LOAD-CARRYING CAPACITY OF REINFORCED CONCRETE BEAMS WITH ADHESIVELY BONDED STEEL PLATES

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Abstract

To evaluate the load-carrying capacity of reinforced concrete beams with externally bonded steel plate, the influence of the bonded steel plates to the soffit on the shear strength of the beams, and the effect of shear strengthening using wing-type side steel plates for stocky concrete beams (broad and low web) have been examined. From the results, it was found that the shear strength is properly evaluated by shear capacity equations for non-plated reinforced concrete beams unless a crack develops at the edge of the steel plate, and that the side plate bonding is likely to have a potential to be effective in strengthening for the stocky beams without stirrup. Additionally, loading tests using two 75-year-old deteriorated reinforced concrete beams with adhesively bonded steel plates to the soffit were carried out. The bonded steel plates no longer contributed to the load carrying capacity after the joints between the steel plates failed due to debonding.

Introduction

In Japan, a number of the concrete girders and decks of existing highway bridges were strengthened with adhesively bonded steel plates or fiber reinforced polymer (FRP) sheets when design loads were changed or when the members deteriorated (Figs. 1 and 2). For the efficient management of existing bridges, the load-carrying capacity of not only conventional concrete beams but also concrete beams with the bonded steel plates should be properly evaluated.

The late 1960s, several researchers dealt with the strengthening method that steel plates were adhesively bonded with a two-part epoxy resin adhesive on the soffit of reinforced concrete beams. In those days, because the fatigue deterioration of concrete decks was frequently found on the Japanese highway bridges, experimental studies on the steel plate bonding technique applying to the bridge decks were carried out.[1-4] Then the FRP sheets were not major materials for construction. The steel plate bonding immediately became a major tool to improve the durability of bridge decks, having the advantage of applicability in service, and the efficient strengthening with the minimum change in

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appearance. The strengthening method using FRP sheets for reinforced concrete was developed in early 1990s. Currently, the materials became used commonly to improve the durability of bridge decks after a revision of the legal maximum weight for trucks from 20 tons to 25 tons in 1993, and the ductility of existing bridge piers after the great Hanshin earthquake in 1995. Extensive research works were carried out not only in Japan, but also in the world. Based on the results, several design guidelines for strengthening reinforced concrete members using FRP have been established.[5-8] On the other hand, the effects of the steel plate bonding technique has not been so sufficiently identified as a guideline could be established. In a research program from FY2008 to 2010, the effect of shear strengthening using the steel plate bonding on reinforced concrete beams was examined. This summary report provides the outlines of three series of experimental studies focusing on; Series I: the influence of the steel plate bonded to the soffit on the shear capacity of reinforced concrete beams (further for the evaluation of plated concrete decks), Series II: the effect of side steel plate bonding on the shear capacity of stocky reinforced concrete beams,[9] and Series III: the load-carrying capacity of two 75-year-old deteriorated concrete girders with the steel plate bonded to the soffit.[10]

Series I: Influence of Steel Plate Bonded to the Soffit on Shear Capacity of Reinforced Concrete Beams

Test Program: A test scheme and details of specimens are shown in Fig. 3 and Table 1. The height of the specimens, except Specimens A-7 and A-8, was set as 190 mm, referring to the thickness of the typical bridge decks built in 1960s. No stirrup was arranged except that on the support points for arranging main reinforcing bars. Specimens A-1 to A-8 had two longitudinal reinforcing bars. For Specimens A-4 to A-8, the steel plate with a thickness of 4.5 mm was adhesively bonded on the soffit of concrete beams. The steel plate and an adhesive layer for every beam were as wide as the beam with a width of 150 mm. The thickness of the adhesive layer was 5 mm. The distance from the center of each the support point to the end of the steel plate x_a was 80 mm, and that from the edge of a steel plate for the support was 30 mm.

The test parameter of Specimens A-4 to A-6 is the ratio of shear span a to effective depth d_s, which excludes the dimension of the steel plate. The effective depth and shear span of Specimens A-7 and A-8 ware larger than those of Specimen A-5 with the same a/d_s ratio. Control specimens A-1 to A-3 with no steel plate had different reinforcement ratios, respectively.

Specimens B-4 to B-6, and B-8 had a main reinforcing bar and a doubly reinforcing bar, the steel plate being bonded to the soffit. The thickness of the steel plate of Specimens B-4, B-5, and B-6 was 4.5, 6, and 12 mm, respectively. Specimens B-4 to B-6 were simply supported on the bonded steel plate, as the shear span was 400 mm. Then the bonded steel plate had no curtailment near the supports. For Specimen B-8, the steel plate was bonded after the concrete beam had been precracked under cyclic loading with a shear span of 500 mm. Specimen B-8 was monotonically loaded with a shear span of 1000 mm, having the curtailment of the steel plate with a distance to the center of the support x_a of 80 mm.

Plate bonding was carried out at three weeks or more after casting. Anchor holes were drilled and mechanical steel anchors were installed on the soffit of concrete beams, which was roughened with a hand grinder. The steel plate was supported with the anchors as the gap to the surface of concrete was held with 5 mm spacers. Side openings ware capped with epoxy resin putty. The epoxy resin adhesive was filled into the gap by grouting. In Specimens A-4 to A-8, inorganic zinc-rich primer for protecting the steel plate up to bonding remained similarly to practice. The steel plates for Specimens B-4 to B-6, and B-8 were not coated.

