to insufficient anchorage beyond the support.

Figure 4 shows the load vs midspan deflection characteristics for the type 2 beams from which it can be seen that the addition of plates has not only served to stiffen the beams but also leads to a more ductile failure mode. Indeed, the strains in the main tension reinforcement were approaching the yield strain of 2100$\mu\epsilon$ at failure, although the maximum plate strains recorded were of the order of 1000$\mu\epsilon$. Thus, the full potential of the plates is not being realised at ultimate which would suggest that in theory thinner plates could be used. However, practical experience has shown that use of plates less than 3 to 4mm thick can result in unacceptable distortion during the grit blasting process.

The addition of plates also caused a significant improvement in service load of type 2 beams taken as the load at which a maximum crack width of 0.1mm occurred. For the 4mm and 6mm thick plates, the percentage increases in service load over the control beam were 280% and 370%, respectively.

CONCLUDING REMARKS

An experimental programme has demonstrated that it is possible to achieve significant increases in the shear capacity of reinforced concrete beams by bonding steel plates to both vertical faces. This improvement in ultimate capacity reduces with the shear-span to depth ratio. In the majority of cases the failure loads were in excess of the predicted capacities.

There is some evidence to suggest that the improvements in shear capacity are sensitive to beam size, although further work is necessary to quantify this effect. The addition of bonded plates also resulted in a stiffened section with greatly increased service loads, based upon a crack width criterion.

The practical requirements of plate thicknesses greater than 3 to 4mm mean that the full potential of the plates as shear reinforcement is not realised. Results emphasize the importance of providing sufficient plate anchorage on either side of the shear failure zones otherwise premature debonding, particularly with the low shear-span to depth ratios associated with deep beams, can occur. In such circumstances it may be necessary to utilise mechanical fastenings to supplement the anchorage available. Further work is in progress utilising 'T' section beams.

ACKNOWLEDGEMENTS

The authors acknowledge the financial support provided for this research by the UK Engineering and Physical Sciences Research Council. The investigations into the influence of beam size were undertaken in collaboration with the University of Dundee, Scotland.

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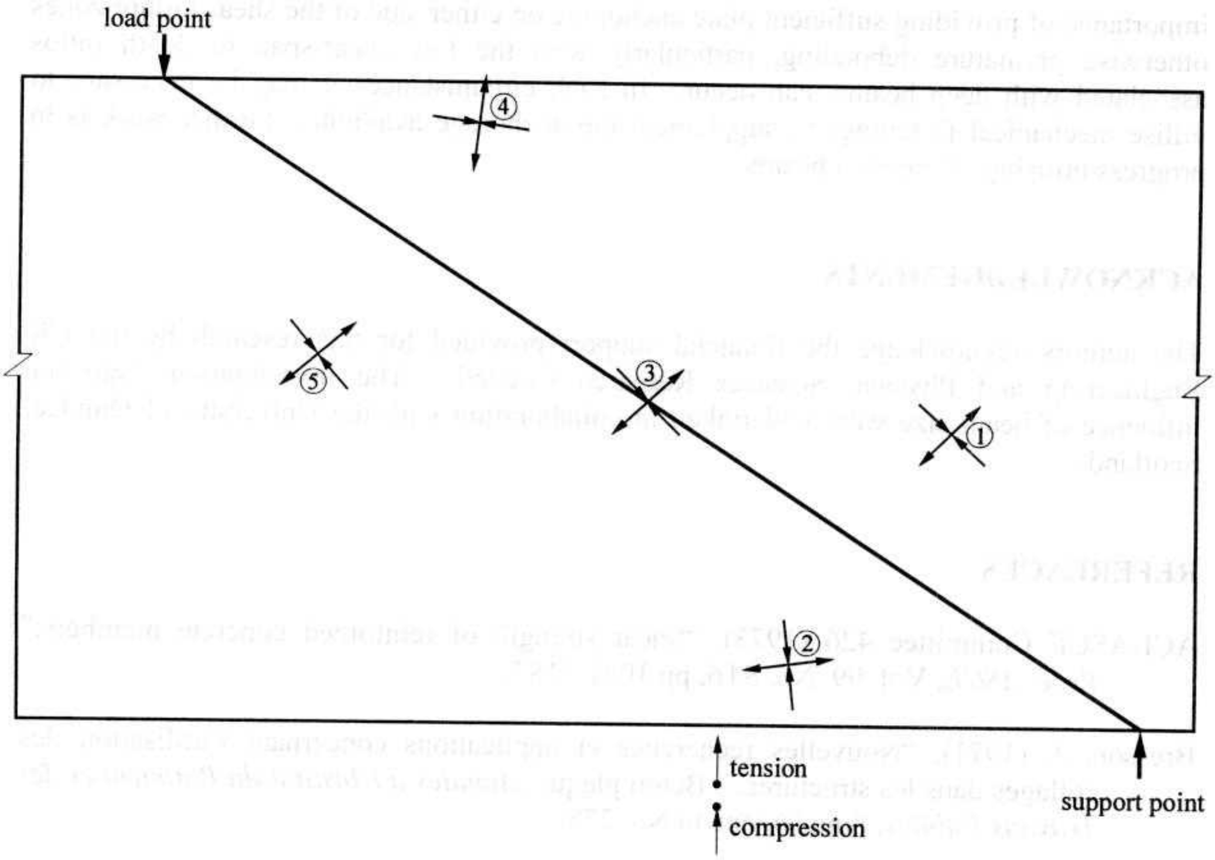