Loading tests were conducted at a week or later after the grouting. The putty on the sides was ground; that at the ends of the steel plates remained. The tensile-shear bond strengths of the epoxy resin adhesives were 12.3 MPa to 18.4 MPa.

Results: All test results are also shown in Table 1. Specimens A-4 to A-6 failed due to cracking at the end of steel plate (Fig. 4).The shear force at cracking at the end of the steel plate V_{p2} may be calculated based on a model provided by Tumialan et al.[11] In the model, the maximum (tensile) principal stress acting on the concrete near the end of the bonded steel plate is estimated based on the approximate solution by Roberts[12], being compared with the tensile strength of concrete. Although Tumialan et al.[11] applied the modulus of rupture f_r to the threshold tensile strength, the splitting tensile strength f_t is applied in this report, because it is more understandable for the model. Figure 5 shows the relationship between the calculated shear force at the cracking V_{p2} and the experimental results V_{ex} including the results of Specimens A-4 to A-6 and the other experimental results obtained from previous research[13-24] except the results of specimens with a ratio x_a/a of more than 0.25. The calculated values V_{p2} are somewhat larger than the experimental values when the value is larger. However, considering that the previous data contain the several unknown parameters, it appears that the shear force at the cracking can be properly estimated. Incidentally, the database based on the previous research shows that the results of specimens with a ratio x_a/a of more than 0.25 showing debonding failure do not depend on the value of V_{p2}, rather they mainly depend on the ultimate flexural strength of non-plated reinforced concrete beams.

In Fig. 6a, the results of specimens exhibiting flexure-shear failure in concrete are shown in relation to the calculated shear strength V_{sh} based on the conventional empirical equation[25] for estimating a shear force at flexure-shear failure. The influence of the bonded steel plates is estimated by substituting the dimensions containing the steel plate for the calculation of the reinforcement ratio and the effective depth in the equation. The other triangle symbols indicate the results of control specimens containing Specimens A-1 to A-3 and the previous research related. For the results of specimens with a ratio a/d of 1.7 to 1.75, the other empirical equation for deep beams[26] is applied. From the results, it was found that the shear capacity of the plated beams can be estimated by the conventional equations. For reference, the calculated shear strength based on an equation in ACI318 (Eq. 11-5)[27] with a factor of 1.3 is also compared with the test results except the deep beam in Fig. 6b.

Series II: Effect of Side Steel Plate Bonding on Shear Capacity of Stocky Reinforced Concrete Beams[9]

Test Program: Configurations of specimens and a test setup are shown in Figs. 7 to 8. Of the same dimension of two reinforced concrete beams, Specimen S, of which wing-type steel plates was adhesively bonded on the sides of web at both shear spans. The other beam with no steel plate was named Specimen N. The concrete beams were designed as a simply supported T-shaped beam with a stocky cross section consisting of a web width of 600 mm and a height of beam of 850 mm, containing ten No.11 longitudinal reinforcing bars with a yield point of 534 MPa. For both specimens, four 9 mm dia. stirrups with a spacing of 250 mm were arranged at a shear span (right side in Fig. 8), no stirrup being at the other shear span. Similarly to the old reinforced concrete bridges, all the stirrups were round steel bars with a yield point of 292 MPa. The compressive cylinder strength of concrete was 33 MPa.

Four steel plates with a size of 2400 mm x 520 mm x 4.5 mm and a yield point of 360 MPa, coated by inorganic zinc-rich primer, were prepared. The procedure for bonding was the same as Series I. The tensile-shear bond strength of the epoxy resin adhesive was 16.1 MPa. Twelve steel anchors with a size of M10 and an embedded length of 70 mm for supporting each the steel plate at grouting were preinstalled before casting. The thickness of adhesive of 5 mm was kept.

Two point loading were monotonically applied to the beams with a shear span of 2 m. Cracking inside the web concrete covered by the steel plates were monitored using molded gauges and gauges on the stirrups. After shear failure occurred at the shear span with no stirrup, once unloaded. Then a vertical restrainer consisting of four tendon bars with 13 mm dia. were installed at 1/3 of the shear span with no stirrup from the near loading point. Reloading beyond the yielding of stirrups became available by using the restrainer, although finally the flange yielded at the shear span with no stirrup.

Results: Relationships between load and deflection at midspan for both specimens are shown in Fig. 9. Cracks observed on the web at the first peak due to shear failure at the shear span with no stirrup are shown in Fig. 10. The load at each event in the loading tests is summarized in Fig. 11. The load at diagonal cracking (detected with the molded gauges in Specimen S) was attributed to neither the stirrup nor the bonded steel plate. For the web without stirrup, the bonded steel plates showed an appreciable effect of improving the shear capacity, which was approximately 1.5 times as high as that of the non-plated specimen.

The side steel plates of both shear spans were debonded at similar loads regardless of stirrups, while the debonding of the steel plates on the web with stirrups did not immediately expand compared with that on the web without stirrup. At the shear span with stirrups, even after debonding, the steel plates were to some extent likely to contribute to the delay in failure. The stirrups, however, showed ductile behavior compared with an estimated level based on the modified truss theory despite the bonded steel plates.

Series III: Loading Tests of Old Concrete Beams with Steel Plate Bonding[10]

Outline of test beams: To examine the load-carrying capacity of old reinforced concrete beams with externally bonded steel plates to the soffit, six beams with a span length of 10 m were taken from the first and seventh spans of a concrete highway bridge built in 1935 (Fig. 1). In about 1981, steel plates were bonded to the soffit of most decks and beams in the bridge. Because the significant deterioration of the decks and beams was found when the administrator was considering the delivery of the bridge to the other small administrator, eight deteriorated spans of the bridge were replaced in 2010. According to the results of the investigation and observation before the removal, the degree of the deterioration of eight spans differed widely from each other. On the beams at the first span, corrosion of steel plates, water leakage and cracks in concrete were widely observed. The beams at the seventh span less deteriorated than that at the first span. Of six beams obtained, a beam from the seventh span and another beam from the first span, named Specimen S1 and S2, respectively, were tested.