Figure 1 Direction of principle strain on the bonded external steel plate of a strengthened reinforced concrete beam.

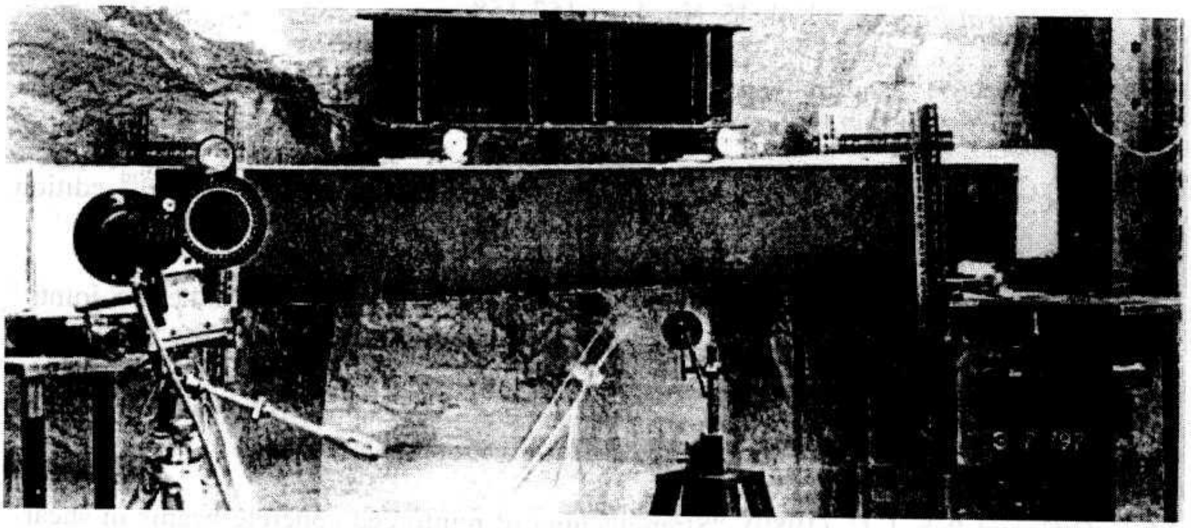

Figure 2 Loading arrangement and failure mechanism for plated rectangular beam.