Observations and survey of materials: Cracks observed on the test beams before the loading test and each cross section cut after the test are shown in Fig. 12. Coating partially covering web concrete was removed for the observations. The cracks were widely observed on the web of Beam S2. The crack depth, however, was found to be shallower than that concerned. Innumerable horizontal cracks were found in the flanges of both beams. Hammer soundings detected the debonding area of the steel plates as shown in Fig. 12. From a partial dissection survey conducted after the loading tests, two 8 mm dia. stirrups with a spacing of approximately 200 mm were found. The mechanical properties of concrete cores at uncracked parts, reinforcing bars and the steel plate taken from the tested beams are shown in Tables 2 to 3. It should be noted that the modulus of elasticity of the concrete cores taken from Beam S2 was considerably lower than that of Beam S1.

Test program: Two point loading were carried on the beams with a span length of 10 meters as the original span. Two strain gauges were mounted on the main reinforcing bars at a loading point section, after the web concrete was dug as small as possible. Asphalt surfacing remained at loading, because the removal would affect the cracked flange, and an additional preparation for the surface would be required. The tests were carried out in winter. For Beam 2, vertical restrainers were installed at the joints between the bonded steel plates in order to temporarily attempt to mitigate the debonding of the splice plate, although no effect was found from the result.

Results: Relationships between load and deflection at midspan of both the old beams are shown in Fig. 13. In Table 4, the maximum loads are compared with calculated ultimate loads, which were obtained using the measured dimensions and the properties of materials, except the asphalt surfacing. The sign of debonding of splice plates in Beam S1 was detected at 400 kN by using LVDTs (Fig. 14a), and in Beam S2 at 320 kN. After the sign of debonding in Beam S1, the load further increased up to the ultimate flexural capacity of the conventional reinforced concrete beam with no deterioration, while the rigidity significantly decreased. The flexural capacity of the deteriorated beam S2 was 8% lower than that of Beam S1, being 5% lower than the estimated value. After the peak, yielding of the main reinforcing bars was recognized by strain readings, and the flange was moderately crushed (Fig. 14b).

Conclusions

1 - From the test results and the previous research, it was found that the shear capacity of reinforced concrete beams with a steel plate bonded to the soffit is properly evaluated by substituting the dimension containing the steel plate for the calculation of the reinforcement ratio and the effective depth in the shear capacity equations for non-plated reinforced concrete beams, unless a crack develops at the edge of the steel plate.
2 - The side plate bonding is likely to have a potential to be effective in strengthening for the stocky reinforce concrete beams without stirrup. The effect of the side steel plates, however, may be limited depending on the web reinforcement ratio in existing beams.
3 - In the results of loading tests using old reinforced concrete beams, the bonded steel plates no longer contributed to the load carrying capacity after the steel splice plates failed due to debonding.

Acknowledgments

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References

1. Ohta, M., Aizawa, I., Momota, K., and Nakamura, T., *Experimental Study on Strengthening Using Steel Plate Bonding for Reinforced Concrete Decks*, Tech. Note of PWRI, No. 681, April 1971. (in Japanese)
2. Ishitani, T., Higai, T., and Hosoda, K., *Experimental Study on Strengthening Methods for Reinforced Concrete Decks, -Steel Plate Bonding Technique-*, Report of Japan Highway Corporation Laboratory, pp.150-157, Dec. 1975. (in Japanese)
3. Iioka, Y., and Higai, T., *Experimental Study on Strengthening Methods for Reinforced Concrete Decks, -Steel Plate Bonding Technique (Part 2)-*, Report of Japan Highway Corporation Laboratory, pp.142-154, Nov. 1976. (in Japanese)
4. Iioka, Y., Higai, T., and Hosoda, K., *Experimental Study on Strengthening Methods for Reinforced Concrete Decks, -Steel Plate Bonding Technique (Part 3)-*, Report of Japan Highway Corporation Laboratory, pp.143-151, Dec. 1977. (in Japanese)