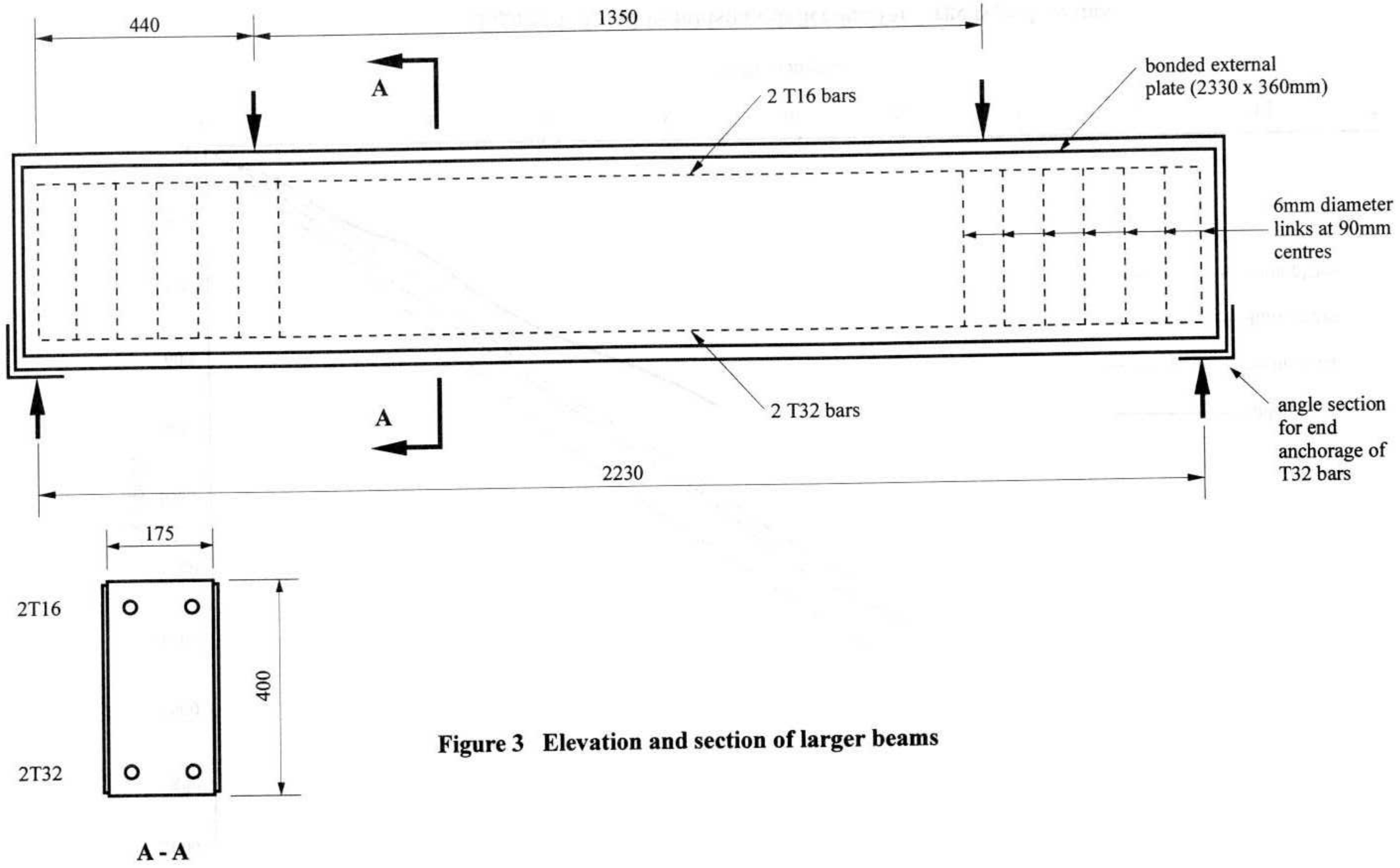

Figure 3 Elevation and section of larger beams

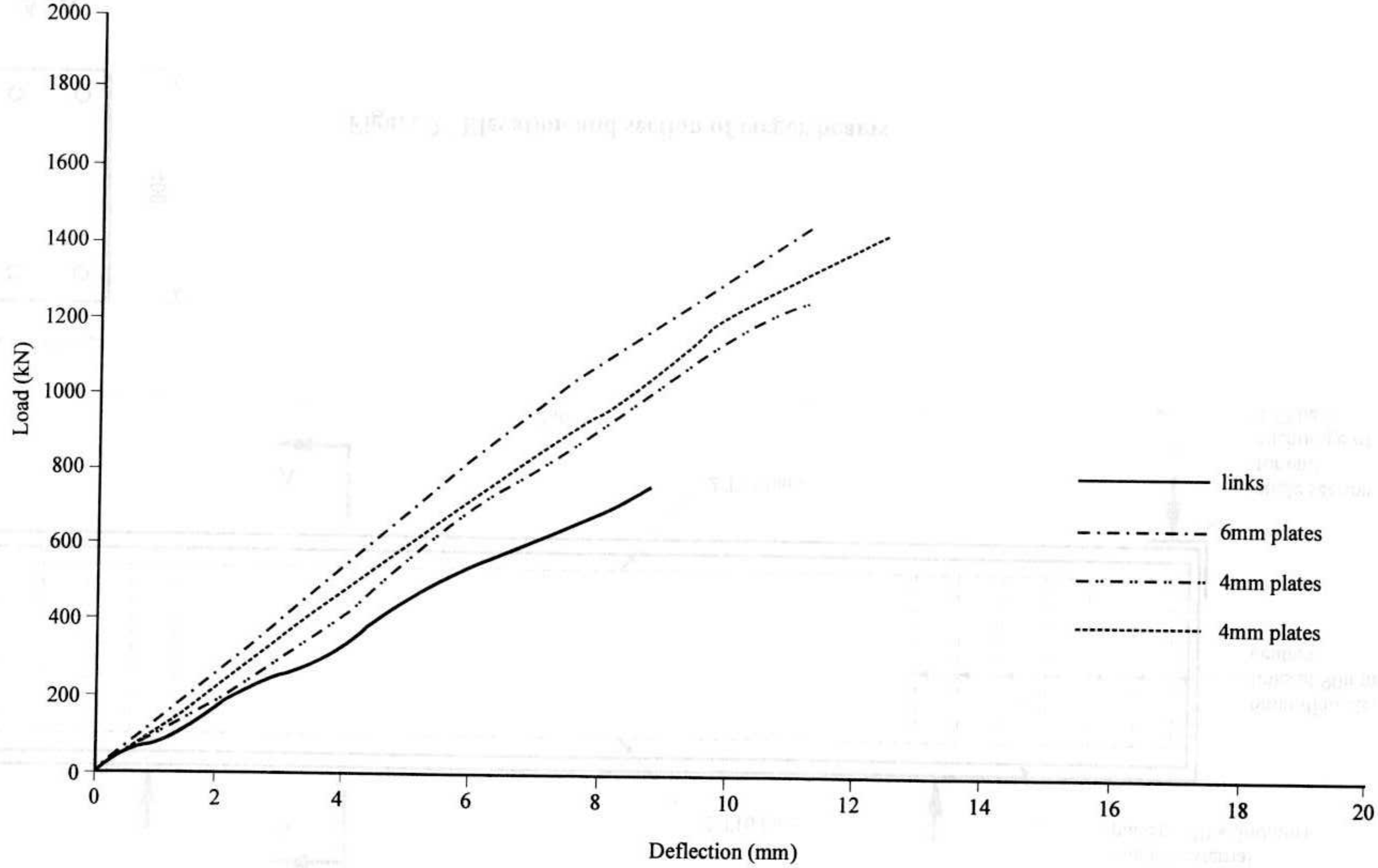

Figure 4 Load vs midspan deflection for large type 2 beams