5. *Recommendations for Upgrading of Concrete Structures with Use of Continuous Fiber Sheets*, Concrete Library 101, Japan Society of Civil Engineers, July, 2000. (in Japanese)
6. Mufti, A.A., et al., New Canadian Highway Bridge Design Code design provisions for fibre-reinforced structures, *Can, J, Civ. Eng.*, 34, pp.267-283, 2007.
7. ACI 440 Committee, *Guide for the design and construction of externally bonded FRP systems for strengthening concrete structures*, ACI, July 2008.
8. Zureick, A., Ellingwood, B. R., Nowak, A. S., Mertz, D. R., and Triantafillou, T. C., *Recommended Guide Specification for the Design of externally Bonded FRP Systems for Repair and Strengthening of Concrete Bridge Elements*, NCHRP Report 655, TRB, 2010.
9. Yoshida, E., Murakoshi, J. and Tanaka, Y, Effect of Adhesively Bonded Steel Plates on Web Reinforcement for Reinforced Concrete Beams, *Proceedings of JCI*, pp.703-708, 2010. (in Japanese)
10. Yoshida, E., Murakoshi, J. and Tanaka, Y, Structural Testing of Deteriorated Reinforced Concrete Girders Strengthened by Externally Bonded Steel Plates, *Proc. JSCE Annual meeting*, V, pp.377-378, 2010. (in Japanese)
11. Tumialan, J. G., Belarbi, A., and Nanni, A., *Reinforced Concrete Beams Strengthened with CFRP Composites: Failure due to Concrete Cover Delamination*, Center for Infrastructure Engineering Studies, 99-01, Department of Civil Engineering, Univ. of Missouri - Rolla, March 1999.
12. Roberts, T. M., Approximate analysis of shear and normal stress concentrations in the adhesive layer of plated RC beams, *Structural Engineer*, 67(12), pp.229-233, 1989.
13. Jones, R., Swamy, R. N., Bloxham, J., and Bouderbalah, A., Composite behaviour of concrete beams with epoxy bonded external reinforcement, *Int J Cem Compos*, 2(2), pp.91-107, 1980.
14. Macdonald, M. D., *The flexural performance of 3.5m concrete beams with various bonded external reinforcements*, TRRL Supplementary Report 728, 1982.
15. Jones, R., Swamy, R. N., and Ang, T. H., Under- and over-reinforced concrete beams with glued steel plates, *Int J Cem Compos Lightweight Concr*, 4(1), pp.19-32, 1982.
16. Swamy, R. N., Jones, R., and Bloxham, J. W., Structural behaviour of reinforced concrete beams strengthened by epoxy-bonded steel plates, *Structural Engineer*, 65A(2), pp.59-68, Feb. 1987.
17. Jones, R., Swamy, R. N., and Charif, A., Plate separation and anchorage of reinforced concrete beams strengthened by epoxy-bonded steel plates, *Structural Engineer*, 66(5), pp.85-94, 1988. Correspondence from Roberts, 67(12), pp.187-188, 1988.
18. Oehlers, D. J., and Moran, J. P., Premature failure of externally plated reinforced concrete beams, *J. of Structural Engineering*, ASCE, 116(4), pp.978-995, April 1990.
19. Basunbul, I. A., Gubati, A. A., Al-Sulaimani, G. J., and Baluch, M. H., Repaired reinforced concrete beams, *ACI Materials J.*, 87(4), pp.348-354, July-Aug. 1990.
20. Oehlers, D. J., Reinforced concrete beams with plates glued to their soffits, *J. Structural Engineering*, ASCE, 118(8), pp.2023-2038, 1992.
21. Hussain, M. et al., Flexural behavior of precracked reinforced concrete beams strengthened externally by steel plates, *ACI Structural J.*, 92(1), pp.14-22, Jan.-Feb. 1995.
22. Baluch, M. H., Ziraba, Y. N., Azad, A. K., Sharif, A. M., Al-Sulaimani, G. J. and Basunbul, I. A., Shear strength of plated RC beams, *Mag. of Concrete Res.*, 47, No. 173, pp.369-374, 1995.
23. Swamy, R. N., Jones, R., and Charif, A., Contribution of externally bonded steel plate reinforcement to the shear resistance of reinforced concrete beams, *Repair and Strengthening of Concrete Members with Adhesive Bonded Plates*, SP-165, ACI, pp.1-24, 1996. (The session was held in 1992)
24. Sano, M., and Miura, T., A study on a design method for strengthening concrete members by steel-plate-bonding, *Proc. of JSCE*, 550, pp.117-129, Nov. 1996. (in Japanese)
25. Okamura, H., and Higai, T., Proposed design equation for shear strength of reinforced concrete beams without web reinforcement, *Proc. of JSCE*, 300, pp.131-141, Aug. 1980.
26. Niwa, J., Equation for shear strength of reinforced concrete deep beams based on FEM analysis, *Proc. of JCI 2nd Colloquium on Shear Analysis of RC Structures*, pp.119-128, Oct. 1983. (in Japanese)
27. *ACI 318 Building Code Requirements for Structural Concrete and Commentary*, ACI, 2005.

Fig. 1 Steel plates bonded to the soffit of decks and girders in a concrete bridge

Fig. 2 Steel plates bonded on the webs of concrete girders

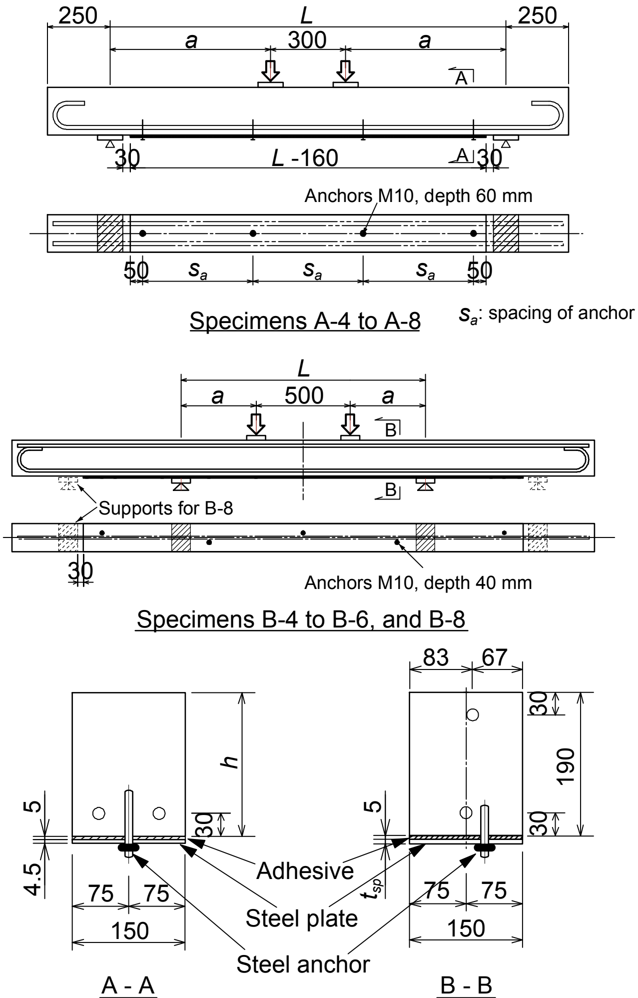


Fig. 3 Configurations of specimens in Series I

Table 1 Details and test results of specimens in Series I

Specimen	A-1	A-2	A-3	A-4	A-5	A-6	A-7	A-8	B-4	B-5	B-6	B-8
Steel plate strengthening	No	No	No	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes	Yes
Continuity of steel plate at support	—	—	—	No	No	No	No	No	Yes	Yes	Yes	No
Height of beam h, mm	190	190	190	190	190	190	250	330	190	190	190	190
Shear span a, mm	480	480	480	320	480	640	660	900	400	400	400	1000
Effective depth d, mm (after strengthening)	160	160	160	183	183	183	237	315	189	191	197	189
a/d	3.0	3.0	3.0	1.7	2.6	3.5	2.8	2.9	2.1	2.1	2.0	5.3
Longitudinal reinforcing bars	2-#7	2-#5	2-#6	2-#5	2-#5	2-#5	2-#7	2-#8	1-#5	1-#5	1-#5	1-#5
Ratio of reinforcement before strengthening, %	3.23	1.66	2.39	1.66	1.66	1.66	2.35	2.25	0.83	0.83	0.83	0.83
Ratio of reinforcement after strengthening, %	-	-	-	3.90	3.90	3.90	4.07	3.57	3.09	3.83	6.77	3.09
Doubly reinforcing bars	-	-	-	-	-	-	-	-	1-#5	1-#5	1-#5	1-#5
Thickness of steel plate t_{sp}, mm	-	-	-	4.5	4.5	4.5	4.5	4.5	4.5	6.0	12.0	4.5
Comp. cylinder strength of concrete, MPa	27.7	28.0	28.5	29.4	28.6	29.7	29.8	30.6	27.8	27.8	27.8	28.3
Splitting tensile strength of concrete, MPa	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.3	2.0	2.0	2.0	2.4
Yield point of reinforcing bars, MPa	369	390	374	390	390	390	369	362	348	348	348	348
Yield point of steel plate, MPa	-	-	-	300	300	300	300	300	306	318	330	306
Diagonal cracking load $P_{cr,ex}$, kN	88	77	79	78	75	73	121	158	90	112	131	86
Maximum load $P_{max,ex}$, kN	122	94	79	121	78	74	121	158	-	-	-	86
Type of failure	S	S	S	D	D	D	S	S	**	**	**	S†

Note: 1) Width of beams and plates : 150 mm, Thickness of adhesive layer : 5 mm
2) **: Loading was stopped at diagonal crack load.
3) A steel plate was installed on the soffit of Specimen B-8, after initial cracks developed due to cyclic loading when the shear span was set as 500 mm.
4) S : shear failure, D : failure due to cracking at the edge of steel plate,
† : Shear failure happened after the yielding of steel plate.

Fig. 4 Failure due to cracking at the edge of the bonded steel plate in Specimen A-6

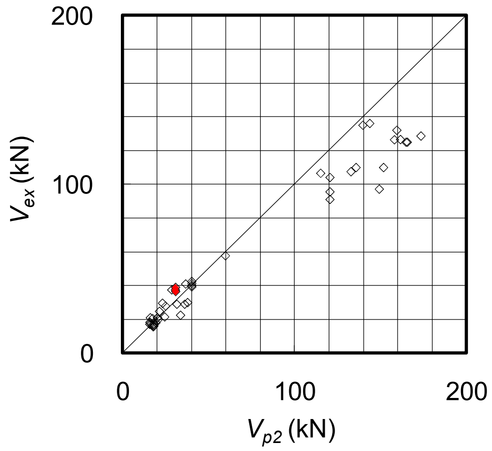
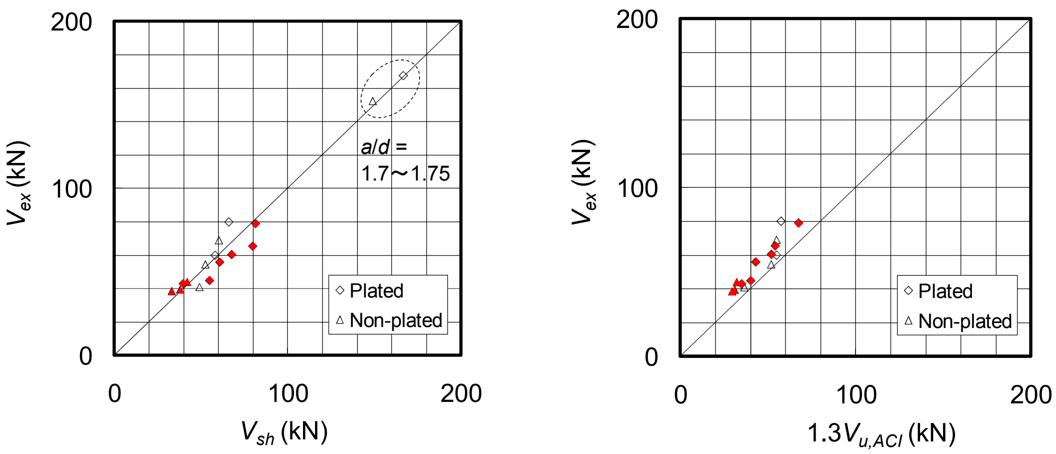


Fig. 5 Comparison between the measured shear forces at cracking at the edge of steel plate and the calculated shear force at debonding of steel plate

Note: Painted symbols indicate the results obtained from this study.



(a) V_{sh} (b) 1.3 $V_{u,ACI}$ (except the deep beam)

Fig. 6 Comparisons between the measured shear force of the plated beams at diagonal cracking and the calculated shear capacity for flexure-shear cracking.

Note: Painted symbols indicate the results obtained from this study.

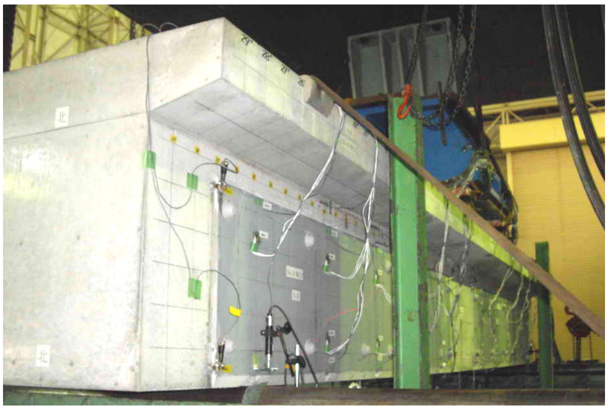
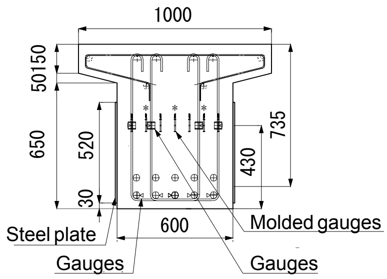


Fig. 7 Wing-type steel plates bonded to the sides of web in Series II



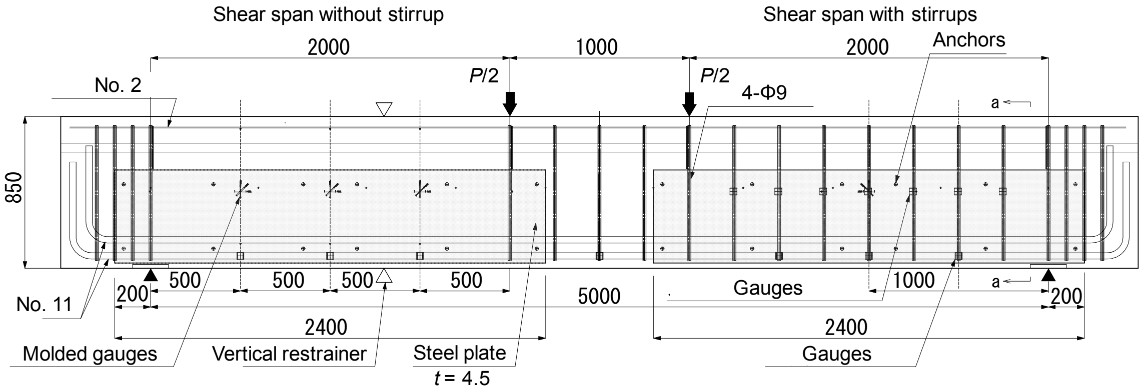
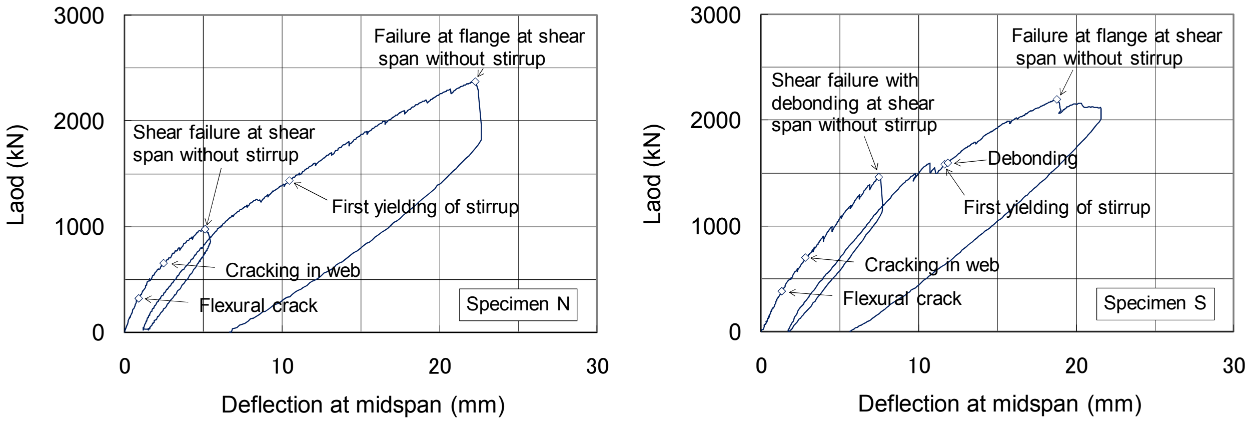


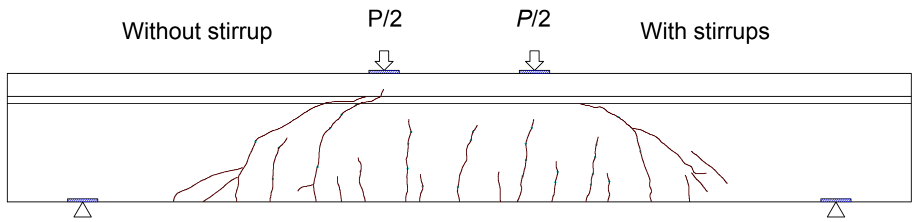
Fig. 8 Details and test setup of side-plated beams in Series II

Note: Steel plates were bonded on the web in Specimen S.

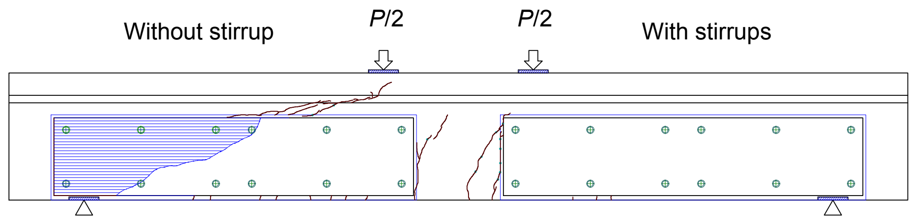


(a) Specimen N (no plate) (b) Specimen S (plated)

Fig. 9 Relationship between load and deflection of side-plated reinforced concrete beams



(a) Specimen N (no plate)



(b) Specimen S (plated)

Fig. 10 Cracks observed at shear failure at the shear span without stirrup
Note: Hatching indicates the area of debonding of the steel plates.

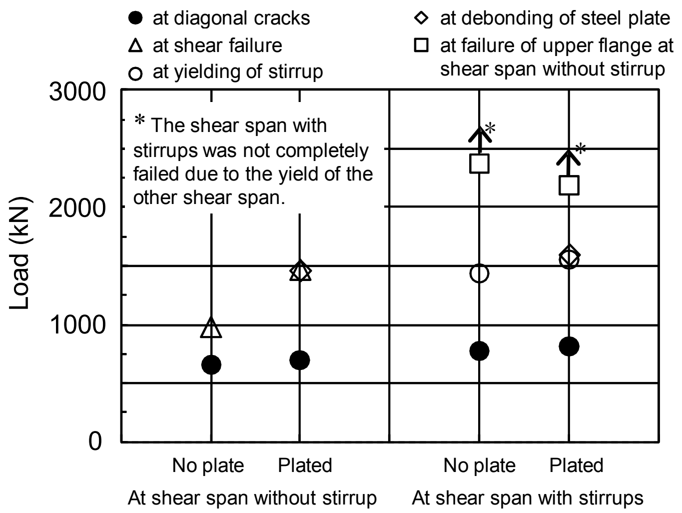


Fig. 11 Comparisons of the load at diagonal cracking, the load at debonding, and the maximum load in Series II

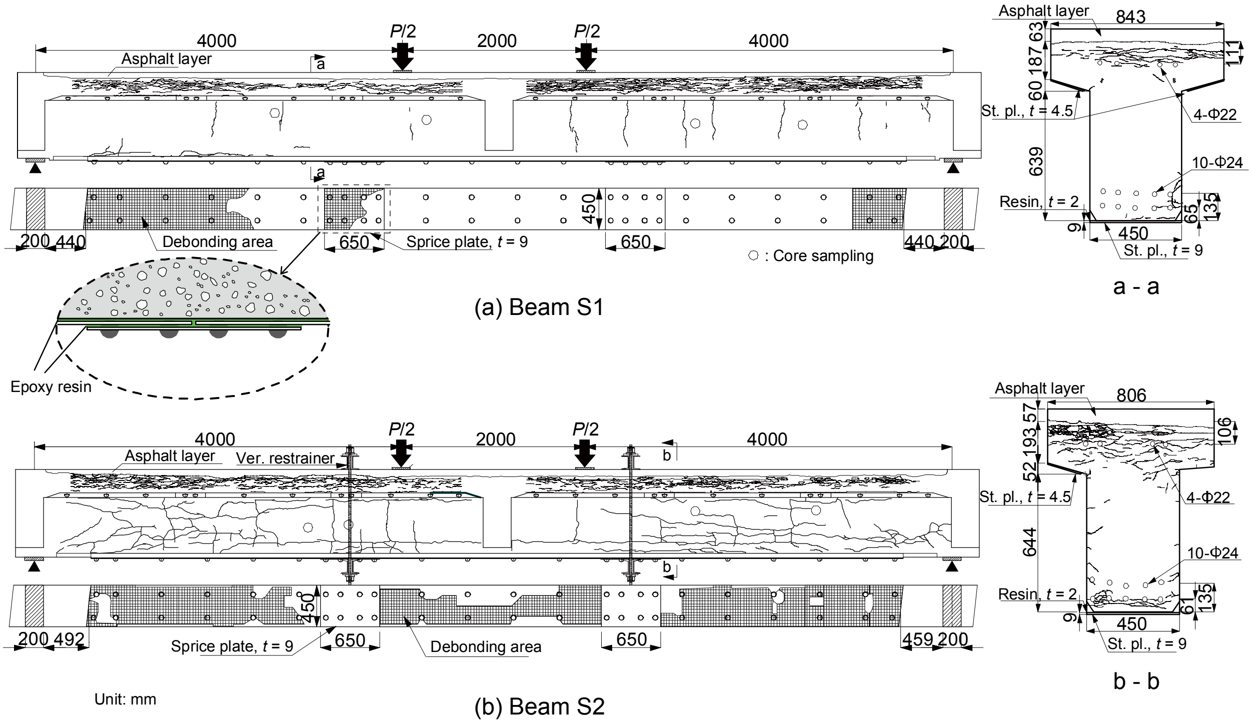


Fig. 12 Deterioration of old reinforced concrete beams with bonded steel plates

Note: Cracks in decks near cross beams are not drawn.
Cracks observed in cross sections were investigated after loading tests.

Table 2 Test results of concrete cores taken from the old beams

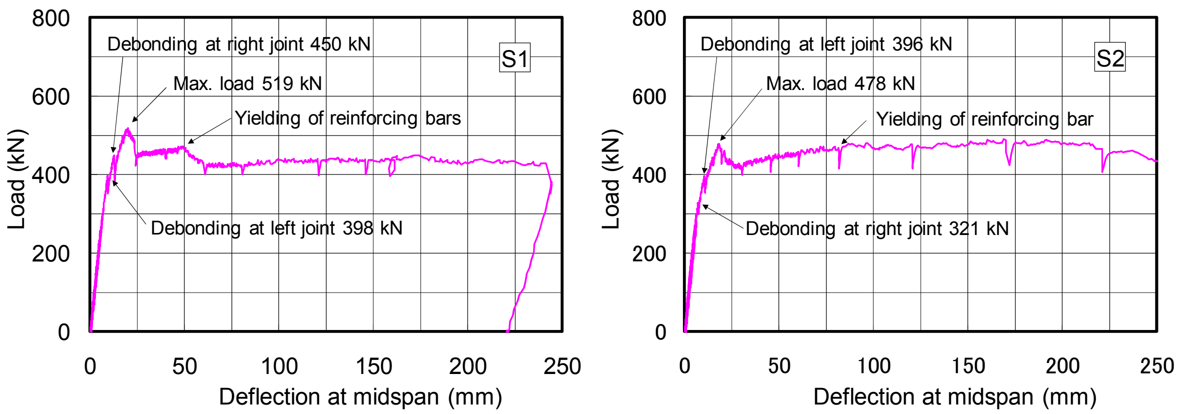
Beam	Core No.	Cylinder strength, MPa	Modulus of elasticity, GPa	Density, kg/m^3
S1	1	23.0	26.1	2390
	2 *	35.2	30.9	2400
	3	30.2	18.6	2360
	4	28.7	30.6	2380
S2	1 *	32.3	10.9	2400
	2	24.4	4.6	2400
	3	31.0	9.7	2430
	4	17.4	-	2420

*) The values of the cores were used for the calculation of the ultimate loads shown in Table 4.

Table 3 The results of tensile tests of reinforcing bars and steel plates

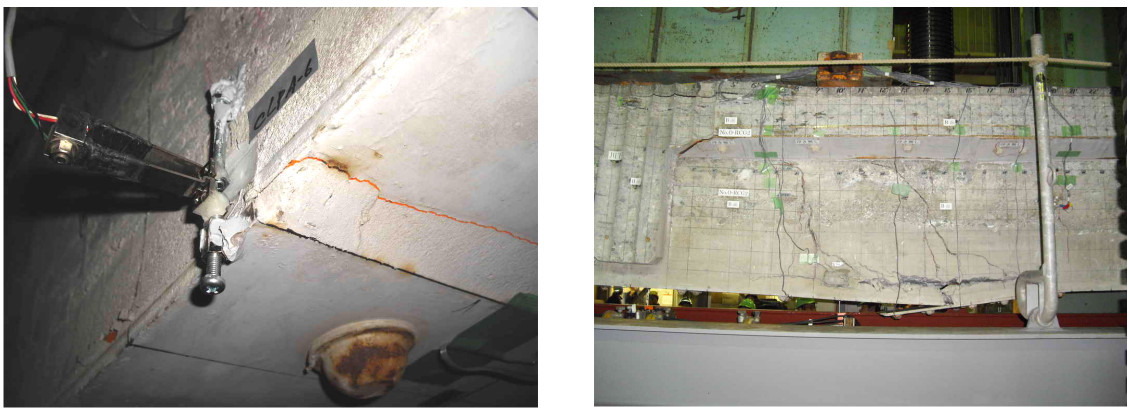
Types and sizes		Yield point (MPa)	Tensile strength (MPa)	Modulus of elasticity (GPa)
Stirrups	8 mm dia.	299	418	209
Doubly reinforcing bars	22 mm dia.	301	460	209
Main reinforcing bars	24 mm dia.	287	400	212
Steel plates	9 mm thick.	393	530	205

Note: All data were obtained from tensile tests, being the averages of three test pieces.



(a) Beam S1 (b) Beam S2

Fig. 13 Relationship between load and deflection at midspan of the old beams



(a) At sign of debonding of a splice plate (b) Flexural failure

Fig. 14 Failure observed in Beam S1

Table 4 Comparisons of ultimate loads of the old beams

Beam	Measured ultimate load, $P_{u,ex}$ (kN)	Calculated ultimate load, $P_{u,cal}$ (kN)	$P_{u,ex}$ / $P_{u,cal}$
S1	519	1130 (502)	0.46 (1.03)
S2	478	1110 (501)	0.43 (0.95)

Note: Parentheses indicate the calculated ultimate loads of reinforced concrete beams without steel plates.